Integrated Traffic Control with Variable Speed Limits and Coordinated Ramp Metering based on Traffic Stability

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Summary

For the last decades, congestion on highways has been a serious issue. Congestion causes more air pollution, accidents and delays, resulting in late arrival for employment or meetings. One of the main goals of the field of transportation engineering is to reduce or prevent delays and accidents caused by congestion, although, the last years it has been becoming more apparent that congestion cannot be completely solved. The main reason is simple; the highways are carrying too much vehicle miles, more than for it was designed

At first sight, an easy way to resolve congestion is the construction of new roads. By constructing new roads the capacity of the highway system will be higher and the number of traffic jams will be reduced, but is an expensive solution. Innovative solutions such as ramp metering and variable speed limits can be used to reduce congestion on a highway. In order to resolve or prevent congestion essential is the predictability of congestion.

Congestion is a result of a combination of three ingredients; a high traffic volume, a local perturbation and a spatial inhomogenity. Studies by Lee et al. (2000) and Treiber et al. (2000) have shown that the relation between these three ingredients can be predicted by traffic stability. The influence of a local perturbation (e.g. a lane change) on the flow can eventually cause congestion if the traffic flow is unstable. This validates that it is essential to measure the stability of traffic flow.

Two discussed stability indicators are able to determine the stability of traffic in free flow. A stochastic approach is able to determine whether traffic has entered the metastability regime (only large perturbations lead to congestion) and a reliability indicator, based on the variance in flow, is able to determine the stability.

To make sure congestion will not arise, traffic can be controlled. Two control strategies, ramp metering and variable speed limits, are discussed. Ramp metering strategies control the amount of vehicles entering the mainline. Literature has shown that local and coordinated ramp metering strategies are able to improve the efficiency of a highway. Variable speed limits can be used in order to homogenize traffic or to prevent traffic breakdown by limiting the inflow of a highway segment. The homogenization approach is, based on the definition of traffic stability, causing a lower capacity. The more complex approach, preventing breakdown, is able to reduce travel time as simulation studies have shown an improvement of travel time up to 50 % (Lu et al., 2011; Su et al., 2011).

As it was shown that ramp metering strategies and variable speed limits are able to reduce travel time and stability indicators are able to predict congestion, these two are combined for new control strategies. Based on a local ramp metering strategy, a correction factor is proposed based on traffic stability. If traffic flow is unstable fewer vehicles should be allowed to enter the highway, if traffic stable the correction factor is high. Further research is needed to determine the performance and the influence of infrastructure layout and flow characteristics on the correction factor.

An integrated control strategy, where the variable speed limit is determined before the ramp meter rate, is proposed based on the preventing breakdown approach. As the detector measurements show that traffic is getting unstable, the control is switched on. Here, the speed limit on a highway segment is based on the difference between the desired and the measured occupancy. After the variable speed limits are set, a model predictive control scheme is used to find the optimal ramp meter rate. Every detector interval (30 seconds) the control scheme minimizes the total travel time and maximizes the total traveled distance. These calculations are used for implementation.

The proposed strategies are implemented in the simulation environment in order to test their performance. The network, modeled in the microsimulation software Aimsun, is a part of the I-880 NB in California, USA. Implemented in the simulation software is the section from post mile 9

(Dixon Landing Road, Milpitas) to post mile 18.66 (Central Ave, Fremont). Five different strategies are compared; the uncontrolled case, the local ramp metering strategy ALINEA, the proposed variable speed limit strategy, the proposed coordinated ramp metering strategy and the proposed integrated strategy. Results show that the integrated strategy is best performing, improving the total time spent with 6 %. Other control strategies show less improvement and the ALINEA strategy shows even less performance. The results are in line with literature but show less performance, expectedi s that the short on-ramps are highly influencing the performance. Concluded is that the control strategy is able to resolve the congestion earlier.

Before considering application in real world, variable message signs are needed. Besides, the performance could be improved by increasing the prediction horizon or considering an extra constraint based on the reliability indicator. Essential is, testing the proposed strategy in a simulation model on several days.

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Contents

1 Introduction

For the last decades, congestion on highways has been a serious issue. Congestion causes more air pollution, accidents and delays, resulting in late arrival for employment or meetings. One of the main goals of the field of transportation engineering is to reduce or prevent delays and accidents caused by congestion. The last years, it has been becoming more apparent that congestion cannot be completely solved. The main reason is simple; the highways are carrying too much vehicle miles, more than for it was designed.

At first sight, an easy way to resolve congestion is the construction of new roads. By constructing new roads the capacity of the highway system will be higher and the number of traffic jams will be reduced. It has been becoming clear that it is impossible to construct extra roads to meet the demand of vehicle miles; new roads will cause a higher demand. Increasing the capacity of the existing road network, without the construction of new roads, requires innovative solutions. Traffic flow control, such as ramp metering, route guidance, driver information systems and variable speed limits, can be used to improve traffic flow, traffic safety and to reduce the environmental impact.

Basically, congested traffic is the result of a too high demand. If the demand for a highway section is exceeding the supply (bottleneck) a traffic jam arises. Empirical findings by Treiber & Kesting (2013) have shown that bottlenecks on highways is the result from a combination of three ingredients; high traffic volume, spatial inhomogenity and temporary perturbations of the traffic flow. These three ingredients come together in a bottleneck and causes a limited flow upstream of the bottleneck, approximately 5 to 20 % below its capacity (Lu et al., 2010b). So, the highway carries, during congestion, less vehicle miles than it was designed for.

Several methods have been proposed in the past to limit congestion and to improve the flow on highways. The last decade a lot of studies were done proposing ramp metering of an on-ramp and/or setting variable speed limits to homogenize or to limit the inflow of traffic. The most of the approaches are very complex but show an improvement of total travel time between 10 and 20 %. A simulation study of Lu et al. (2011) reported even a decrease of 50 % in travel time on the I-80 highway in California, USA.

Essential in proposing different traffic control strategies is the predictability of the network. If it is possible to predict where congestion arises and how long it takes before it disappears, it is more feasible to find a solution. Therefore, the most highways are supplied with detectors. These detectors measure several characteristics of a traffic flow and based on these characteristics it is possible to measure the congestion. More complex methods are required to predict the location and the 'amount' of congestion.

Studies by Lee et al. (2000) and Treiber et al. (2000) have shown that the type of congestion can be predicted using the fundamental diagram. Lee et al. (2000) made an empirical study of a highway, and found a relationship between the on-ramp and upstream highway flow and the type of congestion that arises. Treiber et al. (2000) further identified these types of congestions and proposed a theory for several macroscopic and microscopic models in which it is possible, based on the flow and stability, to predict which kind of congestion will arise. These studies have shown that based on traffic stability one is able to predict congestion. However, this theory has not been used previously for improving traffic flow and lowering the travel time. This report proposes an integrated traffic control strategy to improve travel time based on traffic stability.

2 Research Objective and Approach

Congestion is an undesired phenomenon on the highway and can be limited by constructing new roads or using innovative solutions such as ramp metering and variable speed limits. If it is possible to predict the congestion on the highway, it is more feasible to prevent or resolve congestion. It was shown (Lee et al., 2000; Treiber et al., 2000) that using the stability of traffic one can predict congestion. Goal of this research is providing a traffic control strategy based on traffic stability.

2.1 Research Objective

Studies by Lee et al. (2000) and Treiber et al. (2000) have shown that stability of traffic gives a prediction about the arising of congestion. Central in this report is measuring the stability with existing measuring techniques, and based on these measurements developing a traffic control strategy. As a wide range of traffic control strategies are available, only two strategies (ramp metering and variable speed limits) are used which have proven their performance. The research objective is defined as follows:

> *Developing a variable speed limit and coordinated ramp metering strategy based on traffic stability.*

As it is essential to measure the performance of the strategy, the control strategy will be tested on a part of the I-880 highway in California, USA in a simulation environment.

2.2 Research Questions

A central research question is formulated, derived from the research objective:

> *How can congestion be limited using a variable speed limit and coordinated ramp metering strategy based on traffic stability?*

In order to answer the central research question and to reach the research objective, the following sub research questions are formulated:

> *What is traffic stability and how can it be measured using existing measuring techniques?*

> *How can traffic stability contribute limiting the congestion on highways?*

> *How can variable speed limits and coordinated ramp metering improve traffic flow?*

> *What is the microsimulation performance of the control strategy on the I-880 highway in California, USA?*

> *How can the control strategy be best introduced in the real world given the simulation results?*

2.3 Research Methodology

This research consists out of two phases, a literature study and a simulation study. First of all, a literature study will be made. Literature gives insight into the concept of traffic flow theory and how the dynamics of congestion can be described. Literature will also gain insight into the current measurement techniques and whether these techniques are able to explain the arising of congestion. The theory of traffic stability will be investigated and stability indicators will be tested in order to measure the stability of traffic.

In order to control traffic, a lot of variable speed limits and controlled ramp metering have been developed the last years. Strategies for ramp metering, variable speed limits, as well as the combination of these strategies, will be discussed including their contributions to limit congestion on highways. The strategies can be used for expansion or as starting point for a new approach. Based on the theory of traffic stability and traffic control strategies, a strategy will be proposed to limit or prevent congestion.

As the strategy is formulated, the second phase of the research is a microsimulation study. Simulation is a powerful technique to determine the performance of the strategy. The simulation software Aimsun will be used to model traffic of the I-880 Northbound on one particular day. Goal of the simulation study is measuring the performance of the combined variable speed limit setting and the coordinated ramp metering strategy and comparing the performance with the uncontrolled case. It is clear that one day is not sufficient to give a good approximation of the results in real world, but just limited time is available.

To determine the performance of the strategy, first, the I-880 Northbound highway needs to be modeled into the simulation software. It is essential that the model mirrors real world behavior, therefore, the model needs to be calibrated. After calibration, simulations will be made comparing the strategy and the uncontrolled case. The performance will be based on efficiency; the less time individuals spent on the highway and the better the performance.

Finally, the outcomes of the literature and simulation study will be used to give recommendations for application of the algorithm in real world and it will be used to give recommendations for future research.

2.4 Research Relevance

2.4.1 Theoretical Relevance

Investigating how existing measurement techniques are able to determine traffic stability, including their limitations, could be contributing to the existing publications about traffic stability (summarized in Pueboobpaphan & van Arem (2010)).

Lots of literature is available about setting variable speed limits and/or using controlled ramp metering. On the other hand, no literature (with exception: Elbers (2005)) is available improving travel time based on the stability of traffic flow. This research might give new insights into setting speed limits and/or with coordinated ramp metering.

2.4.2 Practical Relevance

The proposed ramp metering and variable speed limit strategy could be used, if the model shows so, to improve travel time. This research could be helpful to decision makers considering application of variable speed limits and ramp metering to a highway, in order to limit congestion on the highway.

2.5 Institute: PATH

The research is conducted in the United States of America at the California Partners for Advanced Transportation tecHnology (PATH) in Richmond, CA. PATH was established in 1986 as the California Program on Advanced Technoloy for the Highway and is part of the Institute of Transportation Studies, University of California in Berkeley in collaboration with Caltrans. In 2011 PATH has merged with the California Center for Innovative Transportation. PATH's mission is to develop solutions to the problems of California's surface transportation systems through cutting edge research, divided into three program areas (Transportation Safety, Traffic Operation and Modal Applications). PATH has 45 full-time staff members and supports the research of nearly 50 faculty members and 90 graduate students.

2.6 Overview

The report is structured as follows:

Chapter 3 gives an introduction to traffic congestion. It describes how congestion can be modeled using three ingredients of the arising of congestion. The influence of the ingredients on congestion is described using traffic stability. In this chapter, the arising of congestion and the predictability of congestion using stability indicators are discussed.

Chapter 4, Traffic Control, focuses on improving the efficiency of the network. Based on traffic control objectives, control strategies using variable speed limits and ramp metering are compared and their performance in other studies are indicated.

The theory of traffic stability and traffic control is used, in Chapter 5, in order to provide a local ramp metering and an integrated control strategy.

The integrated control strategy is tested on the I-880 Northbound highway in California, in a simulation environment. The results are compared to other studies and several improvements are suggested.

Finally, Chapter 7 answers the research questions and discusses topics for further research.

3 Background

This chapter delivers a theoretical framework for modeling traffic flow, a discussion of congestion causes and, based on traffic stability and two methods to predict traffic congestion.

3.1 Congestion Causes

Empirical research (Schönhof & Helbing, 2004; Carlson et al., 2010) has shown that congestion (indicated by lower speeds and longer trip times (Bertini, 2006)) on the highway is the result of a combination of (i) high traffic volume, (ii) a spatial inhomogenity and (iii) a temporary perturbation. Considering an example of an on-ramp merging into a highway, a bottleneck can arise as the total traffic volume of the mainline and the on-ramp is high, the lane drops and an individual makes a lane changing maneuver (leaving the merging-lane). Clear is that congestion is caused by a stream of vehicles (macroscopic traffic characteristic), the infrastructure layout and an individual vehicle (microscopic traffic characteristic).

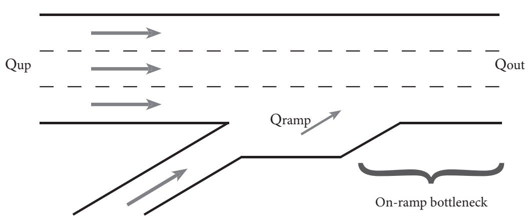


Figure 3.1 *Schematic indication of an on-ramp bottleneck. Three ingredients for congested traffic are available; high traffic volume ($Q_{up} + Q_{ramp}$), a spatial inhomogenity (merging lane drop) and perturbations caused by the merging vehicles.*

3.2 Microscopic Traffic Characteristics

In microscopic traffic theory each vehicle is considered individually. The microscopic approach is focusing on describing the detailed manner in which one vehicle follows another (Gartner et al., 2001), the longitudinal behavior of traffic, and on the lane changing behavior of traffic; the lateral driving behavior.

Considering vehicles are in the same lane of a road, longitudinal driving behavior describes the relationships between a follower and a leader. If the distance between the rear bumper of the predecessor and the front bumper of the following vehicle (gap) is too small, unsafe conditions can occur. In order to obtain a safe gap between vehicles, individuals can brake or change lanes (lateral driving behavior). To make sure the gap is large enough, one can change lanes to improve driving conditions, the so-called discretionary lane change (DLC). If the lane change is required due to e.g. a lane drop, one speaks of a mandatory lane change (MLC). As was indicated (Schönhof & Helbing, 2004) a MLC is caused by a spatial inhomogenity and is causing a temporary perturbation (two of the three congestion ingredients).

The process from considering a lane change to making the maneuver is modeled by Ahmed et al. (1996) and is described as a four step process:

1. Decision to consider a lane change; drivers who want to make a lane changing maneuver estimate the space they need and estimate the available space. Based on this comparison,

they decide to make a maneuver or to postpone it. The required space is dependent on several characteristics of the driver, the vehicle and the road. An individual has to perceive all the characteristics, before coming to a decision. Gipps (1986) formulated six factors (physically possible, location of obstructions, presence of designated lanes, intended turning movement, speed and presence of heavy vehicles) for those considering changing lanes.

2. Choice of a target lane; an individual is considering the possible target lanes. In case of an on-ramp the target lane is clear, one wants to change lanes to the left (for right-hand traffic countries).

3. Acceptance of gaps in target lane; a lane change will only take place if the given gap, to the predecessor and the following vehicle in the target lane, is acceptable (in perception of the subject vehicle). These acceptable gaps should be higher than the minimum acceptable gaps (critical gap), and differ for the lead (target predecessor) and lag (target follower) vehicle. Most gap acceptance models describe the acceptance of a gap stochastically. The critical gap is, based on the MLC-models from Ahmed et al. (1996) and Lee (2006), changing over time and dependent on the volume of traffic on the mainline, the average speed in the mainline, and, majorly, depending on the remaining distance to the end of the merging lane (in case of an on-ramp).

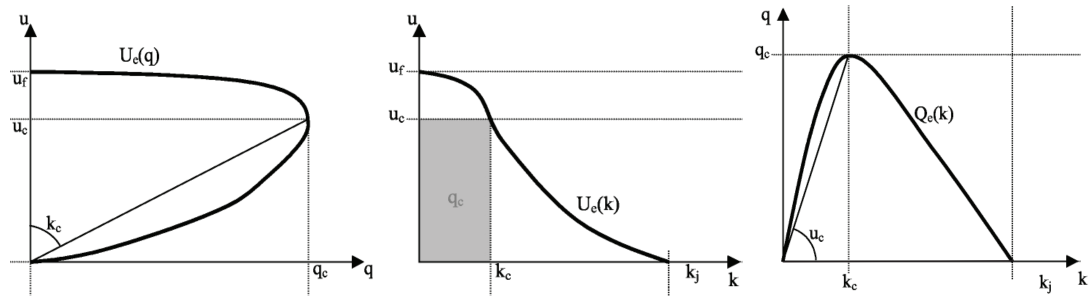
4. Performing the lane change maneuver; if a vehicle wants to make a lane change, has chosen the target lane, and considered the gaps as acceptable, the maneuver is made. In the target lane the lag vehicle wants to remain, after the lane change, a safe gap to its new predecessor and therefore a lane change (local perturbation) can cause a new lane change or a braking vehicle.

The model of Ahmed et al. (1996) identifies that making a MLC is highly dependent on individual behavior. Individuals, approaching the end of the merging lane (due to busy mainline traffic), are getting impatient, accept small gaps which triggers a braking maneuver in the target lane. The follower of the braking vehicle also tries to keep a safe distance to its predecessor; therefore a new braking maneuver is triggered. One will only observe this behavior if the vehicles in the mainline are impeded by each other, in other words: if traffic volume is high.

3.3 Macroscopic Traffic Characteristics

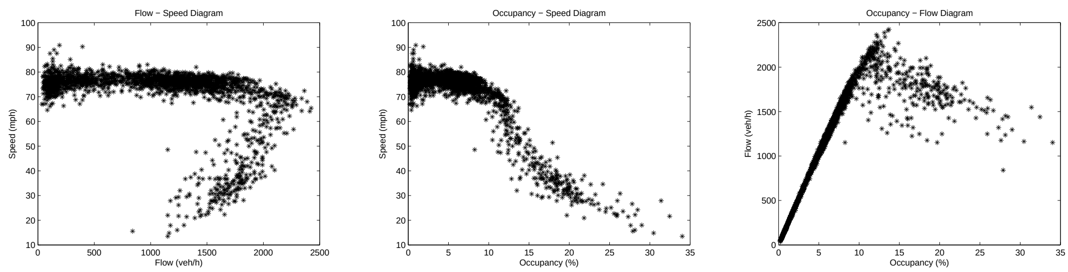
The total volume of traffic is one of the ingredients of the cause of congestion. Here, traffic is considered as a fluid, instead of considering each vehicle separately. The macroscopic traffic theory describes traffic on a system level and consists out of the flow rate (or volume), density and mean speed. Flow rate (q) is defined as the number of vehicles passing a point in a given period of time, usually expressed as an hourly flow rate per lane. The flow is based on vehicle counts in a time period. Traffic density is the number of vehicles occupying on a length of road. It gives an indication how crowded a road section is. The density can be found by making an aerial photograph of a road segment and counting the number of vehicles in a single, one mile (or kilometer) long, lane. The density differs from 0, indicating no vehicle on the lane, to a maximum value, representing vehicles are bumper to bumper. As indicated, it is hard to measure the density of a road section. A widely used technique in the USA, loop detectors, is measuring the occupancy (o). Occupancy is the fraction of time that vehicles are over the detector, and is based on the detector interval, the length of the vehicle, length of the detector and the vehicles speed. Occupancy and density are constants of each others. The final macroscopic traffic flow characteristic is the mean (or harmonic) speed, expressed in miles (or kilometers) per hour. The mean speed differs from the velocity. The mean speed u is the total distance traveled by all the vehicles in the region, divided by the total time spent in the region. It equals the sum of the speed of all vehicles divided by the number of vehicles.

Between the indicated traffic flow characteristics there exist a unique relationship (Greenshields et al., 1935). This fundamental relation of traffic flow theory provides a bond between flow, density (or occupancy) and mean speed: $q = ku$. This relationship is visualized in a fundamental diagram (Figure 3.2), and plays a crucial role in traffic modeling. The fundamental diagram separates the traffic fluid from all other fluids and provides a static relation between the three macroscopic traffic characteristics. Empirical fundamental diagrams (Figure 3.3) show a more scattered pattern of flow, (in this case) occupancy and speed. This is because a theoretical diagram makes two assumptions; traffic is stationary (flow rates do not change over time and space) and homogeneous (all vehicles are equal) (Immers & Logghe, 2002). However, in traffic flow modeling the theoretical fundamental diagrams are used, which can be validated by 'recognizing' the theoretical fundamental diagram in the empirical diagrams.



(a) Flow - speed (b) Density - Speed (c) Density - Flow

Figure 3.2 *Three related fundamental diagrams, assuming that traffic is stationary and homogeneous (Immers & Logghe, 2002).*



(a) Flow - speed (b) Occupancy - Speed (c) Occupancy - Flow

Figure 3.3 *Three related empirical fundamental diagrams, based on 5-minutes measurements of April 4, 2013 on the I-880 Highway in California. Lane 2 of detector 400309 (Caltrans, 2013).*

The fundamental diagram shows different regimes (in literature also called states or phases). Free flow traffic is a regime with light traffic conditions and vehicles are able to travel at their own desired speed (Maerivoet & De Moor, 2005), impeded by the maximum speed limit on the road segment. If the flow has reached a maximum value, the capacity flow is reached. Approaching a congested traffic regime on a highway stretch, more vehicles want to use the highway than capacity flow (demand is exceeding supply). To avoid a collision between two cars individuals have to brake, triggering a speed breakdown (congestion; significant speed drop). In case of a mean speed of 0, density has reached a maximum.

Basically, if the demand for a certain highway section is higher than capacity, breakdown will occur. In the example of Figure 3.1, in case of a breakdown: demand from the on-ramp and the mainline (upstream) is higher than downstream capacity. One can observe, at this bottleneck location, that the flow capacity upstream is higher than the flow capacity downstream of the bottleneck. The nominal capacity of the bottleneck is the maximum traffic flow that can be maintained at the bottleneck location if the traffic upstream is lower or equal to the capacity of the bottleneck (Carlson et al., 2010). If the arriving flow upstream is higher than the nominal capacity, the bottleneck is activated and congestion is formed (Carlson et al., 2010). Empirical findings have shown that the capacity of the activated bottleneck is lower (5 to 20 % (Carlson et al., 2010)) than nominal capacity.

Theoretically, as long as the total demand is not exceeding the nominal capacity, no congestion will arise. However, empirical findings (Koshi et al., 1983; Schönhof & Helbing, 2004) have shown that congestion does not significantly depend on the flow, but on the local perturbation. In contrast, the propagation of the perturbation does depend on the traffic flow. As it is desired to predict congestion (to prevent capacity drop); a relationship between the total flow, the local perturbation (a mandatory lane change due to the spatial inhomogenity) and the arising of congestion is required.

3.4 Traffic Stability

What the influence of a local perturbation is on the flow can be determined by the stability of traffic (Lee et al., 2000; Treiber et al., 2000). A stable traffic system is one that when perturbed from equilibrium state tends to return to that equilibrium state (Pueboobpaphan & van Arem, 2010). In other words; if traffic is stable, it is able to adapt to a lane changing maneuver of a vehicle. For the onset of congestion this has interesting implications; if the upstream traffic is unstable, an on-ramp merging maneuver can cause large perturbations in traffic (stop-and-go waves). If upstream traffic is stable, traffic flow is able to handle disruptions in traffic and to prevent breakdown (Elbers, 2005). This suggests that if one is able to indicate the stability of the traffic flow, it is possible to predict congestion and to prevent the capacity drop.

3.4.1 Classification of traffic stability

Treiber & Kesting (2013) made a classification of traffic stability, depending on the number of vehicles influenced and the amplitude of the perturbation. Based on the number of vehicles influenced, a distinction can be made into three types of traffic stability (Elbers, 2005; Pueboobpaphan & van Arem, 2010; Treiber & Kesting, 2011). Local (in)stability is concerned with the car-following dynamics of a single or a few vehicles. If a perturbation is introduced and the gap and fluctuations of the (one) follower increase in time, it is called locally unstable. If a platoon of vehicles is considered, one speaks of string (or platoon) (in)stability (Treiber & Kesting, 2013; Leutzbach, 1987). If a local perturbation eventually will damp out, the flow of traffic is string stable (Pueboobpaphan & van Arem, 2010). Traffic flow stability is not concerned with the car-following dynamics, but it concerns the disruptions in macroscopic characteristics (speed, density, occupancy and/or flow) of traffic (Elbers, 2005). Similarly to string stability; in flow stable traffic a perturbation will eventually damp out (Darbha & Rajagopal, 1999).

When the amplitude of a small perturbation increases in course of time, one speaks of instability. If the amplitude of a small perturbation eventually will damp out, one speaks of stable traffic. If small perturbations decay, but severe perturbations develop to persistent traffic waves, one speaks of metastability (or nonlinear instability (Yi et al., 2003)) (Treiber & Kesting, 2013). In other words; metastable traffic flow is stable for perturbations with small amplitudes and unstable for severe perturbations (Ossen, 2008).

The classification indicates that measuring traffic stability is essential. The stability of traffic is indicating whether a perturbation will fade out, or, in case of traffic instability, congestion arises.

3.4.2 Indicating Stability

If the stability of traffic (in free flow) can be determined, one can give a prediction of congestion. If traffic is stable, a perturbation will fade out, if traffic is metastable some perturbations will fade out some will not, and if traffic is unstable every perturbation will lead to a breakdown. For application in real world the measured data from the road side should be used to indicate the stability. Notice that the stability needs to be measured before the perturbation occur. This indicates that, in case of an on-ramp, the stability of the upstream flow should be measured.

Traffic is detected in many different ways, a widely spread used method in the United States is the usage of single loop detectors (a cross-sectional method). Single loop detectors provide every 20 or 30 seconds, occupancy and flow as raw measurements (Lu et al., 2010a). Based on the g-factor approach (Jia et al., 2001) the speed can be estimated, as well as the density. Therefore, assuming that the measurements of the detectors are available and good, used for indicating stability could be the (macroscopic characteristics) mean speed, flow, occupancy and/or density. In other words; only traffic flow (in)stability can be measured.

The available stability analysis methods describe traffic flow as stable or unstable. The most classical view indicates that traffic is unstable if the traffic density is above the critical density; otherwise it is stable (Pueboobpaphan & van Arem, 2010). This is in contrast to the classification of stability earlier described. For macroscopic traffic flow models, Yi et al. (2003) based their stability analysis on the nonlinear stability criterion using wavefront expansion. However, for real world application, it is necessary to use a stability indicator which is able to measure traffic instability.

Based on the definition of metastability, a stochastic approach could be used. In this case, traffic flow is metastable as the probability of breakdown is larger than 0. The probability is based on historical data and uses the Product Limit Method of Kaplan & Meier (1958) and the Weibull distribution function (Appendix A). Drawback of this indicator is that it is assuming stationary and homogeneous flow. In other words; at a certain occupancy, in different situations, it will determine the same stability. Based on stationary and homogeneous flow, this indicator could be used to macroscopically measure whether traffic flow is in the metastable regime (regime where chance of breakdown larger than 0). The metastable regime is necessary for traffic control; as traffic has entered the metastable regime, a large perturbation can cause a breakdown. Therefore, this control could be used as it is giving a fixed value for the border of the metastable regime.

To overcome the drawback of the stochastic indicator, the reliability indicator of Ferrari (1988) could be used (Ferrari uses the word 'reliability' for stability). Here, if a decrease in speed of a certain vehicle (in order to obtain a safe gap) can cause greater and greater decreases in the speed of the following vehicles, traffic flow is unstable (Ferrari, 1988). The indicator is based on the flow, the variance in flow and the (log normal) density (see Appendix A). It resolves the drawback of the stochastic stability indicator; the reliability indicator gives different values in different situations with the same demand. The stochastic approach could be used whether traffic has entered the metastable regime (based on historical data) and the reliability indicator could be used to measure the stability of traffic based on instationary flow. Here, this indicator could be used for more local traffic control as local ramp metering (section 5.1).

Note that both indicators should only be used for indicating stability in (uncontrolled) free flow.

3.5 Conclusion

Congestion on a highway is a result of a combination of high traffic volume, a spatial inhomogenity and a temporary perturbation. The influence of a local perturbation (e.g. a mandatory lane change) on the flow can be determined by the stability of traffic: if traffic flow is stable, a local perturbation will eventually damp out, if traffic flow is unstable, a local perturbation will cause a traffic breakdown. This gives proof that traffic stability should be measured using a stability indicator. To measure stability in free flow the reliability indicator (based on instationary flow) or the stochastic indicator (based on stationary flow) could be used.

4 Traffic Control

Previous chapter has shown that congestion on a highway is caused by a combination of high traffic flow, spatial inhomogenity and a temporary perturbation. To make sure the bottleneck will not be activated (and capacity will not drop) traffic can be controlled. Basically, the outflow of traffic can be improved using ramp metering, speed limits, route guidance, dedicated lanes (e.g. lane for high-occupancy vehicles), peak lanes, bi-directional lanes and by applying 'keep your lane'-signs. This report will only focus on two of these strategies; ramp metering and speed limits.

4.1 Traffic Control

4.1.1 Demand for Traffic Control

The increasing number of vehicles on the road has caused some serious congestion problems in the last decades. On a more local scale, the congestion forming at an active bottleneck causes a capacity drop and is blocking off-ramps. As discussed before, bottleneck activation leads to a drop in capacity. This is caused by accelerating vehicles from lower speeds (within the congestion) to higher speeds (downstream of the bottleneck) (Carlson et al., 2010). Besides, the tail of the formed congestion propagates upstream (Papageorgiou & Kotsialos, 2000). It is possible that the congestion covers on- and off-ramps upstream of the bottleneck. Here, vehicles wanting to leave the mainline are also delayed due to the congestion and contribute to an accelerated spatial increase of the congestion (Carlson et al., 2010). One solution to solve these problems is constructing new roads; adding lanes to existing roads or creating alternative new highways, both expensive solutions. Dynamic traffic control (or management) is an alternative; increasing the efficiency of the traffic network without constructing new roads (Hegyi, 2004).

4.1.2 Traffic Control Objectives

Traffic control may be applied for one or several objectives. Increasing the efficiency of the traffic network by minimizing the total travel time of an individual (user optimum) or minimizing the network travel time (network optimum) is one of these objectives. Another objective of traffic control could be safety; as accidents are in some cases the cause of traffic jams, a safer network will cause higher flows. Besides, in the congested flows more accidents arises and therefore less congestion will increase safety. On the other hand, lower densities combined with low speeds influence safety positively (Hegyi, 2004). This can cause a conflict with the efficiency objective. As traffic jams are not always prevented, it is valuable for drivers when travel time is predictable (Hegyi, 2004). If travel time is predictable, the arrival time can be estimated and choosing the departure time is easier. A, more recently developed, objective is lowering (negative) environmental effects of traffic. Emissions of a vehicle are influenced by the status of a vehicle, the technology of a vehicle, infrastructure and external conditions (Zegeye et al., 2009). Emissions per hour increase if the average speed increase, which is conflicting with reducing congestion. This report will only focus on traffic control to enhance the efficiency of a highway, in order to decrease the travel time of individuals.

The efficiency of the network is measured by the total travel time of an individual or network. As this report is focusing on resolving congestion the network travel time needs to minimized. The total travel time in the network is formulated as the total time spent (TTS). The total (travel) time spent is the sum of the travel time of all vehicles between two fixed locations. The total travel time plus the total waiting time (time for vehicles waiting to enter the network) is the total time spent.

TTS is a variable to compare the performance of different control strategies, the lower the TTS the better the performance of the strategy. If the TTS is lower, vehicles spent less time between entering and leaving the network.

Another criterion used to estimate the efficiency performance is the total traveled distance (TTD). TTD reflects the total distance traveled by the vehicles between two points of time (note difference with TTS). If the TTD is higher at a certain time, vehicles were able to make more miles. Note, that in the a simulation environment the total traveled distance at the end of the simulation period is always the same, as no extra vehicles are able to enter the network. The ratio of the total distance covered by the total time spent gives the mean speed; as the total time spent can be minimized and the total traveled distance is maximized; the mean speed of all vehicles is maximized (in drivers' perception: one can drive faster).

Although a strategy may perform better, still, it can show some undesired behavior. The strategy can cause new congestion areas and therefore the number of new traffic jams arisen should be identified empirically.

4.1.3 Principles of Traffic Control

The principles of traffic control are based on the three ingredients of congestion. As it is desired to prevent congestion, control is used to make sure the total flow is not exceeding capacity. Based on stability, it is desired to obtain a stable traffic flow. If traffic is stable, a disturbance will vanish without intervention. If traffic has entered the metastable regime, a large disturbance can cause congestion, the unstable regime indicates that any traffic disturbance will cause congestion. Therefore, if one wants to prevent traffic breakdown one should prevent traffic entering the unstable regime. If traffic has entered the metastable regime, control should be used to control perturbations.

4.2 Ramp Metering

To increase the performance of the network, several control strategies have been developed. Ramp metering is used to control the inflow of a ramp and based on occupancy; if the occupancy (lower than the critical occupancy) can be (approximately) continuous over time, no traffic jams will arise. Ramp metering can be implemented by installing traffic lights at the on-ramp of a highway, controlling the amount of traffic flow allowed to enter the highway (the ramp metering rate) (Figure 4.1). It can be used in order to increase or decrease travel time. When drivers try to bypass congestion on a highway it can be used to increase the travel time of these drivers (Middelham, 1999). Second, used in this report, a ramp metering strategy is used to preserve capacity flow on the mainstream and to avoid congestion (Kotsialos et al., 2002a). Referred is to Appendix B for more information about different ramp metering control strategies.

Basically, ramp metering strategies can be classified as static or dynamic, traffic responsive or feed forward, and local or coordinated (Hegyi, 2004). A fixed strategy, where the amount of vehicles allowed to enter the highway is based on historical demands, assumes, which is naive, that the demand is constant. This strategy is not able to adapt to variations in traffic (Hegyi et al., 2005). To overcome this (static) issue, the ramp metering strategy can be based on on-line data (traffic responsive strategy).

Traffic responsive strategies, such as ALINEA (Papageorgiou et al., 1991), base the ramp metering rate on measurements downstream of the ramp. If the rate is based on the measurements upstream it is called a feed-forward strategy, such as the demand-capacity strategy. The demand-capacity strategy, widely implemented in The Netherlands, bases the rate on a fixed value capacity. As it was shown before that the capacity of the network is not a fixed value (congestion can arise if total flow is lower than capacity), this strategy is naive. The ALINEA strategy bases the rate on a

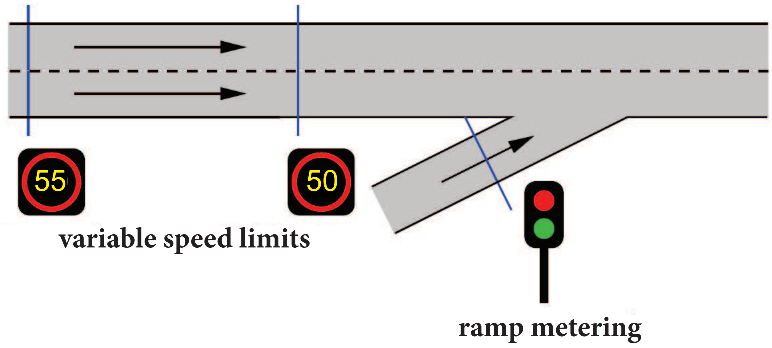


Figure 4.1 *Ramp metering and variable speed limits in order to control traffic (after Hegyi et al. (2005)).*

desired (occupancy) value. This strategy could be validated using traffic stability; as the desired value of occupancy lies in the stable regime congestion could be prevented. The summary of field results (Papageorgiou et al., 1997) shows that this traffic responsive strategy is outperforming other strategies, reducing travel time between 5 and 18 % and improving total traveled distance up to 3 %.

Local control strategies are focusing on controlling the ramp metering of a particular on-ramp. Coordinated ramp metering combines the use of several ramp meters to control the ramp flow on several on-ramps. Note that it is possible to (independently) control several ramps with a local control strategy. It was shown (Papageorgiou et al., 1997) that a coordinated ramp metering strategy is more complex and in case of recurrent congestion is not better performing than ALINEA. This validates that the local ramp metering strategy ALINEA is a standard to which other strategies can be compared and therefore will be implemented in the simulation environment, and will be implemented in the simulation software. Simulation tests from Hegyi (2004), Carlson et al. (2010), Lu et al. (2011) and Bhouri et al. (2011) have shown that other coordinated ramp metering strategies are able to improve TTS performance up to 25 %, the TTD shows very little or no improvement (Bhouri et al., 2011; Lu et al., 2011). Concluded can be that coordinated ramp metering strategies have the potential to improve performance.

4.3 Variable Speed Limits

Nowadays, a lot of highways are equipped with variable speed limit signs. These signs are currently used in The Netherlands to increase safety by lowering speed limits upstream of congested areas (Hegyi, 2004). Although, the signs could also be used to improve efficiency using a speed limit strategy.

Literature shows two approaches using speed limits; homogenization and preventing traffic breakdown (Hegyi et al., 2005). The idea of homogenization is that speed limits can reduce differences in speed and density (and thus flow). A field test (Van den Hoogen & Smulders, 1994) has shown that capacity is not improved by this approach. The speed variations and number of very small gaps decreases using this approach. This could be validated by traffic stability; as the variance in gaps decreases and the flow is high, this would decrease the opportunity to change lanes (less large gaps available) and this will cause more imprudent lane changes triggering a breakdown. The reliability indicator (Ferrari, 1988) supports this statement. This is in contrast with the statement of Van den Hoogen & Smulders (1994) and Zackor (1991) that homogenization causes a more stable traffic flow and, thus, a higher capacity. This can be validated because Van den Hoogen & Smulders (1994) do not define stability and Zackor (1991) shows only a very small (negligible) increase of capacity.

The second approach focuses on preventing or resolving traffic breakdown. This approach uses speed limits lower than critical speed to limit the inflow of the bottleneck (Figure 4.1). Several speed limit strategies have been developed. The SPECIALIST (Hegyi et al., 2008) strategy is the only strategy applied in real world, used to resolve moving jams and showed a gain of travel time of 35 vehicle hours per resolved jam (Hegyi & Hoogendoorn, 2010). Drawback of this algorithm is that the detection of moving jams (relatively short jams with an upstream moving head and tail) requires high dense installed detectors. Strategies used to prevent or postpone traffic breakdown (sometimes in combination with coordinated ramp metering) use a predictive control method, optimizing an objective. The different simulation studies (Carlson et al., 2010; Lu et al., 2011; Hegyi et al., 2005) show an improvement of TTS up to 50 % and an improvement of TTD up to 35 %. These studies show that a combination of (coordinated) ramp metering and variable speed limits are able to further improve the TTS up to 55 % (Lu et al., 2011; Hegyi et al., 2005).

It is clear that a local ramp metering strategy and variable speed limit strategies (in combination with coordinated ramp metering) are able to improve travel time significantly. The literature shows that a strategy using variable speed limits in combination with coordinated ramp metering shows the best performance. Condition is that a speed limit approach focusing on preventing or resolving traffic breakdown is used.

4.4 Conclusion

The increasing number of vehicles on the road has caused some serious congestion problems in the last decades. Traffic control is able to increase the efficiency of the traffic network without constructing new road. The discussed traffic control strategies, ramp metering and variable speed limits (or a combination of these two), have shown a (simulation) improvement in total time spent up to 55 %. As these control strategies are able to improve performance and stability indicators are able to predict congestion, these two principles are used in the following chapter to propose a control strategy.

5 Control Strategy based on Traffic Stability

In the previous chapters is indicated that if traffic flow is entering the metastable regime, traffic needs to be controlled. In uncontrolled cases, disturbances can eventually lead to a breakdown. Traffic flow entering the metastable regime can be determined using historical traffic data. As the main objective of ramp metering and variable speed limits is preventing traffic breakdown, traffic may never enter the metastable regime. As the metastable regime has a significant lower flow than the capacity flow, it is undesired letting traffic flow never entering this regime. Besides, not every disturbance in the metastable regime will cause a breakdown. The desired control is, therefore, maximizing the flow in the metastable regime and controlling the disturbance to make sure the disturbances fade out. As in the previous chapter is shown that ramp metering strategies and variable speed limit strategies are able to improve the efficiency, here, these strategies are used for traffic control.

5.1 Local Ramp Metering Strategy

A local ramp metering strategy is able to improve the efficiency of a highway. It is able to control the inflow of the on-ramp and is therefore able to (temporary) prevent traffic breakdown. As the local ramp metering strategy is based on the total flow, it does not take into account the influence of a perturbation. According to Ferrari (1988), if traffic is unstable, vehicles further upstream will have a larger decrease in speed. This leads to an intuitive correction factor to the ramp meter rate Elbers (2005): if traffic flow is stable more vehicles are allowed to enter the highway. The correction factor can be validated as the ramp metering strategy controls traffic based on the stability regime and the correction factor controls traffic based on the instationary stability (measured by the reliability indicator) of the flow:

$$r_{applied}(t) = r_{strategy}(t) * c \tag{5.1}$$

Here, $r_{strategy}(t)$ is the ramp meter rate (number of allowed vehicles to pass the traffic light in one hour) calculated by the local ramp metering strategy (based on stationary flow) at time step t. The correction factor (c) is based on the instationary stability and gives the applied ramp meter rate $r_{applied}(t)$. Referred is to Appendix B for the calculation of the ramp meter rate by local and coordinated ramp metering strategies.

Based on literature, it is suggested that the correction factor should be determined based on the following variables:

- Length on-ramp (fixed): Lee (2006) has shown that the gap acceptance is mainly influenced by the distance left to the end of the on-ramp. If the length of the on-ramp is small, vehicles will lower the gap acceptance more quick, which will lead to more disturbances. Lee (2006) has shown that more variables are influencing the gap acceptance, only the length of the on-ramp is introduced in this correction factor as Lee (2006) has shown that this is the major factor influencing the gap acceptance.

- Mainline shoulder lane flow (measured): if the mainline flow is increased, the variance in gaps increases and the average gap is lower (Vasconcelos et al., 2012; Brilon, 1988; Sullivan & Troutbeck, 1994); assuming that every vehicle will eventually make the mandatory lane change this is giving a higher probability of a serious disturbance.

- Stability of the shoulder lane (indicated): if the flow is high and the stability of the traffic flow is also high, the chance that the disturbance will lead to a breakdown is low.
- Stability all the other lanes (indicated): if traffic is facing a merging vehicle (leaving the on-ramp) this indicates whether this vehicle is able to make a discretionary lane change (to the left), and so on.
- Number of vehicles passing the ramp meter (calculated): a ramp metering model (such as ALINEA) will calculate the number of vehicles entering the highway, Ahn & Cassidy (2007) indicated that a disturbance is amplified by another local disturbance.
- Speed at the shoulder lane (measured): if traffic flow has a high speed at the shoulder lane, lag vehicles will face a lower speed of the lane changing vehicle and the amplitude will be higher.
- Homogeneous traffic (measured): If a variable speed limit is active, the variance in gaps decreases. Literature has shown that this will decrease the capacity of the bottleneck, it was shown that the reliability indicator is able to adapt to this situation.

The mainline shoulder lane flow and the homogeneous traffic and their influence on the stability are already indicated using the stability indicator of Ferrari (1988). The total stability of the the flow upstream can be determined as follows:

$$Stability = \frac{1}{n}\sum_{i=1}^{n}\phi_i \tag{5.2}$$

Here, ϕ_i is the stability of lane i (Appendix A), and n is the number of lanes excluding a designated lane (HOV-lane). Mostly, designated lanes are not allowed to use by all vehicles and give, therefore, not the opportunity to change lanes to this designated lane. One must notice that the parameters of the reliability indicator should be changed as variable speed limit control is active. The correction factor has the following form:

$$c = \beta\left(\frac{1}{n}\sum_{i=1}^{n}\phi_i\right) + (\alpha_1 l - \alpha_2 \Delta v) \tag{5.3}$$

Here, l gives the length of the on-ramp, it is the length of the ramp entering the mainline and where this lane drops. Δv gives the speed difference between the most right line (the merging lane) and the shoulder lane.

Where α_1, α_2 and β are control parameters. α_2 has a negative sign as the speed difference is negatively influencing the chance of a serious disturbance. Note that the ramp meter rate is based on measurements downstream of the on-ramp and the correction factor is based on measurements (stability) upstream of the on-ramp. Assuming that the measurements are made just upstream of the bottleneck, indicating that vehicles are only able to make a single lane change, stability will not change before facing the merging lane. The exact location of the detectors, influence of the variables and the control parameters is topic for further research before used as a control strategy. The performance of the strategy can, therefore, not be predicted. A, more simple, correction factor (Elbers, 2005) shows an improvement up to 15 % in total travel time in comparison with the ALINEA strategy. The correction factor of Elbers (2005) is based on microscopic measurements and is therefore not a proper indication of the performance of the proposed strategy in this section.

5.2 Integrated control

The efficiency of the road network can be improved by using variable speed limits lower than the static speed limit. Integrated control, using variable speed limits and ramp metering, have shown a (simulation) improvement in total time spent up to 55 %. Therefore, an integrated strategy is proposed using both control strategies. To integrate these two control strategies three possible ways exist (Lu et al., 2011):

- Determine ramp metering rate before determining variable speed limits;
- Determine variable speed limits first before determining ramp metering rate;
- Determine ramp metering and variable speed limits simultaneously.

The third approach is more complex, a method for this strategy is proposed by Ghods et al. (2010), using a game theory approach. The first approach has some practical implications as highways already have implemented ramp meters (Lu et al., 2011). The second approach is used by Su et al. (2011) and Lu et al. (2011), showing an improvement of 55 % in total travel time. As this approach has shown it capability to improve the efficiency, it is used, here, as a starting point.

5.2.1 Speed Limit Design

Assume a bottleneck on a highway caused by an on-ramp. Variable speed limits are able to limit the inflow of the mainline. As it is desired to differ the speed limit of different part of the highway (only a small section of the highway is a bottleneck and only the inflow needs to be controlled), the corridor is divided into N links ($m \in \{0, 1, 2, \dots, N-1, N\}$). Each link has a set of loop detectors (for measurements), one on-ramp (as congestion is caused by a spatial inhomogenity) and may contain off-ramps. Although more loop detectors may be available in the corridor, the speed limit design does not use all these detectors for now. Assumed is that it is possible to set one speed limit per link.

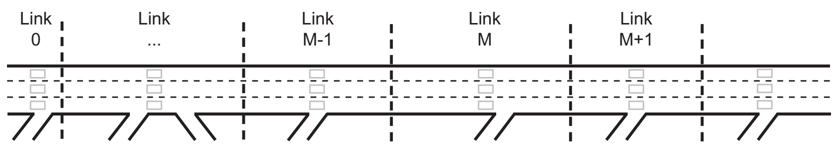


Figure 5.1 *Schematic corridor is divided into N links, every link with one set of detector and an on-ramp. The link where the congestion is detected is link M.*

Here, the strategy is activated if traffic is approaching the metastable regime. This link is set as bottleneck link M and will be controlled. In practice, the actual location of the bottleneck, caused by an on-ramp, is near the end of the on-ramp. Due to implementation reasons the entire link is set as bottleneck (Figure 5.1). Based on to the local ramp metering strategy ALINEA, the speed limit in the link upstream ($M-1$) of the bottleneck is based on the desired occupancy (o_c) and the measured occupancy ($\hat{o}$) (Su et al., 2011) in the bottleneck link. The set speed limit $u_m(k)$ is based on these variables and is a responsive strategy including a regulation parameter (ζ). The speed limit in the link upstream of the bottleneck can be calculated as new measurements are available from the loop detectors (every T seconds). The time interval used is kT.

The equation used to set the speed limit in the link upstream of the bottleneck ($M-1$) is as follows (Su et al., 2011):

$$u_{M-1}(k) = u_{M-1}(k-1) + \zeta(o_c - \hat{o}(k-1)) \tag{5.4}$$

This equation indicates that if the measured occupancy is exceeding the desired occupancy, the speed limit should be lowered in order to limit the inflow of the bottleneck link (to make sure traffic will not get unstable). As this variable speed limit setting equation is only setting the speed limit for one link ($M-1$), this may cause an irresponsible speed drop between two links ($M-2$) and ($M-1$). Therefore, the speed limits in the links upstream are gradually decreased from upstream to downstream (see Figure 5.2). Assumed is that the most upstream link ($m = 0$) is in free flow and cannot be controlled (speed limits here could influence links outside of the considered section). Also, links downstream of the bottleneck and the bottleneck itself ($m \geq M$) are not controlled by speed limits. The downstream links are not controlled as there is not demand for control (no approaching congestion detected), the bottleneck link is not controlled as only the inflow of the bottleneck (supply) needs to be controlled.

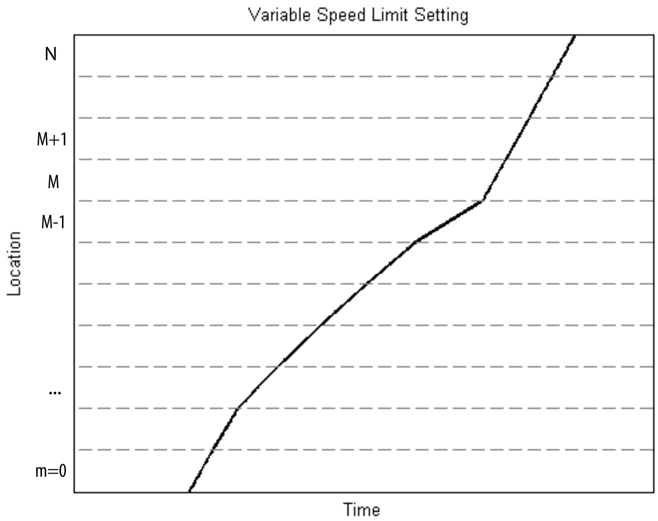


Figure 5.2 *Schematic visualization of the variable speed limit strategy where the speed limit in a link m is gradually decreased up to the bottleneck link M. Downstream links of the bottleneck, the most upstream link ($m = 0$) and the bottleneck link itself, are not controlled.*

The following equation assumes that the speed limit in the upstream link is equal to the static speed limit (V_f) of the highway:

$$u_0(k) = V_f \tag{5.5}$$

The variable speed limit for each link is based on interpolation between the free flow speed (in u_0) and the speed in the link just upstream of the bottleneck. It can be determined as follows (Lu et al., 2011):

$$u_m(k) = u_{m-1}(k) + \max\{-\Delta u, \min\{(\eta\alpha_m(k) + (1-\eta)\beta_m)[u_{M-1}(t) - u_0(k)], 0\}\} \tag{5.6}$$

Here, the maximum speed limit difference between two links is Δu (e.g. 5 mph). The speed limit in a link is, due to this equation, always lower or equal than the adjacent section upstream, and higher or equal than the section downstream. The speed limit in a section should be lower if the on-ramp demand (d_m) is higher or if the on-ramp length $L_{m,o}$ is lower. If on-ramp demand is high, the speed limit should be lower to create more 'space' for the on-ramp flow (Appendix B). Besides, if traffic is not able to leave the on-ramp a queue will grow and may be spill back to upstream adjacent infrastructure (outside of the network).

Therefore, Lu et al. (2011) and Su et al. (2011) have defined α and β based on the length of the on-ramp, the fixed capacity (Q_m) and the flow of the link ($q_m(k)$). η is used as a control parameter, prioritizing the mainline or the on-ramp flow.

$$\alpha_m(k) = H(Q_m - q_m(k)) \tag{5.7}$$

α causes a lower speed limit if the available 'space' is low. β causes a lower speed limit if the on-ramp length is low:

$$\beta_m = H(1/L_{m,o}) \tag{5.8}$$

To allow more vehicles to be injected from the on-ramp the speed limit reduction at that link should be greater (Lu et al., 2011). Besides, if the available space is low on a link, the speed limit should be lowered. If the speed limit reduction is greater, would implicate that H is increased. The harmonic function calculates the ratio of a link based on all other controlled links. Let $\mathbf{x}=[x_1, x_2, \ldots, x_n]$ be a real vector, then (Su et al., 2011):

$$H(x_m) = \frac{\frac{1}{x_m^2}}{\sum\limits_{\mu=1}^{M-1} \frac{1}{x_\mu^2}} \tag{5.9}$$

Assume no ramp metering is active on the link. The flow leaving the on-ramp ($R_m(k)$) is restricted by the demand of the ramp, capacity of the ramp ($Q_{m,o}$) or 'space' at the link (vehicles until capacity) (Su et al., 2011). The 'space' at a link is the capacity (Q_m) of a link minus the net measured inflow of the link (measured outflow of the upstream link $\hat{q}_{m-1}(k-1)$). Here is assumed that the off-ramp is located downstream of the on-ramp (within a link). This indicates that the 'space' at the link is restricted by the mainline inflow and the capacity.

$$R_m(k) = \min\{d_m(k), Q_{m,o}, Q_m - \hat{q}_{m-1}(k-1)\} \tag{5.10}$$

The expected flow of the link is the outflow from the previous link (measured: $\hat{q}_{m-1}(k-1)$) plus the on-ramp flow $R_m(k)$ minus the off-ramp flow $s_m(k)$ (Lu et al., 2011). This indicates that the speed limit in a link is updated based on the local (loop detectors) measurements.

$$q_m(k) = \hat{q}_{m-1}(k-1) + R_m(k) - s_m(k) \tag{5.11}$$

Here, as a link traffic is approaching the metastable regime, traffic can breakdown and needs to be controlled. The strategy bases the speed limit upstream of the bottleneck on the difference between the desired and the measured occupancy in the bottleneck link. However, the perturbations are not controlled and breakdown upstream of the bottleneck is still possible. To limit the chance of breakdown in a link, the ramp metering rate on a link should be controlled.

5.2.2 Optimal Ramp Meter Rate

After the variable speed limits are set in the links upstream of the bottleneck a model predictive control scheme is used to find the optimal ramp meter rate. Note that, without setting a speed limit, the predictive control can also be used (instead of speed limit the measured mean speed should be used).

Model Predictive Control

A model predictive control (MPC) scheme is used to find the optimal ramp metering rate (Camacho et al., 2004; Hegyi, 2004). The MPC controller (Figure 5.3) uses a linear traffic model and optimizes the control signal. This signal is applied to the traffic process (applied ramp metering rate) until new data is available. With the new data the signal is re-optimized with a shifted time horizon. In MPC, at each time step (every time new measurements are available), the optimal ramp meter rate is computed (using the simplex method) over a (finite) prediction horizon N_p. Assuming the inflows of the network are in the next time steps the same as the measurements, the MPC scheme calculates how many vehicles are allowed into the network to maximize the efficiency. As traffic situations change rapidly, only the ramp meter rate for the next time step is applied. In the next time step ($k+1$) a new optimization is performed, whereby the prediction horizon is shifted one step further, the so-called rolling horizon.

As the speed limits are set, the optimal ramp metering rate can be determined. The ramp meter rate ($r(k)$) is to be predicted over the predicted time horizon $k+1, \ldots, k+N_p$. Only the optimal

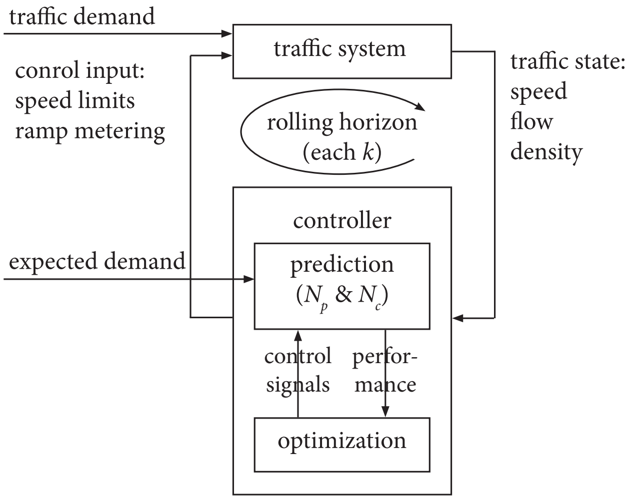


Figure 5.3 *Schematic Model Predictive Control scheme, after Hegyi et al. (2005)*

ramp meter rate until the control horizon N_c are used, other calculations are thrown away. Per time step the following ramp meter rates are calculated, note that here the ramp metering is also applied in the bottleneck link (assuming the on-ramp is cause of the bottleneck).

$$r = [r_1(k+1), \ldots, r_1(k+N_p), \ldots, r_M(k+1), \ldots, r_M(k+N_p)]^T \tag{5.12}$$

The predictive control takes irregular conditions into account, and the prediction is based (as will be shown in the next section) to predict traffic conditions if infrastructure changes. Thereby, the model predictive control makes frequent recalculations and therefore the control is updated frequently if traffic behaves different than expected (Schreiter, 2013).

For the prediction of the traffic situation in the next time steps, the second order model METANET (Messner & Papageorgiou, 1990) is used. The motorway network is represented by a directed graph whereby the links of the graph represent motorway stretches with no on- or off-ramps and no major changes in geometry. Each link has all macroscopic characteristics (Figure 5.4):

- Traffic density $\rho_m(k)$(veh/mi/lane) is the number of vehicles in link m at time kT divided by the length of the link L_m and by the number of lanes λ_m.
- Mean speed $v_m(k)$(mi/h) is the mean speed of the vehicles in link m. In case of variable speed limit, assumed to be equal to the calculated speed limit.
- Traffic flow $q_m(k)$(veh/h) is the number of vehicles leaving link m, divided by T (Loop detectors give measurements per 20 or 30 seconds).

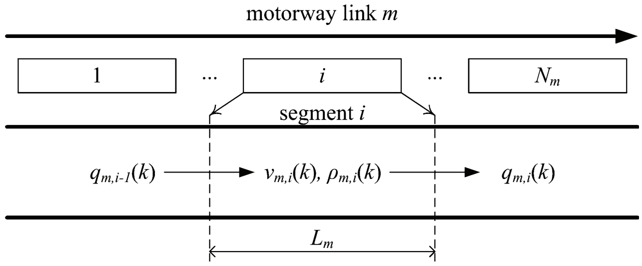


Figure 5.4 *The original METANET (Messner & Papageorgiou, 1990) discretized motorway link, after Kotsialos et al. (2002a).*

For the prediction of the macroscopic characteristics in the next time step the measured data of the loop detectors is used. Each time step k the macroscopic characteristics are calculated (Payne, 1971), where the density in the next time step $\rho_m(k+1)$ is based on the current density ($\rho_m(k)$), the inflow ($q_{m-1}(k)$), the outflow $q_m(k)$, length of the on-ramp (L_m), number of lanes λ_m and the time step kT:

$$\rho_m(k+1) = \rho_m(k) + \frac{T}{L_m\lambda_m}(q_{m-1}(k) - q_m(k)) \tag{5.13}$$

Adding an on- and off-ramp (respectively $r_m(k)$ and $s_m(k)$) to this equation gives the following:

$$\rho_m(k+1) = \rho_m(k) + \frac{T}{L_m\lambda_m}(q_{m-1}(k) - q_m(k) + r_m(k) - s_m(k)) \tag{5.14}$$

The flow can be determined linear, therefore the density in the next time step is calculated as follows (Messner & Papageorgiou, 1990):

$$\rho_m(k+1) = \rho_m(k) + \frac{T}{L_m\lambda_m}(\lambda_m\rho_{m-1}(k)u_{m-1}(k) - \lambda_m\rho_m(k)u_m(k) + r_m(k) - s_m(k)) \tag{5.15}$$

Clear is that the density in the next time step is a linear process, where the detector interval T, length of the link L_m and the number of lanes λ_m are fixed values. Here, assumed is that the speed limit in a link $u_m(k)$ is already calculated. The ramp metering rate are set as unknown and will be set using the predictive control scheme.

Note that every link has detectors available. Each time step the macroscopic characteristics of a link at time k are, therefore, known. As each time step new measurements are available, the model is taking rapid changing traffic situations into account. Assumed is here that the loop detectors give information for the entire link.

The previous equations have determined the influence of the ramp metering rate on the mainline density. Besides influencing the mainline density, the ramp meter rate is also influencing the queue at an on-ramp.

The on-ramp demand ($d_{m,o}$) at link m is forwarded into the network. The queue at an on-ramp is the old queue $w_m(k)$, the demand of the on-ramp ($d_m(k)$) minus the on-ramp leaving flow ($q_{m,o}$). In case of ramp metering rate, the on-ramp leaving flow is the ramp meter rate (Messner & Papageorgiou, 1990).

$$w_m(k+1) = w_m(k) + T[d_m(k) - q_{m,o}(k)] \tag{5.16}$$

Due to this linear model, the macroscopic characteristics of the highway can be calculated. The unknown variables are the ramp meter rates for all the links until the prediction horizon. The optimal ramp metering rate will be calculated using an objective function.

Objective Function

As the goal of the strategy is to maximize performance, the total time spent needs to be minimized and the total traveled distance needs to be maximized. First, to obtain the optimal ramp metering rate the total time spent, in all speed limit sections and the bottleneck, need to be minimized. Due to this optimization function the ramp metering rate is set such that vehicles spend as little as possible time on the highway. Here it is important to also take the time delay due to on-ramp queue into account; this is the number of vehicles waiting times the time interval. Minimizing this value minimizes the total time spent and avoids all traffic waiting at the on-ramp (Lu et al., 2011; Hegyi, 2004; Hegyi et al., 2005; Carlson et al., 2010).

$$TTS = T\sum_{j=1}^{N_p}\sum_{m=1}^{M} L_m\lambda_m\rho_m(k+j) + T\sum_{j=1}^{N_p}\sum_{m=1}^{M} w_o(k+j) \tag{5.17}$$

The total traveled distance should be maximized as to maximize the link flow (Lu et al., 2011):

$$TTD = T\sum_{j=1}^{N_p}\sum_{m=1}^{M} L_m\lambda_m q_m(k+j) \tag{5.18}$$

The optimization problem becomes (Su et al., 2011; Lu et al., 2011):

$$\min J = TTS - \sigma TTD \tag{5.19}$$

Where σ is a factor to balance the TTS en TTD.

Constraints

The designed ramp meter rate needs to satisfy a set of (technical) constraints to avoid unrealistic and undesired situations:

- The queue length may not exceed the length of the on-ramp. Here formulated as: the number of waiting vehicles may not exceed the maximum possible vehicles on the on-ramp (length times maximum density) :
  $0 \leq w_m(k+j) \leq L_{m,o}\rho_J$
- The ramp meter rate should not be higher than the ramp demand ($d_m(k)$), ramp capacity ($Q_{m,o}$) or space available in the mainline (this validates equation 5.16) (Lu et al., 2011):
  $0 \leq r_m(k) \leq \min\{d_m(k), Q_{m,o}, \lambda_m Q_m - \hat{q}_{m-1}(k)\}$
- The density in a section may not exceed the jam density (unrealistic situation):
  $0 \leq \rho_m(k) \leq \rho_J$

Algorithm

The proposed strategy assumes a bottleneck link M, the location of this bottleneck is based on the detection of congestion. The strategy is more adaptive for non-recurrent congestion (e.g. accidents) if it is dynamic (no fixed bottleneck location M. An algorithm is developed:

1. **Congestion detection**: After measurements, test whether a the occupancy is exceeding a threshold (based on metastability or intuitive value).If several links are congested, pick the most downstream area. If the control strategy is already active and still congested, go to step 2. If no congestion is detected, wait for next measurements.
2. **Set Variable Speed Limits**: Based on the occupancy of the bottleneck link, set the speed limits upstream of the bottleneck. For application of the control scheme it is necessary that there are enough links upstream available for control. As the maximum speed limits between two links is set as Δu, indicating that the minimum speed limit in link $M-1$ is $\max = \{u_{min}, (u_{max} - (M-1)\Delta u)\}$. Go to step 3.
3. **Set Ramp Meter Rate**: Based on Model Predictive Control and the linear density dynamics, set the ramp meter rate for the links upstream of the bottleneck as well as for the bottleneck link itself. Apply the speed limits and the ramp metering rate, go to step 1.

Using this algorithm, allows the designer of the strategy to select the following tuning parameters:

- $o_{start,m}$ threshold which switches the control on/off, could be different per link and is based on the metastability principle.
- $o_{c,m}$ desired occupancy in a link.

- η: prioritizing the mainline or the on-ramp flow. Maximum value is 1 and minimum value 0. A value higher than 0.5 is prioritizing the mainline flow.
- σ: balance the optimization of TTS and TTD.
- u_{max}: the maximum allowed speed limit, assumed is the fixed speed limit of a highway.
- u_{min}: the minimum allowed speed limit.
- Δu: maximum speed limit difference between two links.
- ζ: a control parameter to set the 'aggressiveness' of the control.
- N_p: setting the prediction horizon of the Model Predictive Control.
- N_c: the control horizon.

5.2.3 Discussion

Note that the strategy is only using stability of a traffic flow switching the control on or off. An extra constraint could be added to make sure traffic flow will not get unstable and a local perturbation will cause a traffic breakdown. The maximum number of allowed vehicles should be based on the reliability indicator (Ferrari, 1988). This constraint could be based on measurements of the detectors just upstream of the on-ramp within a link. As the linear density dynamics do only take into account one set of detectors per link, it is not possible to calculate the expected stability in the next time step. If the linear density dynamics would also calculate the traffic characteristics upstream of the on-ramp, it is possible to calculate the expected stability. Here, further research is needed setting the indicator parameters (which are speed limit dependent), and the location of the upstream detectors. The expected stability could be used as an objective (maximizing stability) or as constraint, making sure the stability is not too low (setting a threshold). Expected is, using the stability as a constraint, the chance of breakdown reduces. This could positively influence the total time spent and total traveled distance, as the breakdown and capacity drop can be prevented.

The road segment is divided into links, further dividing the model into smaller sections (Daganzo (1994) uses cells) gives the opportunity to further set the speed limits. Besides, due to the large links the exact location of on- and off-ramps are not considered. Upstream of many on-ramps an off-ramp is located, and congestion, caused by an on-ramp, could block these off-ramps. As the exact location of off-ramps and on-ramps are not included in this model, the effect of variable speed limits and ramp-metering on off-ramp blocking should be investigated (Hegyi et al., 2005) for further improvement.

As an entire link is set as bottleneck (in real world it is just a small road section), expected is that a higher dense installed detectors would improve the performance. Here, the dynamics of real world would be better captured by the model. Extra detectors would also improve the detection of congestion. It would be more feasible to detect local congestion, but it is questionable whether it is desirable switching the control strategy on for temporary local congestion. In the discussed model it is possible not all detectors are used, because the speed limits setting is based on on-ramps. Improvement of the model should be able to use all available measurements.

5.3 Conclusion

Based on traffic stability, a theoretical local ramp metering strategy is proposed. Further research is needed to determine the influence of variables on the stability. Besides, a macroscopic strategy is proposed the optimal ramp meter rate after setting the variable speed limits. Here, a model predictive control approach is used to, based on real world measurements, minimize the total time spent and to maximize the total traveled distance. In the following chapter, the integrated ramp metering and variable speed limit strategy is tested in a simulation environment.

6 Case Study

The effectiveness of the traffic control strategy can be evaluated by verifying whether the proposed strategy is able to improve the performance. In this chapter a case study is presented and the results are analyzed.

6.1 Simulation

The simulation software Aimsun is used to evaluate the control strategies. The development of the model is based on Dowling et al. (2004) (see Appendix C).

6.1.1 Introduction

It is widely accepted that simulation is a technique to provide an experimental test to compare different controls. Trying out the controls in real world is not an option because costs are too high and the expected performance is unknown. Simulation software could be used to perform 'What-If' analysis and is a cost-effective tool.

A key point is to ensure the model is valid. It is essential that the model mirrors real world behavior. Therefore, calibration is needed. Still, after calibration, simulation models give only an approximation of real world behavior, a small error in the parameters setting for the simulation software can lead to large errors in the overall results (Force, 2004). Besides, real world driving behavior also includes accidents, the simulation environment does, in this case, not simulate accidents. Another disadvantage of the simulation software is the system size. In the simulation software not the entire system (I-880 and underlying network) can be captured and therefore only a part of the system is modeled.

Traffic simulation models can be classified as microscopic, mesoscopic or macroscopic. Macroscopic models are based on the deterministic relationship of the flow, speed, and density of traffic. Mesocopic simulation models combine the properties of microscopic and macroscopic simulation models. Traffic flow in this type of simulation is an individual vehicle but the movement is using the approach of the macroscopic mode (Ronaldo et al., 2012). Microscopic models simulate the movement of individual vehicles based on car-following and lane-changing theories. As the microscopic models capture the behavior of individual drivers exposed to control strategies, the commercial microsimulation software Aimsun (based on Gipps (1986)) is used for the purpose of this report.

6.1.2 Selection

It is desired to model an existing part of the highway. The characteristics of the highway can be modeled, the measured data can be used for calibration and the results can help decision makers. The traffic network considered is a part of the Interstate 880 (I-880) in California, USA. The I-880 is a 46 mile long highway from San-Jose, CA to Oakland, CA and is a major congested highway. Considering the time-space diagram of the I-880 NB (Figure 6.1), several congested areas can be identified. Empirical findings have shown that the I-880 NB has, in the evening hours, two recurrent bottlenecks: at post mile 15.12 (at Automall Pkway, Fremont) and post mile 30.0 (Hesperian Blvd, San Leandro).

The recurrent bottleneck at post mile 15.12 is considered, at April 4 2013. This day is considered, taken into account several restrictions. Based on an internal report it is possible that the congestion,

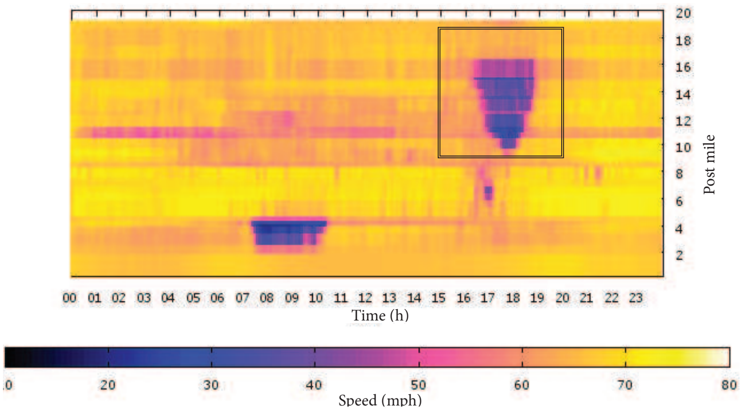


Figure 6.1 *The time-space speed contour plot for the I-880 NB on April 4, 2013 (Caltrans, 2013). Bordered is the considered section in the simulation model. The dark areas indicate low speed and can be determined as congestion areas.*

started at post mile 30 and post mile 15, get coupled. On this day the congestion does not get coupled with other large high densities areas. Besides, a lot of traffic accidents occur during congestion hours. It was shown that during congestion, caused by the on-ramp near post mile 15 at April 4 2013, no accidents occurred in the evening hours. At the considered section an HOV-lane is available from 3pm until 7pm, empirical findings have shown that the HOV-lane is not always used properly during congestion hours. Implemented in the simulation software is the section from post mile 9.8 (Dixon Landing Road, Milpitas) to post mile 18.66 (Central Ave, Fremont) from 3pm to 8pm.

6.1.3 Implementation

The implementation of the model consists out of implementing the infrastructure layout and setting up an origin-destination-matrix. Based on the real world road structure, the highway is implemented with the associated length, number of lanes, HOV-lane, on-ramps, off-ramps, detectors, speed limit etc. Here, the different highway sections are also connected. Based on the available detector measurements is tried to capture real world behavior (calibration).

During the calibration process the virtual detectors are used to match the real world detectors. The virtual detectors are located at the same location as the real world loop detectors (Appendix E), as the simulated detectors show the same measurements as the real loop detectors the model is said to be ’calibrated’. Calibration is an iterative process where the flows (inflow mainline, inflow on-ramps, outflow off-ramps and outflow mainline) and the parameter settings (minimum gap, desired speed etc.) of the model are set. First, the most upstream and downstream detector are used for calibration. This implies that the mainline inflow and mainline outflow are captured in the model. Detectors located on the ramps were not working or were not available, the up- and downstream located detectors are used for setting up the ramp flow.

The data used, for the I-880 NB calibration, is 5-minutes aggregated data of the PeMS database (database of traffic measurements in California, USA) (Caltrans, 2013). As the aggregated data is showing too high speeds, tried is to capture the general pattern of the speed. PeMS receives measurements from, in this case, mostly single loop detectors. The single loop detectors provide 30-seconds raw measurements including flow and occupancy. The 30-seconds data is aggregated to 5-minute data. As the raw measurements may contain holes (missing data) or wrong data, these are replaced by the Daily Statistics Algorithm (Chen et al., 2003) based on the correlation between detectors. Not working detectors are not used for calibration as there is no comparison possible between the simulation model and the real world measurements.

During the calibration process several assumptions are made:

- Only (HOV-)car and truck traffic is considered in the model. Besides, there is no variance in length, size etc. The feature of mixed traffic is not well reflected in the simulation. Assumed is a 5 % percentage of trucks in all the demand levels.
- The highway sections have no slope. In real world it is possible that a slope causes increasing or reducing speed, which is not captured in the model.
- Vehicles are not allowed to change their destination, departure time or cancel the trip.
- The demands are fixed; no extra vehicles can enter the highway. In real world, if congestion is earlier solved, more vehicles could consider using the highway.
- To enable all vehicles to enter the network, extra highway sections are placed upstream of the on-ramps to make sure no vehicles are waiting outside the network.

Based on the comparison between the model and the detector measurements (Appendix D) it can be stated that the calibration process is not yet finished as the congestion is moving too far upstream. Besides, the simulation model does not capture local variance in flow and speed. The lane distribution is not very well captured, in all cases the flow in lane 2 is too low.

In some cases the flow is too low, but the speed is also too low. This indicates that the parameter settings need some adjustment. Essential for calibration is using the model parameters for different days. The model is calibrated based on one day data and therefore is the model calibration not finished. The found settings can be used for a comparison of performance but will just give a first indication of the efficiency improvement.

6.2 Traffic Scenario

During the calibration process four different (hour-during) demand levels are found. These demands are used for simulation runs, starting at 3pm and ending time 8pm. Clear is, as expected, that the congestion demand is significant higher than free flow demand. Note that the stated demands are cumulated over all inflow locations (inflow of mainline and on-ramps).

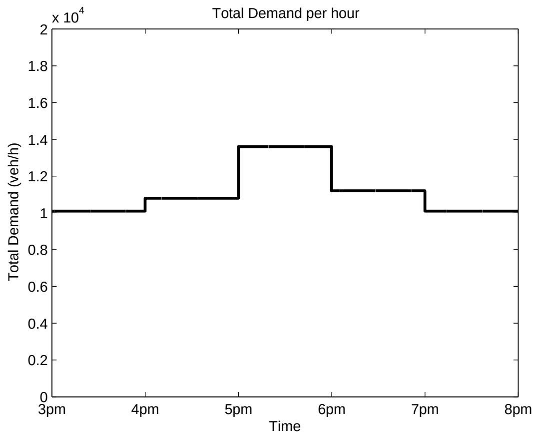


Figure 6.2 *Used demand in the simulation software. Demand includes all inflow locations (on-ramps and mainline) and all vehicle types.*

6.3 Set-up

Different strategies are compared in the simulation software. As was shown before, the local ramp metering strategy ALINEA has proven that it is able to reduce travel time, is outperforming feed forward ramp metering strategies and is therefore implemented. As the most highways have implemented ramp metering, a coordinated ramp metering controlling more on-ramps is implemented. The best performance in literature was achieved using an integrated control strategy, the integrated strategy, proposed in the previous chapter, is implemented. To consider the influence of variable speed limits, a strategy only using variable speed limits is implemented. All strategies are compared to the uncontrolled case. Recall:

- Uncontrolled traffic
- Local ramp metering strategy ALINEA (as proposed in section 5.1 and Appendix B)
- Variable Speed Limit strategy (proposed in section 5.2.1)
- Coordinated Ramp Metering strategy (proposed in section 5.2.2)
- Integrated Variable Speed Limit and Coordinated Ramp Metering strategy (proposed in section 5.2)

The goal of the strategies is to improve efficiency. The objective is minimizing the Total Time Spent and improving the Total Traveled Distance during congestion period.

Aimsun gives the user the possibility to set the compliance to the speed limit. In this case, compliance indicates whether drivers are driving with the speed conforming to the set speed limit. In literature (Lu et al., 2011) a compliance of 100 % indicates a strictly enforced speed limit and a compliance of 30 % indicates an advisory speed limit. Used in the simulation runs is a compliance of 100 %.

For the variable speed limit strategy and the integrated variable speed limit and coordinated ramp metering strategy the highway is divided into links. The length of the links vary between 0.25 miles and 1.25 miles and the total length of the considered area is 9.5 miles. A total of 13 links are considered, with a total of 11 on-ramps and 7 of-ramps. For an overview of the simulation network, referred is to Appendix E. Some control strategies require detectors at all on and off-ramps. In real world, not all on- and off-ramps do have detectors. Here, several virtual detectors are placed in the simulation software at ramps.

6.3.1 Number of simulation runs

Multiple simulation runs are required to get an average result in order to deal with stochastic processes in Aimsun, and to determine the impact of the strategies. The more runs are performed, the higher accuracy in the resulting values will be gained. Wiegand & Yang (2011) did research to determine the number of simulation runs. Clear is that one simulation run is not sufficient, Wiegand & Yang (2011) conclude that the most notable differences were between the 5- and 10-run tests in the software CORSIM, and results generally became stabilized after 10 to 15 runs. Concluded can be that at least 10-simulation runs should be made. Because a simulation run is very time consuming, only 5 runs per strategy could be made. Assumed is that this number of simulation runs is sufficient to get an indication of the impact of the strategies. All scenarios are based on same random seed numbers.

6.3.2 Parameter Settings

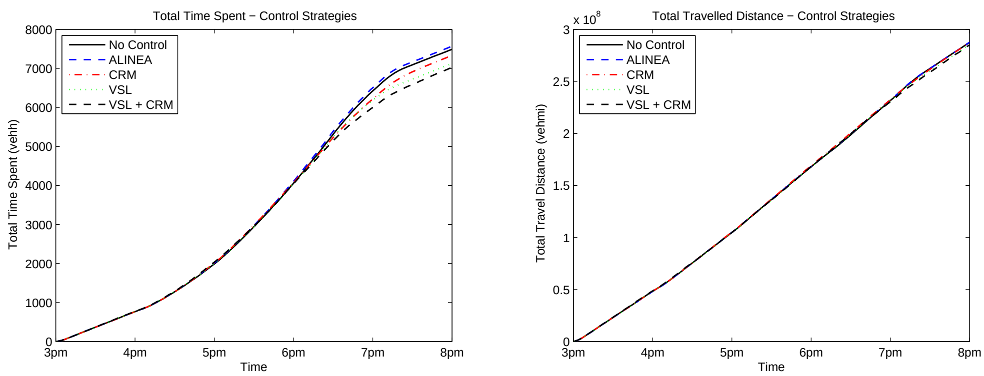
All strategies are activated when, based on historical data, the measured occupancy of a link enters the metastable regime. The control is switched off when the measured occupancy is in the stable regime. For the variable speed limit strategies is, due to practical reasons, the speed limit set every 60 seconds. Besides, the ramp metering is switched off, to prevent an upstream spill back, when the queue at an on-ramp is approaching its capacity.

The parameter settings for the control (coordinated ramp metering, variable speed limit and the integrated strategy) are as follows:

- $o_{start,m}$ threshold which switches the control on/off, is the border between the stable and metastable regime.
- $o_{c,m}$ to maximize flow in a link, assumed to be 10 % higher than the threshold $o_{start,m}$.
- η: prioritizing the mainline or the on-ramp flow: 0.5.
- σ: balance the optimization of TTS and TTD: 0.5
- u_{max}: maximum allowed speed limit is 65 mph.
- u_{min}: the minimum allowed speed limit is 20 mph.
- Δu: maximum speed limit difference between two links is 5 mph.
- ζ: a control parameter to set the 'aggressiveness' of the control: 3.
- N_p: Due to the complexity of the control, the predicted horizon is 60 seconds ($N_p = 2$).
- N_c: The control horizon is 30 seconds ($N_c = 1$).

6.4 Results

For the comparison of improvement in traffic conditions, the total time spent (TTS) and total traveled distance (TTD) are used. The cumulative TTD over the entire simulation time, as all vehicles have entered the network, will not improve. In real world, possibly, more vehicles are able to enter the network if the travel time is improved. Individuals, choosing an alternative route because of the congestion on the network, are now, possibly, choosing this highway. These effects are not simulated.



(a) Total Time Spent (b) Total Traveled Distance

Figure 6.3 *Results of the microsimulation performance of the different control strategies. The integrated variable speed limit (VSL) and coordinated ramp metering (CRM) strategy is best performing. The local ramp metering strategy ALINEA does shows an increase in total time spent. The total traveled distance is not significantly changed due to different strategies.*

The simulation environment shows that the control strategies are able to improve the efficiency of the network. The performance indicator TTD is not significantly improved (performance within 0.5 % in comparison with the uncontrolled case) using different control strategies. ALINEA, a local ramp metering strategy, does not improve the efficiency of the network, as the simulated environment shows an increase in TTS of 1 %. This is in contrast to the results of Papageorgiou et al. (1997). The coordinated ramp metering strategy is slightly able to improve TTS, a 2 % gain of travel time is obtained using this strategy. It is shown, similar to results in Lu et al. (2011) and Su et al. (2011), that RM control alone is not able to improve the efficiency significantly. It can be shown that the usage of variable speed limits in this network is able to improve the total time spent with 5 %. The usage of an integrated control strategy using variable speed limits and coordinated ramp metering shows even an improvement of 6 %. As empirically is determined, no (new) large congested areas were created by the different control strategies.

6.5 Discussion

It can be concluded that the results are in line with other variable speed limits and coordinated ramp metering strategies, but the proposed strategies perform less well than the strategy on which it was based (Su et al., 2011; Lu et al., 2011; Lee et al., 2013). Besides, the ALINEA strategy is not improving performance, which in contrast to available literature. The limited performance of the strategies can be validated by the high demand of the on-ramps and the lengths of the on-ramps. As the on-ramp storage has reached 90 % of its capacity, the ramp metering is switched off to make sure the queue is not spilling back to adjacent lanes upstream and outside of the network.

The results are in line (qualitatively) with Lu et al. (2011) and Su et al. (2011) where the coordinated ramp metering shows an improvement of approximately 20 % in time spent and the variable speed limit strategy shows an improvement of approximately 50 %. Su et al. (2011). This is accordance with the results in Carlson et al. (2010) where coordinated ramp metering improves TTS with approximately 20 % and the controls with variable speed limits and improvement of 45 %. In this case, the TTS is improved with 5 % using a variable speed limit strategy. The total travel distance results shows no improvement. This in contrast to the results in Lu et al. (2011) and Su et al. (2011). Concluded can be that a control strategy should (at least) include variable speed limits as it is outperforming other single control strategies.

Coordinated ramp metering strategy is, in this case, outperforming the local ramp metering strategy. This is contrast to Papageorgiou et al. (1997), where is concluded that a coordinated ramp metering strategy is better performing in case of non-recurrent congestion. Here, recurrent congestion is considered but an other coordinated ramp metering strategy is applied. The results can be validated as the local ramp metering strategy is only controlling one on-ramp and is often released. In contrast, the coordinated strategy limits the inflow of the bottleneck via several on-ramps. The limitations of any ramp metering strategy are clear, as the length of the storage capacity of an on-ramp is influencing the performance. Results of this research are in contrast with the statement of Papageorgiou et al. (1997), where is concluded that ALINEA is the strategy to which other strategies should be compared. The coordinated ramp metering strategy shows little improvement in TTS (2 % gain) and no improvement in TTD. These results are qualitatively in line with results of other coordinated ramp metering strategies (Papageorgiou et al., 1997; Lu et al., 2011; Bhouri et al., 2011).

The performance is less well than several other control strategies (Hegyi et al., 2005; Carlson et al., 2010), measured using the macroscopic model METANET (Messner & Papageorgiou, 1990). This can be validated due to several reasons. First, Hegyi et al. (2005) has shown that a larger prediction horizon in the METANET (Messner & Papageorgiou, 1990) model can improve the performance of the model. Here, only a small ($N_c = 2$) prediction horizon is used. Second, a microscopic model shows individual behavior and is therefore more close real world behavior.

For the model predictive control scheme the simplex method is used. As empirical findings have shown that this is a time consuming method, only 10 iterations were allowed. This can lead to a suboptimal solution, further increasing the maximum number of iterations (maximum iterations should be infinite) would, likely, improve the performance. The control strategy could also be improved using, not simulated in this case, all time metering, all time variable speed limits and all time integrated control. Whether it is desirable to use control strategies for a fixed time is questionable, but it was shown that this can improve the performance of a strategy Lu et al. (2011). Besides, an all time strategy could only be used for a fixed bottleneck location.

The local ramp metering strategy's performance is likely to be highly influenced by the density of the loop detectors. As the strategy is based on the downstream loop detectors, in some cases the downstream loop detector is far away and therefore influenced by other road characteristics (such as an off-ramp). Besides, the minimum variable speed limit is restricted by the number of control links upstream of the bottleneck. The simulation has shown that the minimum speed limit is influencing the performance. As more upstream highway links are available in real world, it is expected that the performance will be better.

In the simulation environment only a part of the available detectors were used. More detectors would be able to improve the performance of the model, as here one set of detectors is used for traffic characteristics over an entire link. This could be done by aggregating the detector measurements or by further dividing the network into more links (or cells). The proposed algorithm overcomes the drawback of the SPECIALIST algorithm, as serious congestion could be detected by low dense installed detectors. However, high dense installed detectors would give the opportunity to this algorithm to resolve moving jams (relatively short jams with an upstream moving head and tail). Virtual detectors are used, in this case, for measuring the off- and on-ramp demand. For optimal use, it is required to have on- and off-ramp demand data available. Therefore, new (virtual) detectors need to be placed. For control, at least one set of detectors should be working within a link. In case of a not working link, data imputation or historical data could be used, this can lead to less performance.

Simulation runs were performed with a speed compliance of 100 %. Shown in Su et al. (2011) is that the 30 % driver compliance with the advisory speed limits shows similar performance to the results with 100 % compliance. In practice, the compliance differs, and further research is needed to set an appropriate compliance rate. Besides, it should be tested whether the advisory speed limit would show the same performance in this particular case. The used speed limits were set without increments. In real world, using variable message signs, it could be more desirable to use increments of e.g. 5 mph. In a study of Hegyi (2004) increments of 10 km/h shows similar performance (in a macroscopic simulation model) as the control without increments, but this should be tested in a microscopic simulation environment.

The control strategy is based on measurements averaged over all lanes. The HOV-lane shows in the simulation, similar to the measured loop detector data, a lower flow and a higher speed than the other lanes. As not all vehicles are allowed to use the designated lane, it could be desirable, for as well switching the strategy on and off as for setting the ramp meter rate, to exclude the HOV-lane measurements for control. This has as drawback that an accident an HOV-lane could not be captured. Here, simple algorithms, measuring the speed difference between HOV and other lanes, could be used to overcome this drawback.

Real World Application

The results give implications for real world introduction. Discussed are drawbacks and improvements regarding real world application.

Necessary for the control strategy is dividing the highway into links. Dividing into links is dependent on the location of the loop detectors and the on-ramps. For real world application every on-ramp should have a set of loop detectors available for measurements. Besides, every on- and off-ramp need detectors to measure the demand and the outflow.

Based on the available literature and the results of this research one can say that a control strategy using coordinated ramp metering and variable speed limits should be applied in order to minimize the total time spent. Here, every on-ramp needs a ramp metering to control the inflow and every link should contain variable message signs to show the speed limit. The considered literature (Su et al., 2011; Lu et al., 2011; Carlson et al., 2010) shows the best performance for (different) variable speed limit strategies and is outperforming (coordinated) ramp metering strategies. One can conclude that in order to limit the total time spent variable message signs are required. However, in practice, these variable message signs are not widely used for controlling flow. Mostly a (fixed) ramp metering strategy is used. In this research, the coordinated ramp metering strategy shows little improvement due to earlier discussed reasons, further research is needed to use coordinated ramp metering in order to improve the performance significantly.

As a simple algorithm is proposed for the strategy, it is expected that the strategy could also be applied on other infrastructure layouts and other demands. In Su et al. (2011); Lu et al. (2011); Lee et al. (2013) a slightly different approach is used for limiting congestion caused by weaving or caused by a mainline lane drop. These simulation studies show improvement of the efficiency. As the proposed strategy is dynamic it can be applied to different highway sections. It is expected that the strategy is able to improve efficiency, but its performance is highly influenced by the demand and the length of the on-ramps.

In the linear prediction model, used for model predictive control, assumed is that the queue at the on-ramp is known. For real world application, more advanced measurement techniques are required to measure the exact queue. As stated before, the length of the queue is influencing the performance. Therefore, several detectors can be placed measuring the speed to give an indication of the queue or the length of the queue can be assumed (e.g. based on historical data). Further research is required for a method measuring the length of the queue at an on-ramp.

The length of the links are defined by the distance between two adjacent on-ramps. As two on-ramps are very close to each other, the length of the link is very short. In the strategy the maximum speed difference between two links is set and is based on the idea that drivers do not have to face large speed differences. In the case of short link lengths, drivers will face in a short time different speed limits. In this particular case (Appendix E), between link 9, 10 and 11 drivers can face speed limits difference of 15 mph within a mile. This is undesired in real world application.

Assumed in a simulation environment is, essential for control strategies, that all measurements are available and good. However, in practice, it is possible that detectors are not working or are providing 'wrong' data. To handle wrong or missing data, Lu et al. (2010a) provided a set of tools to correct or input data. It is questionable whether imputed data can be used for dynamic traffic control as it introduces noise or systematic deviations. Further research is needed whether it is possible to use corrected or inputted data for dynamic traffic control in practice.

Future work should include the discussed points and it is highly recommended to further calibrate the model and after calibration, using the same parameters (but other demands) for other days. If the control strategy improves the performance of the model significantly the control strategy could be considered for application in real world. However, it is unknown, as no variable speed limit strategies have been applied in real world, whether the simulation environment is able to capture drivers' behavior in variable speed limit situations.

6.6 Conclusion

The strategies, proposed in the previous chapter of this report, are applied in the microsimulation environment. Here, a part of I-880 NB is implemented in the software, and is calibrated for one day. The different control strategies show a minor improvement of the performance.Total Traveled Distance does not improve significantly and the Total Time Spent shows the best performance with a combined variable speed limit and coordinated ramp metering strategy (improvement of 6 %). All the strategies are not able to prevent traffic breakdown, but some strategies are able to resolve the congested area earlier. The short on-ramp lengths are presumambly the cause for the minor improvement of the ramp metering strategies, simulation has shown that metering were often released.

Clear is that the control strategy needs improvement and several improvements are proposed. Further research should include testing the strategy for different days in a simulation environment.

7 Conclusions

The goal of the research was developing a variable speed limit and coordinated ramp metering strategy using traffic flow stability. This chapter recalls the sub research questions and gives an answer to the central research question.

7.1 Conclusions

What is traffic stability and how can it be measured using existing measuring techniques?

A stable traffic system is one that when perturbed from an equilibrium state tends to return to that equilibrium state (Pueboobpaphan & van Arem, 2010). The classification of (in)stability depends on the number of influenced vehicles and the amplitude of the perturbation. Based on the number of vehicles influenced, one can distinguish local (in)stability, string (in)stability and flow (in)stability. Where local and string (in)stability focus on two or a few vehicles, traffic flow stability is concerned with the disruptions in macroscopic characteristics of traffic (Elbers, 2005). What the influence is of a disruption is depending on the amplitude of this perturbation. If the amplitude increases in the course of time, traffic is unstable. If small perturbations decay, but severe perturbations develop to serious waves, one speaks of metastability. If every perturbation is causing a serious wave it is called instability.

These perturbations can be caused by the individual behavior of a driver. A lane change (and the reaction of the lag vehicles) can, based on the stability of the traffic flow, cause a breakdown. Therefore it is desirable to measure the stability of the upstream traffic flow. A stochastic indicator is able to determine whether traffic has entered the metastable regime and the stability of traffic before approaching a merging lane can be determined based on the reliability indicator. Both indicators only use macroscopic traffic characteristics (speed, flow, density and/or occupancy), which can be measured by loop detectors or determined based on these measurements.

How can traffic stability contribute limiting the congestion on highways?

The stability of a traffic indicates whether congestion will arise or not. Bottlenecks have a certain capacity and in congestion this capacity drops. In simple models, this capacity drop will occur if the total demand of a certain highway section is larger than the capacity. Although, based on stability, traffic can also breakdown if the total demand is not exceeding the (fixed-value) capacity. If traffic flow has entered the metastable regime, traffic has a chance of breakdown. This gives a serious implication for traffic control; traffic needs to be controlled in the metastable regime. For optimal control traffic needs to be controlled to make sure the total flow stays in the metastable regime and a perturbation, caused by lane changing at an on-ramp, needs to be controlled to make sure the breakdown does not occur.

How can variable speed limits and coordinated ramp metering improve traffic flow?

To make sure congestion will not arise, traffic can be controlled. Traffic can be controlled using ramp metering, speed limits, route guidance, dedicated lanes etc. This report has only focused on two of these strategies; ramp metering and speed limits.

To measure the improvement of the control strategies, the efficiency of the strategies are indicated. Increaseing the efficiency is minimizing the total time spent (TTS) and maximizing the total traveled distance (TTD). The total time spent is the travel time of all vehicles in the network and the total traveled distance is indicating how much miles all the vehicles have made within a certain time range. The ratio of the total distance covered by the total time spent gives the mean speed; as the total time spent can be minimized and the total traveled distance is maximized; the mean speed of all vehicles is maximized.

Real world application of ramp metering strategies have proofed that controlling the on-ramp flow can improve the performance. It was shown by Papageorgiou et al. (1997) that the ALINEA strategy can improve TTS up to 18 % and TTD up to 3 %. It is not inferior to other ramp metering strategies and, therefore, became the international standard to which other strategies are compared.

Variable speed limits are also able to improve the performance. Basically, variable speed limits can be used to prevent and resolve breakdown or to homogenize traffic. A prediction control method shows, in a simulation environment, large performance improvements of the approach to prevent breakdown. In combination with coordinated ramp metering these methods show even a better performance (up to 55 % in TTS and up to 25 % in TTD (Lu et al., 2011)). Drawback of all these methods is that they are not tested in real world. A variable speed limit algorithm to suppress moving localized clusters is applied in real world and shows improvement of travel time. This algorithm can not be applied in low density equipped detector areas. Researchers do not agree whether the control strategy to homogenize traffic is improving the performance of the network. Based on the defintion of stability it can be validated that homogenization can not be used to improve the traffic performance.

Literature is used to propose a local ramp metering strategy and an integrated strategy, using variable speed limits (to prevent traffic breakdown) and coordinated ramp metering, in order to control traffic. The local ramp metering strategy bases the ramp meter rate on the stability of the upstream traffic flow. Further research is needed to set up and test the strategy. An integrated strategy, based on Lu et al. (2011) and Su et al. (2011), is proposed using the metastable regime. As traffic flow on a highway segment is metastable, control is switched on using variable speed limits and coordinated ramp metering in order to limit the inflow of the highway segment. The variable speed limit on the highway is based on the difference between the measured and the desired occupancy. Used is a Model Predictive Control to optimize the ramp meter rate.

What is the microsimulation performance of the control strategy on the I-880 highway in California, USA?

The strategy, based on Lu et al. (2011) and Su et al. (2011), is tested for a limited time (3pm to 8pm) on the I-880 NB highway from post mile 9 to post mile 18.66 in California, USA. The control strategy shows in the microsimulation software an improvement of 6 % in total time spent, and the total travelled distance is not improved. Other control strategies show less improvement and the ALINEA strategy shows even no improvement. This is contrast to the results of Papageorgiou et al. (1997) where is stated that ALINEA is able to improve the performance. In this case, the coordinated ramp metering strategy is outperforming the ALINEA strategy.

It can be concluded that the results are in line, qualitatively, with other variable speed limit and coordinated ramp metering strategies, but the proposed strategies perform not as good as the strategy on which it was based. Lu et al. (2011), Carlson et al. (2010) and Su et al. (2011) show an improvement up to 55 % in total time spent. The difference in performance can be validated by the short on-ramps. The ramp meterings are often released, therefore is the control strategy, only using variable speed limits, is almost as good performing (improvement in total time spent of 5 %) as the integrated strategy.

Overall, the integrated proposed strategy is not able to prevent or resolve the major congested area. The variable speed limit strategies are able to resolve the congested area earlier, but no improvement is shown in the simulation environment before 6:30 pm.

How can the control strategy be best introduced in the real world given the simulation results?

For real world application of the control strategy, an integrated strategy should be used. Variable message sings should be used to for variable speed limits and ramp meters should be used to co-ordinated ramp flow. The integrated strategy is, similar to results in Lu et al. (2011), Carlson et al. (2010) and Su et al. (2011), outperforming other strategies. Here, the highway should be, based on the infrastructure layout and location of detectors, divided into links. Besides, ramp meters and extra detectors (measuring the ramp queue and on- and off-ramp demands) are required.

Introducing the strategy in real world requires further research. Expected is that the results could be improved further dividing the highway into smaller links. Besides, it should be modeled what the influence of the drivers' compliance is on the results. It could also be considered to use all time control, but it questionable whether this is desired. Using variable speed limits in real world, it is desired to use speed limit increments of e.g. 5 mph. This is not captured in the current model, but expected is that this will not alter the performance (Hegyi, 2004).

Future work is required and should include further calibration, improving the model and testing the strategy in a simulation model on several days. If the control strategy improves the performance of the model significantly, the control strategy could be considered for application in real world. It is unknown, as no variable speed limit strategies (used to prevent breakdown) have been applied in real world, whether the simulation environment is able to capture drivers' behavior in variable speed limit situations. As a simulation model is not able to capture all real world behavior, different results should be expected.

These conclusions give the possibility to answer the central research question:

How can congestion be limited using a variable speed limit and coordinated ramp metering strategy based on traffic stability?

Congestion on a highway is a result of a combination of a high traffic volume, a spatial inhomogenity and a temporary perturbation. If traffic flow is high, a local perturbation (caused by a mandatory lane change near a spatial inhomogenity) can cause traffic breakdown. To limit the congestion on a highway, variable speed limits and coordinated ramp metering could be used. As literature has shown that variable speed limits and ramp metering are able to prevent traffic breakdown, these strategies are integrated in one control. If traffic on a segment of the highway has entered the metastable regime (serious perturbations will cause a breakdown) traffic control is switched on to prevent a breakdown. The proposed strategy determines the variable speed limits based on the measured and desired occupancy, a model predictive control scheme is used to compute the optimal ramp meter rate.

A microsimulation study has shown that the control strategy improves the efficiency of the highway. The total time spent was improved with 6 %. The performance is highly influenced by the length of the on-ramps, ramp meters were often released (as the storage of the ramp has reached capacity) and therefore the performance reduces. Concluded is that the control strategy is not able to prevent congestion but is able to resolve congestion earlier. For real world application, strategy improvement is required and the strategy should be tested in a simulation model on several days.

7.2 Further Research

Further research is needed to determine the real world performance of an integrated variable speed limit and coordinated ramp metering strategy. No strategies have been applied in real world to prevent traffic breakdown and therefore it is difficult to determine the expected performance.

Another topic for research is testing the proposed strategy on several days for the same highway. Therefore, several improvements are proposed in section 5.2.3 and section 6.5. The performance should be determined again for these different days.

In this report a first proposal is made to determine the ramp meter rate based on the traffic stability upstream of the on-ramp. As only the variables are mentioned on which the meter rate should be determined, future work is required to determine the influence of the different variables and to test the performance.

7.3 Research contributions

Investigated are stability indicators which can be applied using existing measuring techniques. It is shown that the reliability indicator is able to determine the stability of traffic and can be used to set the ramp meter rate.

Based on the definition of traffic stability, it can be concluded that the variable speed limit strategy, using a homogenization approach, is not able to increase capacity. The reliability indicator (Ferrari, 1988) supports this statement. This is in contrast with the statement of Van den Hoogen & Smulders (1994) and Zackor (1991) that homogenization causes a more stable traffic flow and, thus, a higher capacity. This can be validated because Van den Hoogen & Smulders (1994) does not define stability and Zackor (1991) shows only a very small (negligible) increase of capacity.

Practical contribution is made for the I-880 NB Highway in California, USA. As the microsimulation software has shown that the variable speed limits and coordinated ramp metering can improve the performance of the network, eventually application of variable speed limits and coordinated ramp metering on the network should be considered in order to improve the performance of the highway. Here, more work is required before considering application in real world.

7.4 Discussion

This research has proposed a strategy to limit congestion on a highway based on traffic stability. A literature and simulation study were conducted. As a consequence of this methodology, the study encountered a set of limitations.

Lots of literature is available in the field of variable speed limit and (coordinated) ramp metering strategies. As only limited time was available, not all literature could be considered. A selection is made based on recommendations of the supervisors. Due to making a selection, not a complete overview could be given regarding this topic.

Goal of the simulation study was measuring the performance of the strategies. If the calibration process is finished the results can give an approximation for real world application. Drawback is that calibration is a very time consuming process. Here, one day is modeled and the calibration process is not yet finshed. One should notice that data collection is an important part of model development. Unfortunately, the data of the I-880 highway is missing a lot of measurements (due to e.g. loop detector errors) and the speed is often overestimated by the g-factor approach.

7.5 Conclusion

Congestion on highways is a serious issue. In this study, tried is to resolve or prevent congestion using traffic control. Based on traffic stability, a variable speed limit and ramp metering strategy is proposed. A simulation study has shown that an integrated control strategy is able to limit congestion on the I-880 NB in California, USA, but congestion could not be completely solved.

Bibliography

Ahmed, K. I., Ben-Akiva, M., Koutsopoulos, H. N., & Mishalani, R. G. (1996). Models of freeway lane changing and gap acceptance behavior. In *Proceedings of the 13th international symposium on the theory of traffic flow and transportation* (pp. 501–515). Pergamon New York, NY.

Ahn, S., & Cassidy, M. J. (2007). Freeway traffic oscillations and vehicle lane-change maneuvers. In *Transportation and Traffic Theory 2007. Papers Selected for Presentation at ISTTT17*.

Bertini, R. L. (2006). You are the traffic jam: Examination of congestion measures. In *Transportation Research Board 85th Annual Meeting* 06-1419.

Bhouri, N., Haj-Salem, H., & Kauppila, J. (2011). Isolated versus coordinated ramp metering: Field evaluation results of travel time reliability and traffic impact. *Transportation Research Part C: Emerging Technologies*, .

Bloomberg, L., Swenson, M., & Haldors, B. (2003). Comparison of simulation models and the hcm. *TRB, Transportation Research Board, Washington DC*, .

Boxill, S. A., & Yu, L. (2000). An evaluation of traffic simulation models for supporting its, .

Brilon, W. (1988). Recent developments in calculation methods for unsignalized intersections in west germany. In *Intersections without Traffic Signals* (pp. 111–153). Springer.

Brilon, W., Geistefeldt, J., & Regler, M. (2005). Reliability of freeway traffic flow: a stochastic concept of capacity. In *Proceedings of the 16th International symposium on transportation and traffic theory*. volume 125143.

Burghout, W. (2004). A note on the number of replication runs in stochastic traffic simulation models. *Centre for Traffic Research, Royal Institute of Technology, Stockholm, Sweden*, .

Caltrans (2013). Caltrans pems. retrieved april, may & june, 2013, from http://pems.dot.ca.gov.

Camacho, E. F., Bordons, C., Camacho, E. F., & Bordons, C. (2004). *Model predictive control* volume 2. Springer London.

Carlson, R. C., Papamichail, I., Papageorgiou, M., & Messmer, A. (2010). Optimal mainstream traffic flow control of large-scale motorway networks. *Transportation Research Part C: Emerging Technologies*, *18*, 193–212.

Chen, C., Kwon, J., Rice, J., Skabardonis, A., & Varaiya, P. (2003). Detecting errors and imputing missing data for single-loop surveillance systems. *Transportation Research Record: Journal of the Transportation Research Board*, *1855*, 160–167.

Daganzo, C. F. (1994). The cell transmission model: A dynamic representation of highway traffic consistent with the hydrodynamic theory. *Transportation Research Part B: Methodological*, *28*, 269–287.

Darbha, S., & Rajagopal, K. R. (1999). Intelligent cruise control systems and traffic flow stability. *Transportation Research Part C: Emerging Technologies*, *7*, 329–352.

Dowling, R., Skabardonis, A., & Alexiadis, V. (2004). *Traffic analysis toolbox volume III: Guidelines for applying traffic microsimulation modeling software*. Technical Report.

Elbers, J. A. C. M. (2005). *Development of an indicator for traffic flow stability with application to ramp metering*. University of Twente.

Ferrari, P. (1988). The reliability of the motorway transport system. *Transportation Research Part B: Methodological*, *22*, 291–310.

Force, H. C. T. (2004). *A report on the use of traffic simulation models in the san diego region*. Technical Report Technical report, Institute of Transportation Engineers, Highway Capacity Task Force.

Gartner, N. H., Messer, C. J., & Rathi, A. K. (2001). *Traffic flow theory: A state-of-the-art report*. Committe on Traffic Flow Theory and Characteristics (AHB45).

Ghods, A. H., Fu, L., & Rahimi-Kian, A. (2010). An efficient optimization approach to real-time coordinated and integrated freeway traffic control. *Intelligent Transportation Systems, IEEE Transactions on*, *11*, 873–884.

Gipps, P. G. (1986). A model for the structure of lane-changing decisions. *Transportation Research Part B: Methodological*, *20*, 403–414.

Google (2013). Google maps. retrieved april 30, 2013, from http://maps.google.nl.

Greenshields, B. D., Bibbins, J., Channing, W., & Miller, H. (1935). A study of traffic capacity. In *Highway research board proceedings*.

Hegyi, A. (2004). *Model predictive control for integrating traffic control measures*. Netherlands TRAIL Research School.

Hegyi, A., De Schutter, B., & Hellendoorn, H. (2005). Model predictive control for optimal coordination of ramp metering and variable speed limits. *Transportation Research Part C: Emerging Technologies*, *13*, 185–209.

Hegyi, A., & Hoogendoorn, S. (2010). Dynamic speed limit control to resolve shock waves on freeways-field test results of the specialist algorithm. In *Intelligent Transportation Systems (ITSC), 2010 13th International IEEE Conference on* (pp. 519–524). IEEE.

Hegyi, A., Hoogendoorn, S., Schreuder, M., Stoelhorst, H., & Viti, F. (2008). Specialist: A dynamic speed limit control algorithm based on shock wave theory. In *Intelligent Transportation Systems, 2008. ITSC 2008. 11th International IEEE Conference on* (pp. 827–832). IEEE.

Van den Hoogen, E., & Smulders, S. (1994). Control by variable speed signs: Results of the dutch experiment, .

Immers, L., & Logghe, S. (2002). Traffic flow theory. *Katholieke Universiteit Leuven*, .

Jia, Z., Chen, C., Coifman, B., & Varaiya, P. (2001). The pems algorithms for accurate, real-time estimates of g-factors and speeds from single-loop detectors. In *Intelligent Transportation Systems, 2001. Proceedings. 2001 IEEE* (pp. 536–541). IEEE.

Kaplan, E. L., & Meier, P. (1958). Nonparametric estimation from incomplete observations. *Journal of the American statistical association*, *53*, 457–481.

Kerner, B. S. (2004). *The physics of traffic: empirical freeway pattern features, engineering applications, and theory*. Springer Verlag.

Kerner, B. S., & Konhäuser, P. (1994). Structure and parameters of clusters in traffic flow. *Physical Review E*, *50*, 54.

Koshi, M., Iwasaki, M., & Ohkura, I. (1983). Some findings and an overview on vehicular flow characteristics. In *Proceedings of the 8th International Symposium on Transportation and Traffic Flow Theory* (pp. 403–426). University of Toronto: Toronto, Ontario volume 198.

Kotsialos, A., Papageorgiou, M., Diakaki, C., Pavlis, Y., & Middelham, F. (2002a). Traffic flow modeling of large-scale motorway networks using the macroscopic modeling tool metanet. *Intelligent Transportation Systems, IEEE Transactions on*, *3*, 282–292.

Kotsialos, A., Papageorgiou, M., Mangeas, M., & Haj-Salem, H. (2002b). Coordinated and integrated control of motorway networks via non-linear optimal control. *Transportation Research Part C: Emerging Technologies*, *10*, 65–84.

Lee, G. (2006). *Modeling gap acceptance at freeway merges*. Ph.D. thesis Massachusetts Institute of Technology.

Lee, H., Lee, H.-W., & Kim, D. (2000). Phase diagram of congested traffic flow: An empirical study. *Physical Review E*, *62*, 4737.

Lee, J., Dailey, D. J., Bared, J. G., & Park, B. B. (2013). Simulation-based evaluations of real-time variable speed limit for freeway recurring traffic congestion. In *Transportation Research Board 92nd Annual Meeting* 13-4329.

Leutzbach, W. (1987). Introduction to the theory of traffic flow, .

Lu, X.-Y., Kim, Z., Cao, M., Varaiya, P. P., & Horowitz, R. (2010a). *Deliver a Set of Tools for Resolving Bad Inductive Loops and Correcting Bad Data*. California PATH Program, Institute of Transportation Studies, University of California at Berkeley.

Lu, X.-Y., Qiu, T. Z., Varaiya, P., Horowitz, R., & Shladover, S. E. (2010b). Combining variable speed limits with ramp metering for freeway traffic control. In *American Control Conference (ACC), 2010* (pp. 2266–2271). IEEE.

Lu, X.-Y., Varaiya, P., Horowitz, R., Su, D., & Shladover, S. E. (2011). Novel freeway traffic control with variable speed limit and coordinated ramp metering. *Transportation Research Record: Journal of the Transportation Research Board*, *2229*, 55–65.

Maerivoet, S., & De Moor, B. (2005). Traffic flow theory. *arXiv preprint physics/0507126*, .

Messner, A., & Papageorgiou, M. (1990). Metanet: A macroscopic simulation program for motorway networks. *Traffic Engineering & Control*, *31*, 466–470.

Middelham, F. (1999). A synthetic study of the network effects of ramp metering. *AVV Transport Research Centre, Dutch Ministry of Transport, Public Works and Water Management*, .

Ossen, S. J. L. (2008). *Longitudinal driving behavior: theory and empirics*. TU Delft.

Papageorgiou, M., Hadj-Salem, H., & Blosseville, J.-M. (1991). Alinea: A local feedback control law for on-ramp metering. *Transportation Research Record*, .

Papageorgiou, M., Hadj-Salem, H., & Middelham, F. (1997). Alinea local ramp metering: Summary of field results. *Transportation Research Record: Journal of the Transportation Research Board*, *1603*, 90–98.

Papageorgiou, M., & Kotsialos, A. (2000). Freeway ramp metering: An overview. In *Intelligent Transportation Systems, 2000. Proceedings. 2000 IEEE* (pp. 228–239). IEEE.

Payne, H. J. (1971). Models of freeway traffic and control. *Mathematical models of public systems*, .

Ponzlet, M. (1996). *Dynamik der Leistungsfähigkeiten von Autobahnen (Dynamic of motorway-capacity)*. Ph.D. thesis Ph. D.-thesis.

Pueboobpaphan, R., & van Arem, B. (2010). Driver and vehicle characteristics and platoon and traffic flow stability. *Transportation Research Record: Journal of the Transportation Research Board*, *2189*, 89–97.

Rakha, H., Hellinga, B., Van Aerde, M., & Perez, W. (1996). Systematic verification, validation and calibration of traffic simulation models. In *75th Annual Meeting of the Transportation Research Board, Washington, DC*. Citeseer.

Ronaldo, A. et al. (2012). *Comparison of the two Micro-Simulation Software AIMSUN & SUMO for Highway Traffic Modelling*. Ph.D. thesis Linköping.

Schönhof, M., & Helbing, D. (2004). Empirical features of congested traffic states and their implications for traffic modeling. *arXiv preprint cond-mat/0408138*, .

Schreiter, T. (2013). *Vehicle-class specific control of freeway traffic*. TRAIL.

Su, D., Lu, X.-Y., Varaiya, P., Horowitz, R., & Shladover, S. E. (2011). Variable speed limit and ramp metering design for congestion caused by weaving. In *90th Annual Meeting of the Transportation Research Board, Washington, DC*.

Sullivan, D., & Troutbeck, R. (1994). The use of cowan's m3 headway distribution for modelling urban traffic flow. *Traffic engineering & control*, *35*, 445–450.

Treiber, M., Hennecke, A., & Helbing, D. (2000). Congested traffic states in empirical observations and microscopic simulations. *Physical Review E*, *62*, 1805.

Treiber, M., & Kesting, A. (2011). Evidence of convective instability in congested traffic flow: A systematic empirical and theoretical investigation. *Procedia-Social and Behavioral Sciences*, *17*, 683–701.

Treiber, M., & Kesting, A. (2013). The phase diagram of congested traffic states. In *Traffic Flow Dynamics* (pp. 339–352). Springer.

Vasconcelos, L., Silva, A. B., Seco, Á., & Silva, J. P. (2012). Estimating the parameters of cowanâĂŹs m3 headway distribution for roundabout capacity analyses, .

Wiegand, J. D., & Yang, C. (2011). Traffic simulation runs: How many needed? *Public Roads*, *74*.

Yi, J., Lin, H., Alvarez, L., & Horowitz, R. (2003). Stability of macroscopic traffic flow modeling through wavefront expansion. *Transportation Research Part B: Methodological*, *37*, 661–679.

Zackor, H. (1991). Speed limitation on freeways: Traffic-responsive strategies.

Zegeye, S. K., De Schutter, B., Hellendoorn, H., & Breunesse, E. (2009). Reduction of travel times and traffic emissions using model predictive control. In *American Control Conference, 2009. ACC'09.* (pp. 5392–5397). IEEE.

A Stability Indicators

Stability, in combination with lane changes, gives interesting implications for congestion prediction. If the stability of a traffic flow can be determined, one can determine what the influence will be of a lane change. If the traffic is stable, a perturbation will fade out, if traffic is metastable some perturbations will fade out some will not, and if traffic is unstable every perturbation will lead to a breakdown.

The available stability analysis methods describe traffic flow as stable or unstable. The most classical view indicates that traffic is unstable if the traffic density is above the critical density; otherwise it is stable (Pueboobpaphan & van Arem, 2010). Ferrari (1988) identified that traffic is unstable if the average speed of the traffic flow drops rapidly. Yi et al. (2003) based their stability analysis on the nonlinear stability criterion for macroscopic models using the wavefront expansion. However, for real world application it is necessary to use a stability indicator, which is able to measure traffic (in)stability. Pueboobpaphan & van Arem (2010) gave in their paper an overview of the available stability indicators. In the following paragraphs several stability indicators will be tested.

A.1 Classical indicator

This method is based on the basic fundamental diagram. If the occupancy is higher than the critical occupancy, the traffic flow is unstable. This indicator can be described as follows, where ϕ is the stability, o the measured occupancy and o_{crit} the critical occupancy.

$$\phi = \begin{cases} 1 & \text{if } o \leq o_{crit} \\ 0 & \text{if } o > o_{crit} \end{cases}$$

ϕ is zero for unstable traffic and 1 for stable traffic. This stability indicator is not able to determine the influence of a perturbation in a traffic flow. Here, if traffic flow is unstable a breakdown already occurred, in other words: in free flow every perturbation will not cause a breakdown. Due to this interpretation difference of 'stability' this indicator cannot be used.

Considering a traffic breakdown and the capacity drop, the metastable regime can be found using a stochastic capacity method. For a given flow, the metastable regime, indicates that traffic will only breakdown if a large enough disturbance (larger than a critical amplitude) occurs. The critical amplitude becomes larger (Kerner & Konhäuser, 1994) if the traffic flow decreases. In other words; a disturbance, larger than the critical amplitude, occurs in a given occupancy or flow with a certain probability. Used in the fundamental diagram is fixed maximum capacity (q_{crit}) and is treated as a constant value in guidelines (i.e. in the Netherlands: CROW). Ponzlet demonstrated in 1996 that the capacity vary due to external conditions. Ponzlet (1996) implied that the capacity is not a constant value, but has a certain range. This (occupancy) range of capacity will be considered as the metastability range; the probability that traffic will breakdown increases (Kerner, 2004). This gives the opportunity to indicate the border between stability and metastability and on the other hand between metastability and instability.

The most simple method is obtained from the Product Limit Method by Kaplan & Meier (1958) and used by Brilon et al. (2005) to obtain the probability of breakdown at a certain flow. A breakdown is stated as traffic state with a mean speed lower than a certain threshold. Data is collected and the density is ascending ranged. The number of breakdowns (speed in $i+1$ is lower than the threshold, speed in i is higher than the threshold) are determined per traffic density. The

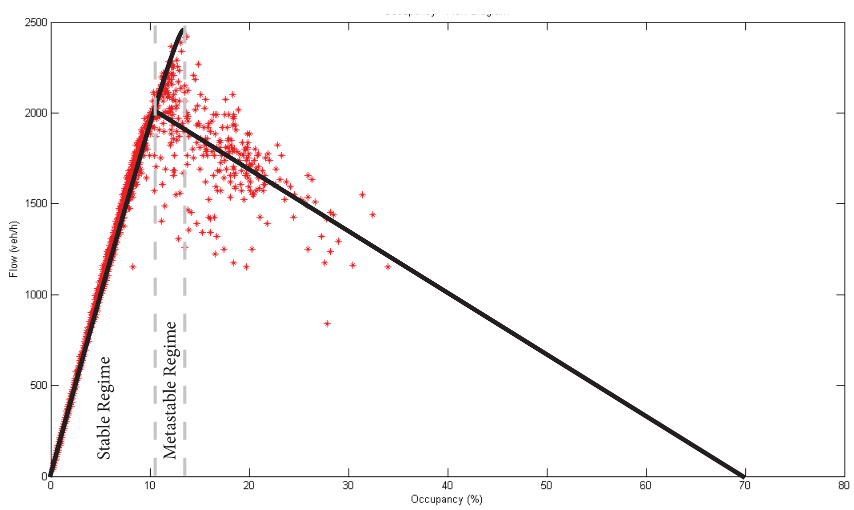


Figure A.1 *Schematic indication of metatable and stable regime based on 5-minutes occupancy - flow fundamental diagram. Notice the capacity drop and that the regimes are only indicated for free flow traffic.*

method of Brilon et al. (2005) is rewritten to traffic occupancy (as the occupancy, in contrast to flow, differs in free flow and congestion):

$$F_c(o) = 1 - \prod_{i:o_i<o} \frac{e_i - d_i}{e_i}, i \in \{B\} \tag{A.2}$$

with:

- $F_c(o)$ is the occupancy distribution function;
- o traffic occupancy;
- o_i traffic occupancy in interval i;
- e_i number of intervals with occupancy o exceeding o_i;
- Number of breakdowns d_i with o_i;
- $\{B\}$ is the number of breakdowns with free traffic in interval i but congested in $i+1$

This method will only contain the value 1 if the maximum observed occupancy was followed by a breakdown. Brilon et al. (2005) have shown that the Product Limit Method will not give a complete distribution function, the highest values observed were not followed by a breakdown. Therefore the Weibull distribution function is used to create a complete distribution. This gives the following stability (ϕ) indicator:

$$\phi = F_c(o) = 1 - e^{-\frac{o}{\beta}^{\alpha}} \tag{A.3}$$

One must notice that α and β are, respectively, shape and scale parameters for the Weibull distribution function to approach the Product Limit Method. Clear is that for this distribution function more data is needed than the used 8-days of data, this will only give a first approximation of the Weibull Distribution Function. Here, the Weibull distribution function is tested on data of April 4, 2013 (Figure A.3).

Note that the indicated method can be used for traffic in free flow, and that simply enough traffic is stable with a probability of breakdown of zero and that traffic is metastable with a probability of breakdown larger than zero. The stability indicator, based on occupancy, is able to 'predict' congestion, as the stability drops before the speed (significantly) drops. Drawback of this indicator is

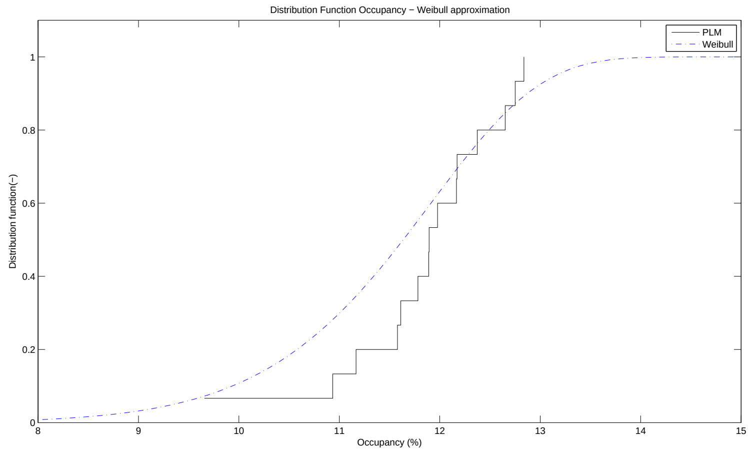


Figure A.2 *Product Limit Method and Weibull distribution function for breakdown, based on data of March 27, 2013 until April 3, 2013. Lane 2, detector 400309 (Caltrans, 2013).*

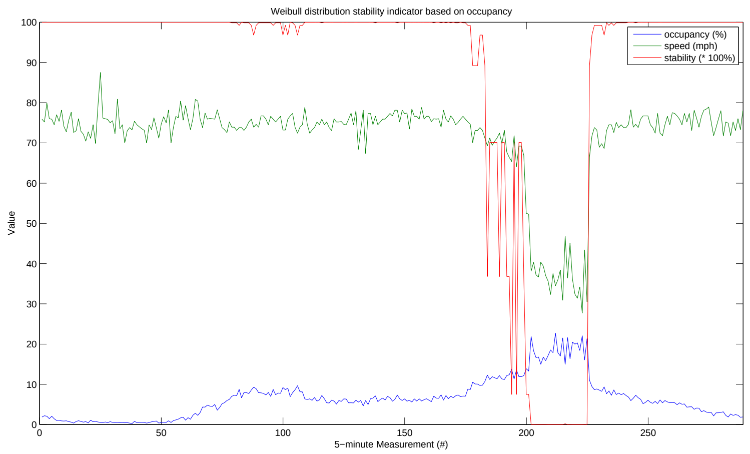


Figure A.3 *Stability indicator based on 8-days of historical data tested on data of April 4, 2013 on the I-880 Highway in California. Lane 2 of detector 400309 (Caltrans, 2013).*

the adaptability to different situations, a stationary and homogeneous flow is assumed and therefore the stability indicator will give the exact same stability for a certain occupancy per day. For instance, if the weather has influenced the driving behavior in the past (larger headways), at a clear day this rainy day will be taken into account. This can be solved by creating several indicators for several conditions, therefore the complexity of the indicator will increase.

A.2 Reliability indicator

Ferrari (1988) uses the word 'reliability' for the stability of a traffic flow. A traffic flow is unstable, according to Ferrari (1988), if a decrease in speed of a certain vehicle (e.g. due to a forced merged vehicle) can cause greater and greater decreases in the speed of the following vehicles. Ferrari (1988) based his indicator on flow, the variance of flow and density. This indicator can be described as follows:

$$\phi = \begin{cases} 1 - p_1(\frac{Q}{1000})^{p_2}\frac{\sigma^2 - p_3}{p_4 - \ln(k)} & \text{if } \sigma^2 > p_3 \& \ln(k) < p_4 \\ 0 & \text{if } ln(k) > p_4 \\ 1 & \text{if } \sigma^2 < p_3 \& \ln(k) < p_4 \end{cases}$$

Where p_1, p_2, p_3, p_4 are control parameters. Here, as the variance of flow increases the stability decreases. Traffic flow is unstable if the log normal density is exceeding a threshold. The reliability indicator is tested on data, using intuitive control parameters, of a detector on the I-880 NB Highway in California, USA (Figure A.4).

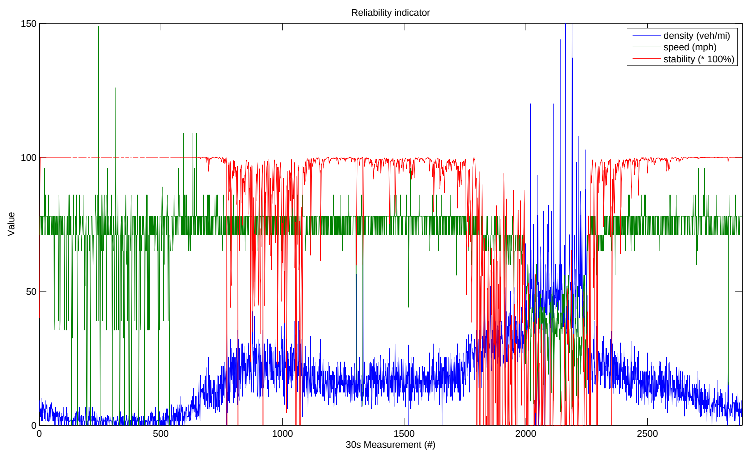


Figure A.4 *Reliability indicator based on 30s measurements data of April 4, 2013 on the I-880 Highway in California. Lane 2 of detector 400309 (Caltrans, 2013). Variance is based on the last 5 measurements.*

The difference with the previous stability indicator, based on historical data, is clear. The reliability indicator of Ferrari (1988) indicates the traffic instability of local flow in a specific situation. This resolves the drawback of the stochastic stability indicator; the reliability indicator indicates the stability of traffic based on the heterogeneity of the flow.

A.3 Conclusion

In this chapter the theory behind traffic stability and their implications for real world traffic are explained. Two methods for indicating the traffic flow stability are indicated. For free-flow traffic a simple stochastic approach, based on historical data, is proposed to indicate whether traffic is stable or metastable. This method gives implications for traffic control, if traffic enters the metastable regime there is a probability that traffic breaks down, necessary is to know that congestion will not propagate further upstream if the upstream off-ramp flow is higher than the on-ramp leaving flow. For traffic control, as traffic is entering the metastable regime, control is necessary. The exact demand for control depends on local traffic characteristics, as can be indicated by the reliability indicator.

B Traffic Control

Traffic control has different objectives, this report will focus on improving the efficiency of the network. Basically, the outflow of traffic can be improved using ramp metering, speed limits, route guidance, dedicated (e.g. HOV) lanes, peak lanes, bi-directional lanes and by applying the 'keep your lane'-signs. This report will only focus on two of these strategies; ramp metering and speed limits. This appendix shows how traffic control is able to limit the inflow of a bottleneck.

B.1 Ramp Metering

Ramp metering is controlling the number of vehicles to enter the mainline road. A traffic light is used and in most cases one or two vehicles are allowed to enter the main road.

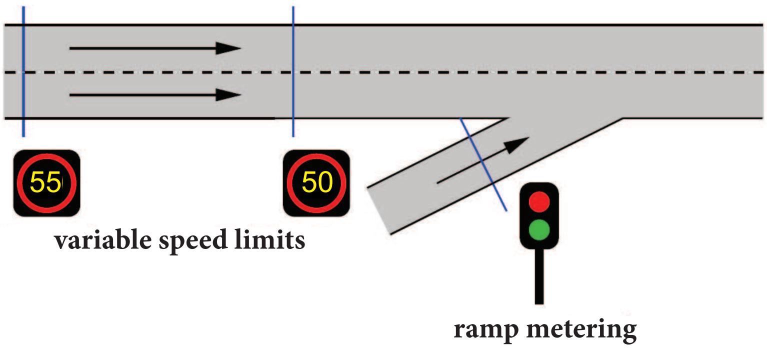


Figure B.1 *Ramp metering and variable speed limits in order to control traffic (after Hegyi et al. (2005)).*

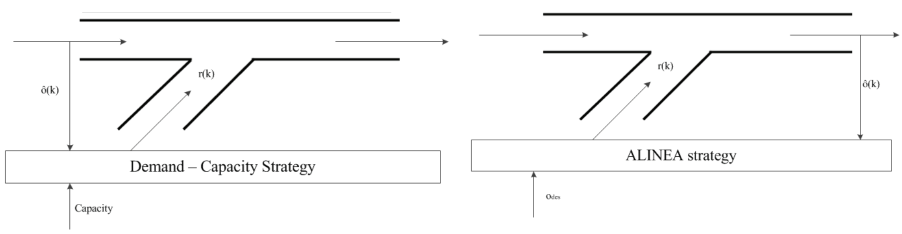
Ramp metering can be implemented by installing traffic lights at the on-ramp of a highway, controlling the amount of traffic flow allowed to enter the highway. It can be used for two purposes; increasing or decreasing travel time. When drivers try to bypass congestion on a highway it can be used to increase the travel time of these drivers (Middelham, 1999). Second, when traffic is the strategy is used to preserve capacity flow on the mainstream and to avoid congestion (the objective of this report) (Kotsialos et al. (2002b);

Basically, ramp metering strategies can be classified as static or dynamic, traffic responsive or feed forward, and local or coordinated (Hegyi, 2004). If the ramp metering strategy is fixed in time, the amount of vehicles allowed to enter the highway is based on historical demands. This demand is assumed to be constant and, is in practice, applied to on-ramps in the rush hours. As the strategy is based on historical data, the strategy is not able to adapt to variations in traffic (Hegyi, 2004). To overcome this issue the ramp metering can be based on on-line data (traffic responsive strategy). The inflow of the traffic is based on the actual traffic conditions.

Local control strategies are focused on controlling the ramp metering of a particular on-ramp. Coordinated ramp metering combines the use of several ramp meters to control the ramp flow on several on-ramps.

B.1.1 Ramp Metering Strategies

Literature has shown that local ramp metering algorithms have a positive effect on the throughput of traffic on busy highways (Middelham (1999); Papageorgiou et al. (1997)). The most popular



(a) Demand Capacity strategy (b) ALINEA strategy

Figure B.2 *Schematic control of a local feed forward ramp metering strategy (a) and a local traffic responsive ramp metering strategy (after Papageorgiou et al. (1997).*

developed algorithm is ALINEA (Figure B.2 (b)) Papageorgiou et al. (1991), where the cycle time is dependent on the difference between the desired (or ideal) occupancy o_{des} (or density) and the measured occupancy ($\hat{o}$) downstream. Choosing occupancy measurements is intuitive; the occupancy is unique for both congested and free flow traffic. Here $r(k)$ indicates the number of vehicles allowed to enter the free in an hour.

$$r(k) = r(k-1) + \zeta[o_{des} - \hat{o}(k-1)] \tag{B.1}$$

Here, ζ is a regulator. The cycle time t (time between two green lights) can be derived from the metering rate and the number of lanes (n) at the on-ramp, i.e.:

$$t = \frac{n \cdot 3600}{r(k)} \tag{B.2}$$

ALINEA is local (dynamic) traffic responsive ramp metering strategy . This is traffic responsive because the rate is based on the measurements downstream of the ramp, i.e.: it is responding to the performance of the strategy. If the rate is based on the measurements upstream it is called a feed forward strategy. A popular strategy is the demand-capacity strategy (Figure B.2 (a)), where the rate is based on the capacity of the mainline and the inflow of the mainline:

$$r(k) = \zeta(\hat{o} - o_{cap}) \tag{B.3}$$

The algorithm has mostly been implemented in The Netherlands. Drawback of this algorithm that it is based on a fixed value, which, in practice, can change over time. The capacity of the network is influenced by a lot of variables (e.g. weather conditions) and this strategy is not able to adapt to capacity-changing situations.

To control more than one on-ramp coordinated ramp metering strategies are used. It is obvious that the coordinated ramp metering strategy is more complex than a local ramp metering strategy. Based on the ALINEA strategy, the METALINE strategy is developed.

$$\mathbf{r}(k) = \mathbf{r}(k-1) - \mathbf{K_1}[\hat{\mathbf{o}}(k) - \hat{\mathbf{o}}(k-1)] - \mathbf{K_2}[\hat{\mathbf{O}}(k) - \mathbf{O_{des}}(k)] \tag{B.4}$$

where

- $\mathbf{r} = [r_1, \ldots, r_m]^T$ is the vector of m controllable on-ramps;
- $\hat{\mathbf{o}} = [\hat{o}_1, \ldots, \hat{o}_N]^T$ is the vector of N measured occupancy along the highway;
- $\hat{\mathbf{O}} = [\hat{O}_1, \ldots, \hat{O}_m]^T$ is the vector of m measured occupancy downstream of the on-ramps;
- $\mathbf{O_{des}} = [O_{des,1}, \ldots, O_{des,m}]^T$ is the vector of the desired value at controllable on-ramp m;
- $\mathbf{K_1}$ and $\mathbf{K_2}$ are two regulator matrices.

The coordinated ramp metering strategy where the metering rate is computed based on the change in measured occupancy, and the deviation of occupancy from critical occupancy for each segment that has a controlled on-ramp. Papageorgiou et al. (1997) has shown that ALINEA is, in absence of accidents, not inferior to METALINE. This indicates that ALINEA is a standard to which other strategies can be compared. Other studies (e.g. Bhouri et al. (2011)) have shown that other coordinated ramp metering strategies are outperforming the ALINEA strategy. This shows the potential of a coordinated ramp metering strategy.

Ramp Metering Drawbacks

At the most of the sites were a ramp metering strategy is installed only one or two vehicles are allowed to enter the highway per green time (Elbers, 2005). This gives some limitations to this algorithm. Assuming that the green time is 2.0 seconds (it is depending on reaction time and the acceleration capabilities of the vehicle (Elbers, 2005) or is a fixed value), the amber time is 0.5 seconds (depending on the speed downstream of the ramp meter (Elbers, 2005) or is a fixed value) and the red time, depending on the metering rate (Elbers, 2005), is assumed to be 2.0 seconds. The maximum metering flow, now, is 800 veh/h. As the difference between the measured occupancy and the desired occupancy is high, and the calculated metering flow is higher than 800 veh/h it is not necessary ton control traffic. In other words; traffic should be controlled as the calculated ramp metering is less than 800 vehicles per hour.

As a queue is developing upstream of the traffic light, it is possible that the queue spills back upstream to adjacent infrastructure. As the standard controller is not able to overcome this undesired behavior, the ramp metering should be switched off when a queue is becoming too large. To measure the length of the queue a complex measurement method is needed. Therefore, suggested is to locate an additional detector at the end of the on-ramp. If this detector is occupied, the control should be switched off. Consequently, this is negatively influencing the performance of the algorithm but is necessary for real world application.

If the mainstream flow is increasing the measured occupancy downstream can exceed the desired occupancy. The ALINEA algorithm uses a single value regulator to control the ramp meter flow.If the occupancy exceeds the desired value the ramp meter rate will lower. Assuming that traffic is controlled in the metastable regime, a suddenly change in mainline traffic demand can cause that the measured occupancy is approaching the instable state. Adding a second regulator to the algorithm will positively influence this behavior. The control is more aggressive as the measured occupancy has exceeded the desired occupancy

The extensions to the algorithm can be described as follows: the control is switched off if the maximum metering flow is reached or when the queue (w) is as long as the on-ramp length l:

$$r(k) = \begin{cases} \text{off} & \text{if } \hat{o}_{des} - o(k-1) > 800 \\ \text{off} & \text{if } w(k-1) \geq l \\ r(k-1) + \zeta_1(o_{des} - \hat{o}(k-1)) & \text{if } \hat{o}(k-1) \leq o_{des} \\ r(k-1) + \zeta_2(o_{des} - \hat{o}(k-1)) & \text{if } \hat{o}(k-1) > o_{des} \end{cases} \tag{B.5}$$

where $\zeta_1 < \zeta_2$

Literature (Papageorgiou et al., 1997) has shown that ramp metering strategies are able to reduce travel time. To overcome the drawbacks of ramp metering, the standard ALINEA strategy is slightly improved for implementation in real world.

B.2 Speed Limits

Nowadays, a lot of highways (especially in The Netherlands) are equipped with variable speed limit signs. These signs are currently used to increase safety by lowering speed limits upstream of congested areas (Hegyi et al., 2005). Although, the signs can also be used to increase traffic flow using a speed limit strategy.

Literature shows two views on the speed limits; homogenization and preventing traffic breakdown. The idea of homogenization is that speed limits can reduce differences in speed and density. The homogenization approach is not able to prevent traffic breakdown. The second approach focuses on preventing or resolving of a traffic breakdown by preventing or resolving high densities areas (Hegyi, 2004). This approach uses speed limits lower than the critical speed to limit the inflow of the areas (Figure B.3). Several speed limit strategies, limiting the inflow, have been developed. The technique of the speed limit can be illustrated using the fundamental diagram (Hegyi et al., 2005). When traffic on the mainline is in state 1, then it can be assumed that traffic metastable or unstable. A small perturbation from the on-ramp can cause a breakdown. By setting the speed limit lower than the critical speed, the flow and density changes to somewhere between 2 and 3. The speed limits are set upstream of the bottleneck, limiting the inflow of the bottleneck (Figure B.1). The decrease of inflow creates some space for on-ramp traffic and decreases the chance of breakdown.

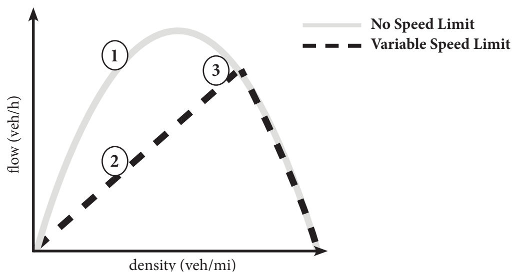


Figure B.3 *By setting a speed limit the (schematic) fundamental diagram changes from the gray to the dotted black line. Setting speed limits while traffic is in state 1, flow changes to somewhere between 2 and 3. Note that the speed limit should be lower than the static speed limit (after Hegyi et al. (2005)).*

The fundamental diagram changes due to the influence of a variable speed limit. The capacity flow reduces and, therefore, fewer vehicles are entering the bottleneck. As less vehicles are entering the bottleneck, the stability of the flow increases and a chance of breakdown decreases.

B.3 Conclusion

This appendix has shown that ramp metering and variable speed limit strategies are able to limit the inflow of the bottleneck. A ramp metering strategy is using a traffic light to limit the on-ramp leaving flow and variable speed limits used variable message sings to change the fundamental diagram.

C Model Development

To determine the performance of the several strategies, the microsimulation software Aimsun is used. This appendix describes the development of the model in the simulation software.

C.1 Aimsun

It is widely accepted that simulation is a technique to provide an experimental test to compare controls. Traffic simulation models can be classified as microscopic, mesoscopic or macroscopic. Old studies of Boxill & Yu (2000) and Bloomberg et al. (2003) give a comparison between different simulation programs, for this report it is necessary whether the software is able to implement new strategies and is able to model highway traffic. One should notice that an up-to-date comparison and evaluation of simulation software is not available. Based on available studies and the availability of software programs, Aimsun is used. Aimsun is a commercial simulation software for macroscopic, mescoscopic and microscopic simulation. In this case, the microsimulation modeling part of the software is used. The simulation software is based on the paper of Gipps (1986).ted that simulation is a technique to provide an experimental test to compare controls. Traffic simulation models can be classified as microscopic, mesoscopic or macroscopic. Old studies of Boxill & Yu (2000) and Bloomberg et al. (2003) give a comparison between different simulation programs, for this report it is necessary whether the software is able to implement new strategies and is able to model highway traffic. One should notice that an up-to-date comparison and evaluation of simulation software is not available. Based on the studies and the availability of software programs, Aimsun is used. Aimsun is a commercial simulation software for macroscopic, mescoscopic and microscopic simulation. In this case, the microsimulation modeling part of the software is used. The simulation software is based on the paper of Gipps (1986).

C.2 Model Development

Using a microsimulation model for a specific traffic analysis consist out of seven major tasks (Dowling et al., 2004):

1. Identification of Study Purpose, Scope and Approach;
2. Data Collection and Preparation;
3. Base Model Development;
4. Error Checking;
5. Calibration;
6. Alternatives Analysis;
7. Final Report and Technical Documentation.

C.2.1 Study Purpose, Scope and Approach

Purpose of the study is testing control strategies' performance. Here, considered is a part of the Interstate 880 (I-880) in California, USA is considered. The I-880 is a 46 mile long highway from San-Jose, CA to Oakland, CA and is a major congested highway. Considering the time-space diagram of the I-880 NB, several congested areas can be identified. Empirical findings have shown that the I-880 NB has, in the evening hours, two recurrent bottlenecks: at post mile 15.12 (at Auto mall Pkway, Fremont) and post mile 30.0 (Hesperian Blvd, San Leandro).

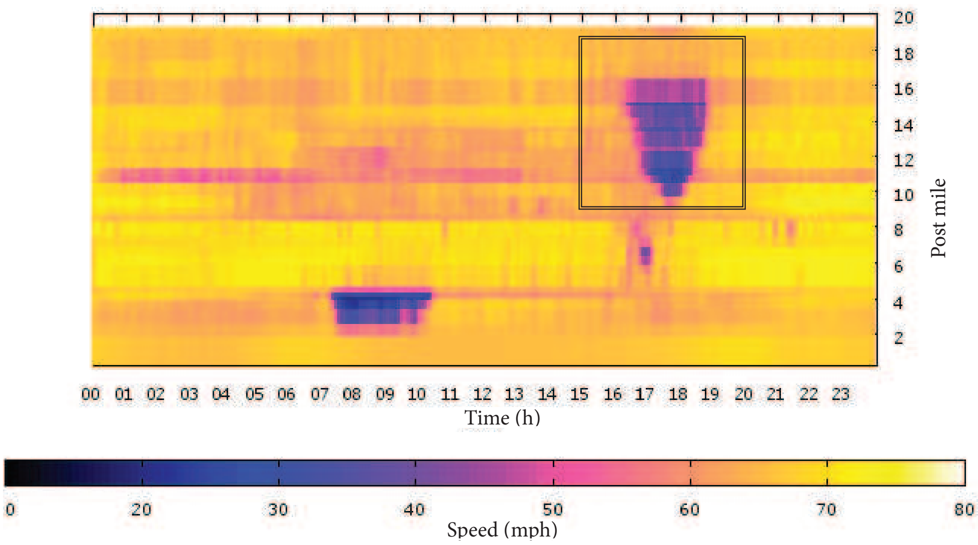


Figure C.1 *The time-space speed contour plot for the I-880 NB on April 4, 2013 (Caltrans, 2013). Bordered is the considered section in the simulation model. The dark areas indicate low speed and can be determined as congestion areas.*

The recurrent bottleneck at post mile 15.12 is considered, at April 4 2013. This day is considered, taken into account several restrictions. Based on an internal report it is possible that the congestion, started at post mile 30 and post mile 15 get coupled. On this day the congestion does not get coupled with other large high densities areas (Figure C.1). Besides, a lot of traffic accidents occur during congestion hours. It was shown that during congestion, caused by the on-ramp near post mile 15 at April 4 2013, no accidents occurred in the evening hours. At the considered section an HOV-lane is available from 3pm until 7pm, empirical findings have shown that the HOV-lane is not always used properly during congestion hours. Implemented in the simulation software is the section from post mile 9 (Dixon Landing Road, Milpitas) to post mile 18.66 (Central Ave, Fremont) implemented from 3pm to 8pm (Figure C.2).

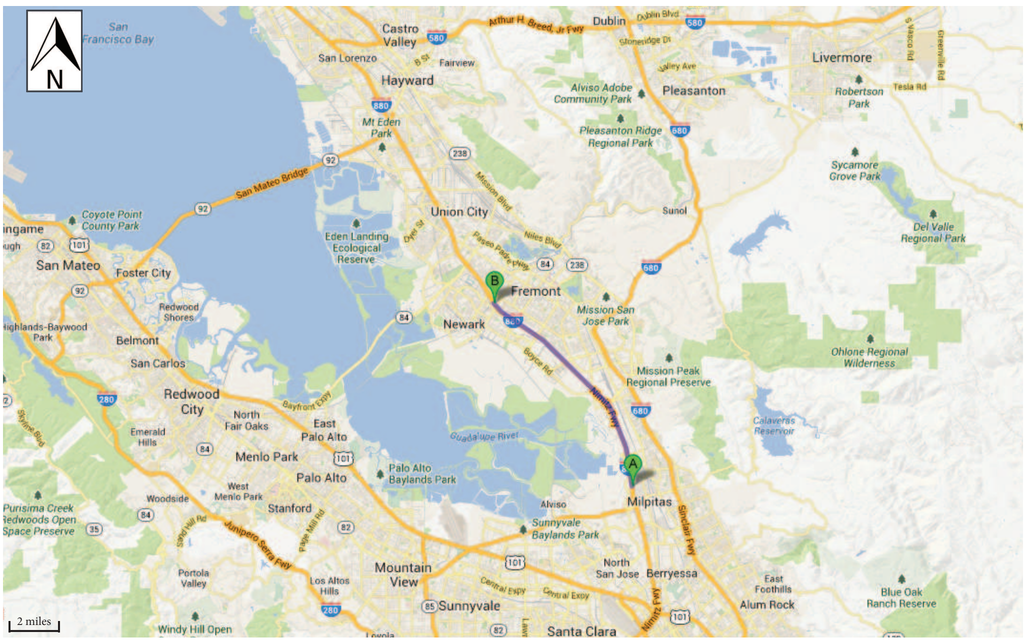


Figure C.2 *The modeled road section: from Milpitas to Fremont (Google, 2013).*

C.2.2 Data Collection and Preparation

The I-880 NB is calibrated based on 5-minutes aggregated data of the PeMS database (Caltrans, 2013). As the aggregated data is giving too high speeds (the speed limit on the highways is 65 mph) tried is to capture the general pattern the of the speed in the simulation software.

Collection and Aggregation

Used will be data from the PeMS database. The database provides measurements made on the California highways. PeMS receives measurements from, in this case, mostly single loop detectors. The single loop detectors provide 30 seconds raw measurements including flow and occupancy. The 30-second data is aggregated to 5 minute data, this data can, in case of not-working loop detectors, include holes. These holes, missing data, are filled using the Daily Statistics Algorithm (Chen et al., 2003). The imputation of data is based on the correlation between neighbor detectors. The used data for this purpose is the 5-minute aggregated data of flow and speed. As speed is not directly measured this speed is calculated using a g-factor (Jia et al., 2001):

$$v(t) = g(t)\frac{q(t)}{o(t)T} \tag{C.1}$$

Here, the speed ($v(t)$) is depending on the flow ($q(t)$), occupancy ($o(t)$), g-factor $g(t)$ and detector interval . The g-factor varies and is based on the length of the loop, average length of the vehicle and free speed. Although this g-factor approach gives a better approximation than using a constant g-factor, it is known that the g-factor approach is overestimating the speed.

C.2.3 Base Model Development

The goal of base model development is developing a model that is verifiable, reproducible, and accurate (Dowling et al., 2004). Satellite data is used to produce the first layer (connection / node diagram). This indicates real world road structure, number of lanes, location on-ramps, location off-ramps etc. The connection between two highway sections are implemented. Basic traffic demand is added to run the simulation software. Assumed is, during developing the model that highway sections have no slope.

A schematic overview of the model can be found in Appendix E.

C.2.4 Error Checking

The goal of adding basic traffic demand is error-checking. The error-checking task is necessary to identify and correct model coding errors (Dowling et al., 2004). The error-checking is an automated process in the Aimsun simulation software. Shown was that vehicles were waiting outside the network (vehicles were not able to enter the on-ramp as it was congested). Therefore extra highway sections are placed upstream of the on-ramps.

C.2.5 Microsimulation Model Calibration

It is widely accepted that simulation is a technique to provide an experimental test to compare different controls. The outcomes of the simulation can be used for decision making, and is an useful to tool to find the most useful control. The process, performed, is based on Rakha et al. (1996) and consist out of three phases: (1) model verification, (2) model validation and (3) model calibration. Only phase 3 is done by the user, phase 1 and 2 belong to the developer of the simulation software.

Prior to applying the model to a study, the model needs to be calibrated. Traffic model calibration consists out of selecting the input parameter values that reflect the local study area's network. Calibration is the process of selecting the set of parameters to meet the field data. In general, during the model calibration process two problems are face: (1) availability field data and (2) provided data. The field data is limited in quality and quantity. As the simulation software is able to provide data for every part of the network, the field data only provides data for portions of the network (e.g. loop detectors). Besides, detector data does not provide essential calibration parameters as driving routing behavior and origin-destination demands. The input parameters, used for calibration, should be applied to several days. Unfortunately, the model is calibrated based on one day data and therefore the model calibration process is not finished yet.

During the calibration process the virtual detectors are used to match the real world detectors. The virtual detectors are located at the same location as the real world loop detectors (Appendix E), as the virtual detectors show the same measurements as the real loop detectors the model is said to be 'calibrated'. Calibration is an iterative process where the flows (inflow mainline, inflow on-ramps, outflow off-ramps and outflow mainline) and the parameter settings (minimum gap, desired speed etc.) of the model are set. First, the most upstream and downstream detector are used for calibration. This implies that the mainline inflow and mainline outflow are captured in the model. Using an iterative process, changing model parameters and traffic demand, tried is to capture the detector measurements in the simulation model. Detectors placed on the ramps were not working or were not available, the up- and downstream located detectors are used for setting up the ramp flow.

The calibration is based on the available data on the highway stretch. The results of the calibration of flow and speed (both averaged over 5 simulation runs) are presented in Appendix D. The comparison, between the model and the PeMS data, is based on a statistical technique and identified by the root mean square error (RMSE). RMSE is based on the difference between the Aimsun values (y) and the 5-minutes aggregated data ($\hat{y}$). The lower the error, the higher the validity of the model:

$$RMSE = \sqrt{\left(\frac{\sum_{t=1}^{n}(y_t - \hat{y}_t)^2}{n}\right)} \tag{C.2}$$

It can be concluded that the RMSE for this purpose is not the best statistical technique. As said, the g-factor approach is overestimating speed and the general pattern is tried to capture. The RMSE does not identify a general pattern. In literature the RMSE technique is widely used for traffic calibration. Besides, the RMSE shows some undesired low error values in for instance (Figure D.2 (b)). Therefore, it is suggested to use an accumulated error technique.

Used are four different (hour-during) demand levels. Clear is that the congestion demand is significantly higher than free flow demand (Figure C.3). In these demands, assumed is 5 % percentage of trucks, a variable percentage of HOV-cars and cars. As the vehicle characteristics (length, width and other characteristics) per vehicle type do not differ it can be concluded that the model is not well reflecting the feature of mixed traffic.

- Free flow demand: 10.100 veh/h (3pm - 4pm and 7pm - 8pm)
- Transition free flow - congestion demand: 10.800 veh/h (4pm - 5pm)
- Congestion demand: 13.600 veh/h (5pm - 6pm)
- Transition congestion - free flow demand: 11.200 veh/h (6pm - 7pm)

Currently, no approach is available trying to capture the amount of HOV-vehicles. As the loop detectors are also placed on the HOV-lane, it is possible that HOV-vehicles are still driving in the other lanes. In the simulation environment the amount of HOV-vehicles is found using an iterative process. Future research is needed to find the amount of HOV-vehicles out of macroscopic traffic data.

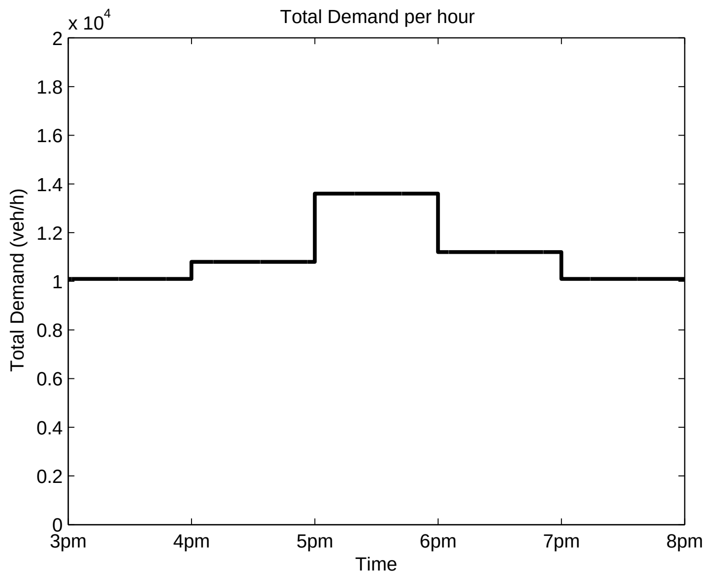


Figure C.3 *Demand per hour used for calibration of the simulation model. Total demand includes all inflow locations (on-ramps and mainline inflow) and all vehicle types.*

The results show that the calibration process is not finished yet. Besides the fact that the calibration should be based on several days, still some high values RMSE are shown. This can be partly validated by the fact that some detectors are showing too high speeds. On a segment where the maximum (static) speed limit is 65 mph, some detectors were showing (based on the g-factor approach) a speed of 80 mph. Other issues is the lane distribution, in all cases the flow in lane 2 is too low. Other detectors are showing too low flow in combination with too low speeds. It is shown in Appendix D that the simulation software was not able to capture big variances in flow and speed. This indicates that more work should be spent on the calibration to further improve and compare the data (variance in densities, variance in speed, accumulated error etc.) As just limited time is available, the model is assumed to be calibrated and one can continue with analysis step.

C.2.6 Alternative Analysis

Now the model is assumed to be calibrated, the model can be used for analyzing alternatives. Here, different strategies are implemented using the API of Aimsun, coded in $VisualC++$. The alternatives are tested based on the total time spent (TTS) and the total traveled distance. Multiple simulation runs are required to get an average result in order to deal with stochastic processes in Aimsun, and to determine the impact of the strategies. The more simulation runs are performed, the higher accuracy in the resulting values will be gained. Based on statistics the following relationship determines the number of simulations which should be performed:

$$n \geq \frac{Z^2}{d^2}\sigma^2 \tag{C.3}$$

The number of simulation runs n depends on the reliability of the statement (Z), the variation in the phenomenon (σ) and the accuracy on the state one wants to make (d). Clear is that the number of simulation runs depends on a lot of criteria, and therefore it is very difficult to determine the number of runs with a required accuracy (Burghout, 2004). Wiegand & Yang (2011) did research to determine the number of simulation runs. Clear is that one simulation run is not sufficient, Wiegand & Yang (2011) concluded that the most notable differences were between the 5- and 10-run tests in the simulation software CORSIM, and results generally became stabilized after 10 to 15 simulation runs. Concluded can be that at least 10-simulation runs should be made. Because a simulation run is very time consuming, only 5 runs per strategy could be made. Assumed is, that this number of simulation runs, is sufficient to get an indication of the impact of the strategies. Notice that all scenarios are based on the same random seed numbers.

Alternatives Set-up

Different strategies are implemented using the API of Aimsun. The local ramp metering strategy ALINEA has proven it is able to reduce travel time, is outperforming feed forward ramp metering strategies, and is therefore implemented. In Appendix B the strategy is improved for real world application, this strategy is implemented. As the most highways have implemented ramp metering, a coordinated ramp metering controlling more on-ramps is implemented. Besides, the integrated control proposed is implemented. To consider the influence of variable speed limits, a strategy only using variable speed limits is also implemented.

- Uncontrolled traffic
- Local ramp metering strategy ALINEA (as proposed in section 5.1 and Appendix B)
- Variable Speed Limit strategy (proposed in section 5.2.1)
- Coordinated Ramp Metering strategy (proposed in section 5.2.2)
- Integrated Variable Speed Limit and Coordinated Ramp Metering strategy (proposed in section 5.2)

The demand levels during all alternatives are the same. In real world it can be assumed that more vehicles will enter the highway if there is no congestion. In all these cases, vehicles are not allowed to change their destination, departure time or cancel the trip. Aimsun gives the possibility to model this behavior, but is not considered in this case.

For the variable speed limit strategy and the integrated variable speed limit and coordinated ramp metering strategy the highway is divided into links. The length of the links vary between 0.25 miles and 1.25 miles and the total length of the considered area is 9.5 miles. A total of 13 links are considered, with a total of 11 on-ramps and 7 of-ramps. For an overview of the simulation network, referred is to Appendix E. Some control strategies require detectors at all on and off-ramps. In real world, not all on- and off-ramps do have detectors. Several virtual detectors are placed in the simulation software at ramps.

C.2.7 Final Report

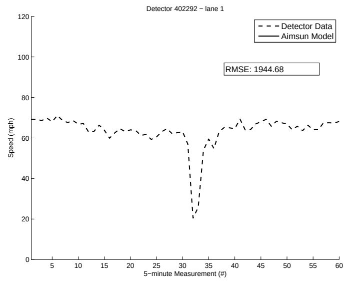
This task involves summarizing the analytical data (Dowling et al., 2004). This thesis can be considered as the final report.

C.3 Discussion

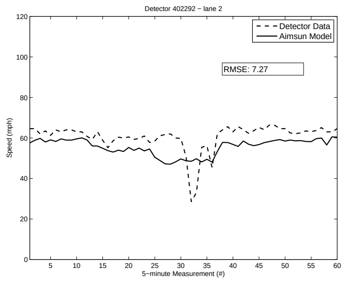
Aimsun has shown some irregular behavior during the calibration process and simulation runs. Aimsun was not able to 'release' the on-ramp. Releasing the on-ramp is configured as a constant green ramp meter, the software has shown vehicles waiting for a green light, therefore a released ramp meter is simulated as a flow metering with a very high flow.

D Calibration Results

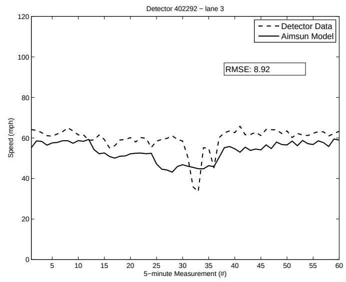
This appendix gives the calibration results. Here, the measurements of the loop detectors are compared to the modeled situation in Aimsun. Note that lane 1 is an HOV lane. The detectors are sorted from upstream to downstream. For an overview of the simulation network and the location of the detectors, referred is to Appendix E.



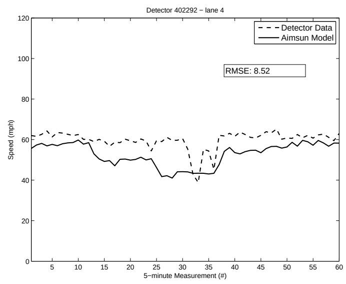
(a) Lane 1 - Speed



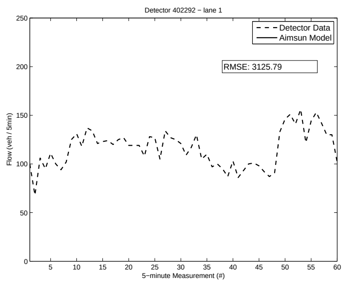
(b) Lane 2 - Speed



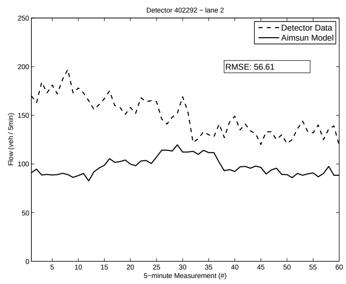
(c) Lane 3 - Speed



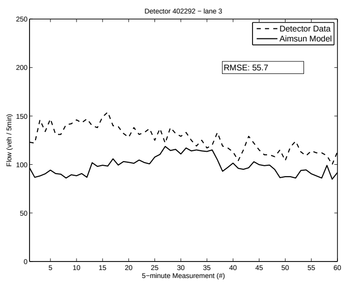
(d) Lane 4 - Speed



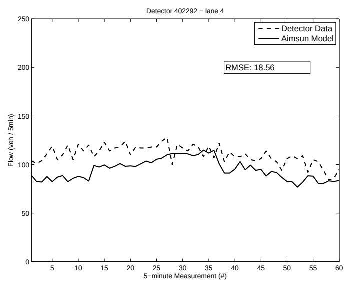
(e) Lane 1 - Flow



(f) Lane 2 - Flow

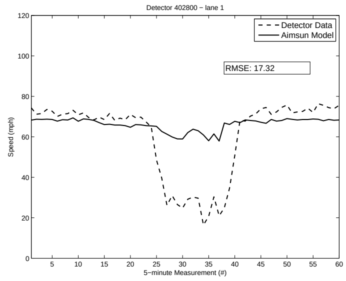


(g) Lane 3 - Flow

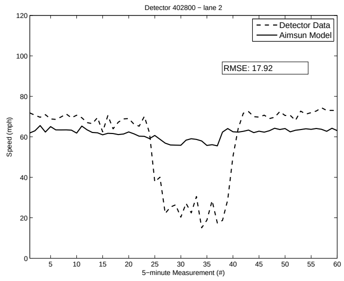


(h) Lane 4 - Flow

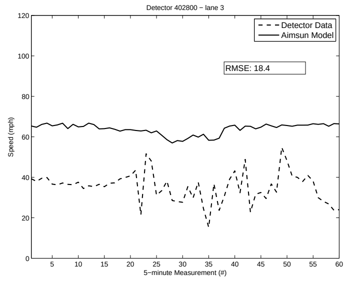
Figure D.1 *Calibration results of detector 402292 - post mile 9.80. Lane 1 has no model results due to technical issues.*



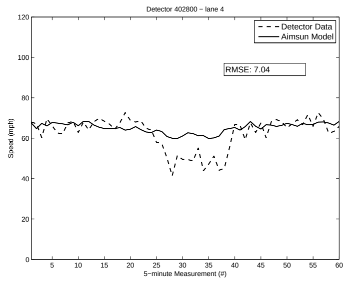
(a) Lane 1 - Speed



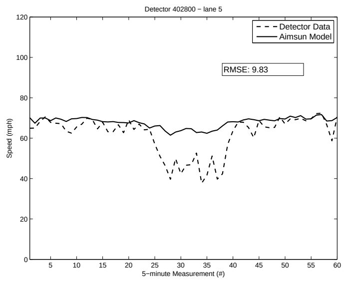
(b) Lane 2 - Speed



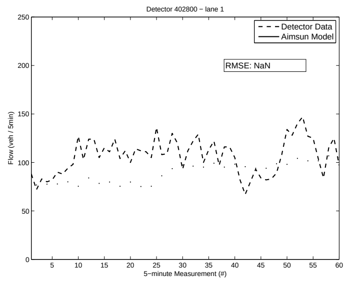
(c) Lane 3 - Speed



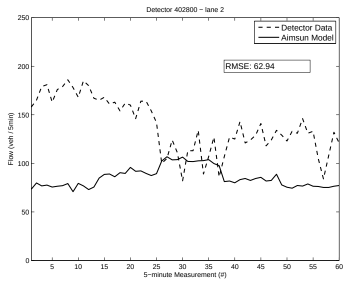
(d) Lane 4 - Speed



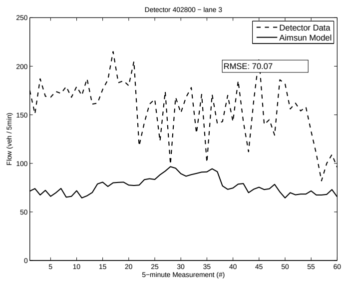
(e) Lane 5 - Speed



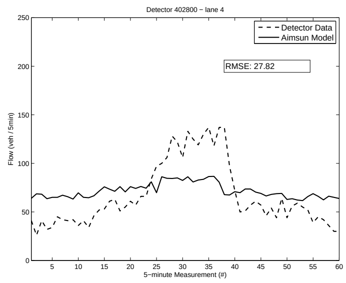
(f) Lane 1 - Flow



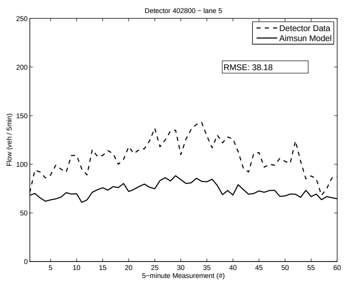
(g) Lane 2 - Flow



(h) Lane 3 - Flow

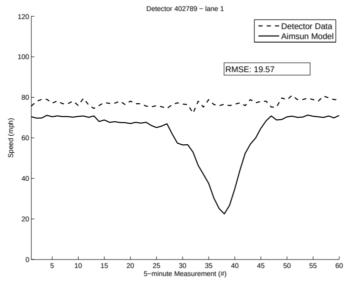


(i) Lane 4 - Flow

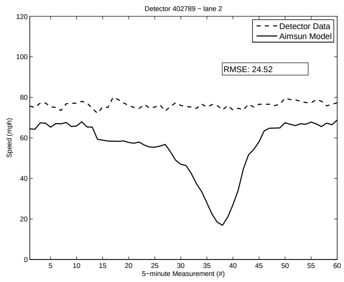


(j) Lane 5 - Flow

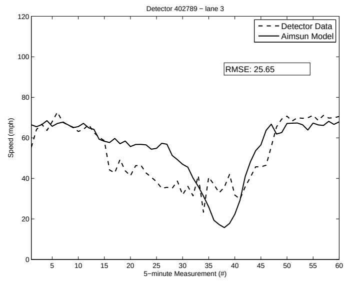
Figure D.2 *Calibration results of detector 402800* **-** *post mile 11.7*



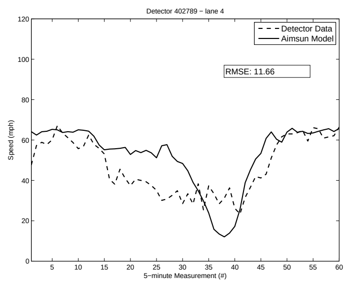
(a) Lane 1 - Speed



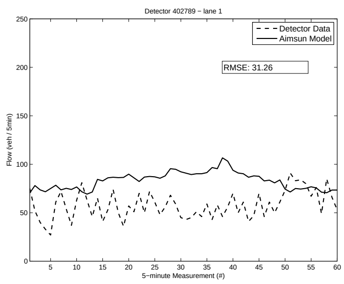
(b) Lane 2 - Speed



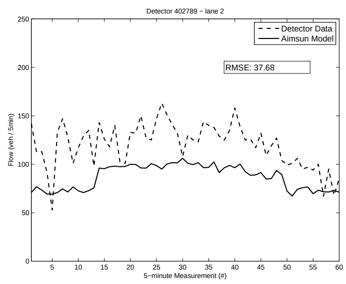
(c) Lane 3 - Speed



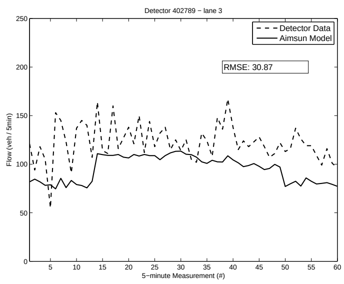
(d) Lane 4 - Speed



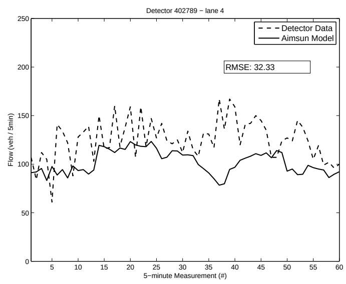
(e) Lane 1 - Flow



(f) Lane 2 - Flow

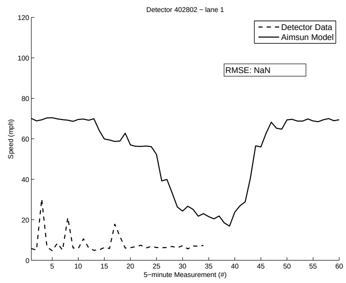


(g) Lane 3 - Flow

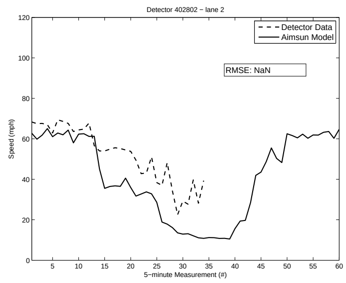


(h) Lane 4 - Flow

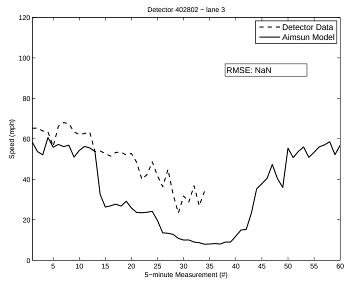
Figure D.3 *Calibration results of detector 402789 - post mile 12.4*



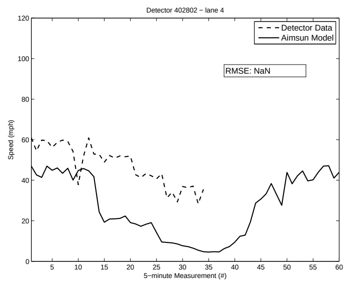
(a) Lane 1 - Speed



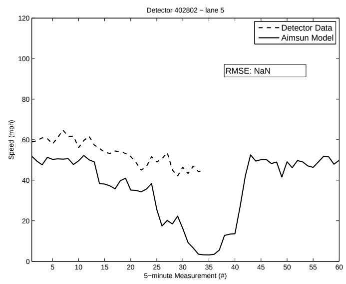
(b) Lane 2 - Speed



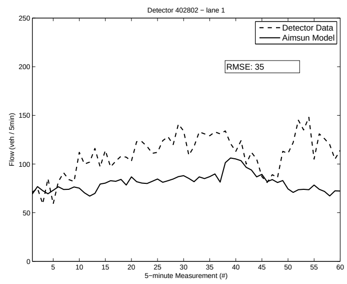
(c) Lane 3 - Speed



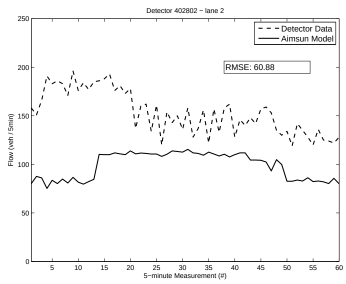
(d) Lane 4 - Speed



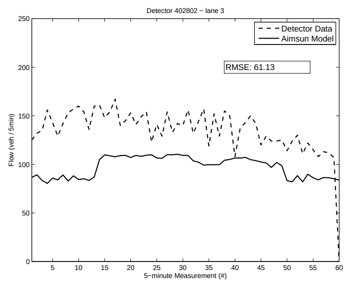
(e) Lane 5 - Speed



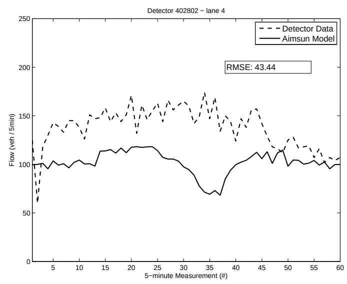
(f) Lane 1 - Flow



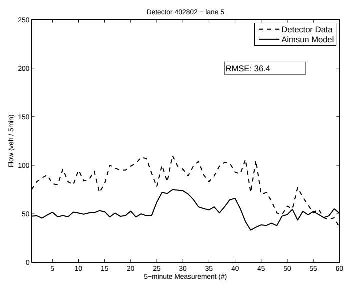
(g) Lane 2 - Flow



(h) Lane 3 - Flow

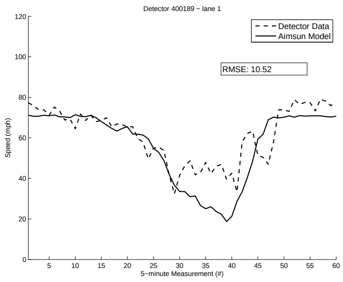


(i) Lane 4 - Flow

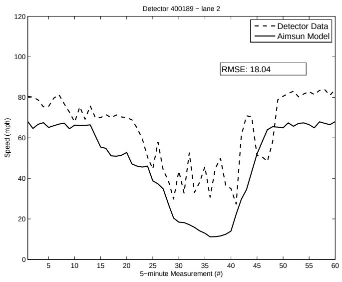


(j) Lane 5 - Flow

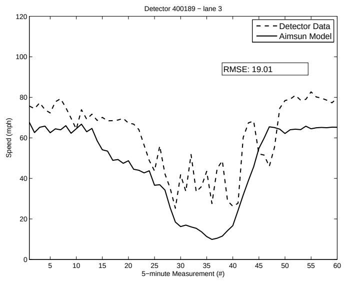
Figure D.4 *Calibration results of detector 402802 - post mile 13.0*



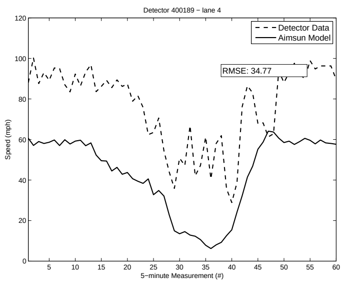
(a) Lane 1 - Speed



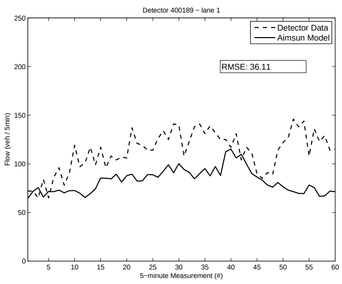
(b) Lane 2 - Speed



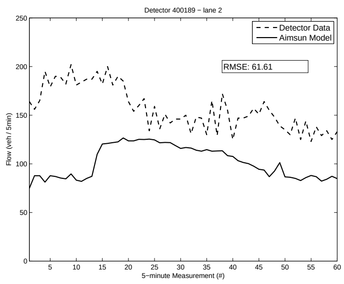
(c) Lane 3 - Speed



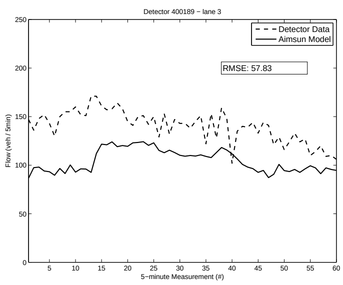
(d) Lane 4 - Speed



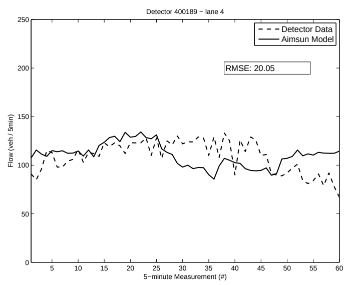
(e) Lane 1 - Flow



(f) Lane 2 - Flow

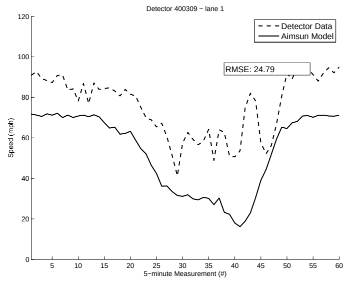


(g) Lane 3 - Flow

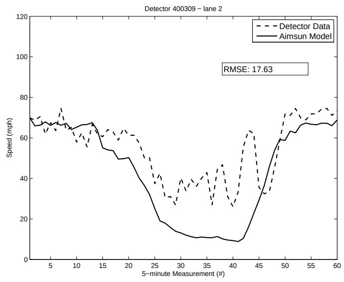


(h) Lane 4 - Flow

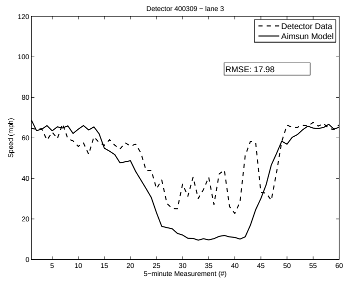
Figure D.5 *Calibration results of detector 400189 - post mile 13.51*



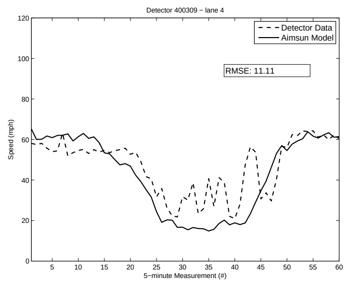
(a) Lane 1 - Speed



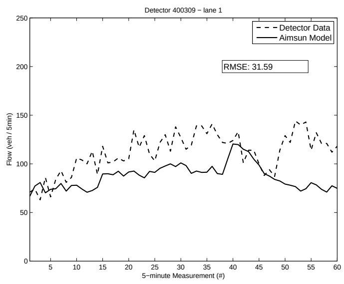
(b) Lane 2 - Speed



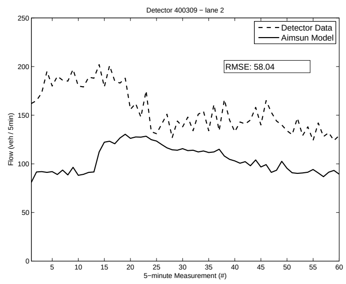
(c) Lane 3 - Speed



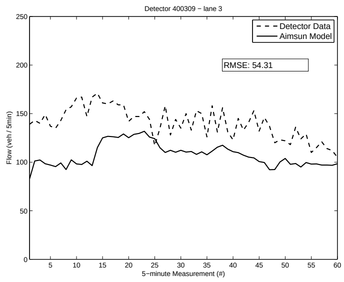
(d) Lane 4 - Speed



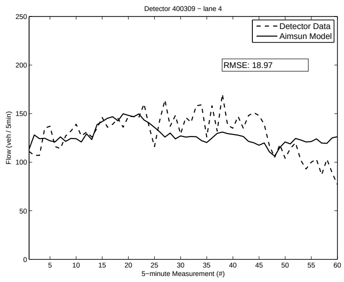
(e) Lane 1 - Flow



(f) Lane 2 - Flow

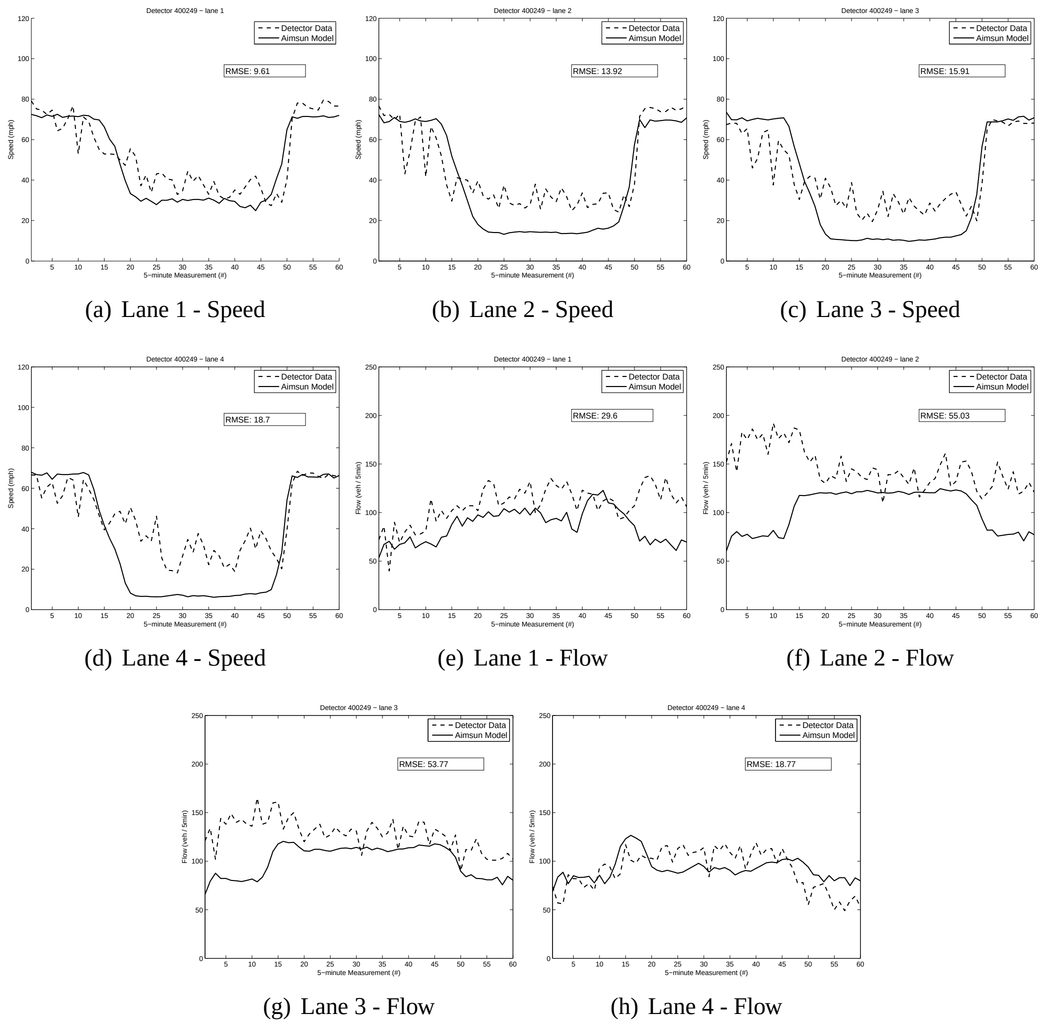


(g) Lane 3 - Flow



(h) Lane 4 - Flow

Figure D.6 *Calibration results of detector 400309 - post mile 13.7*

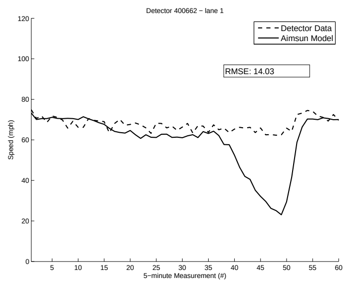


(a) Lane 1 - Speed (b) Lane 2 - Speed (c) Lane 3 - Speed

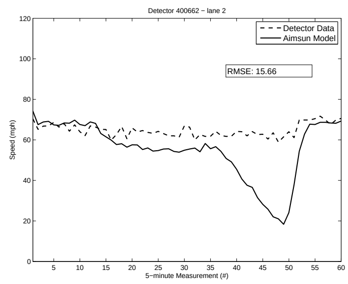
(d) Lane 4 - Speed (e) Lane 1 - Flow (f) Lane 2 - Flow

(g) Lane 3 - Flow (h) Lane 4 - Flow

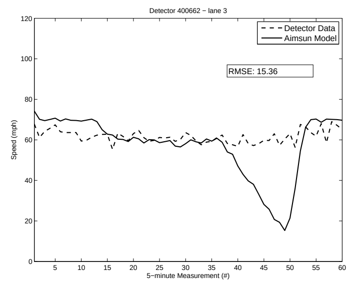
Figure D.7 *Calibration results of detector 400249 - post mile 14.89*



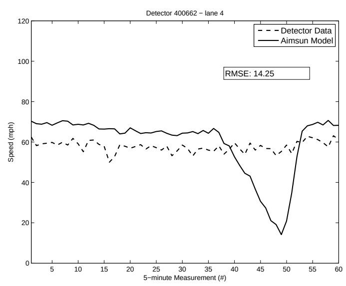
(a) Lane 1 - Speed



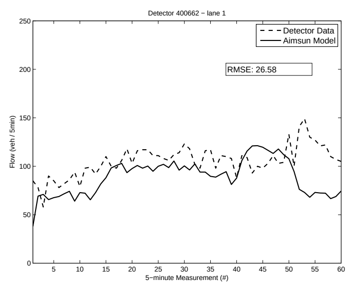
(b) Lane 2 - Speed



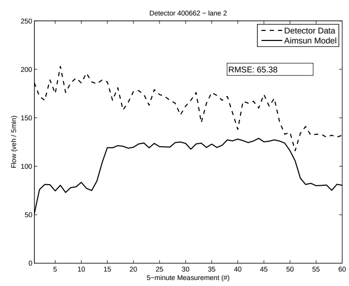
(c) Lane 3 - Speed



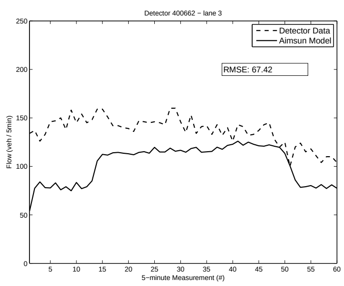
(d) Lane 4 - Speed



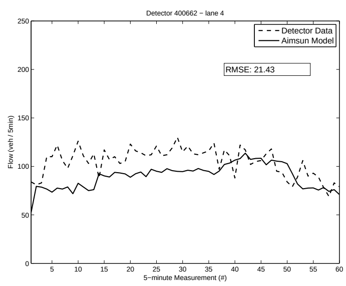
(e) Lane 1 - Flow



(f) Lane 2 - Flow

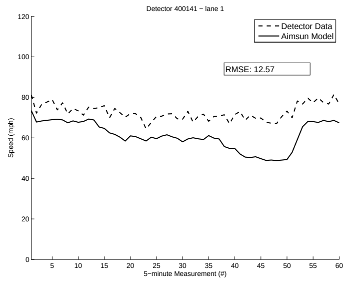


(g) Lane 3 - Flow

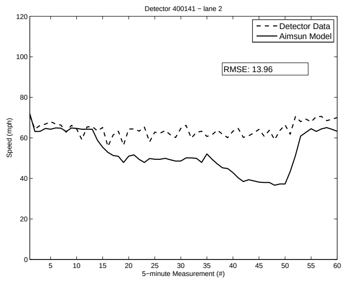


(h) Lane 4 - Flow

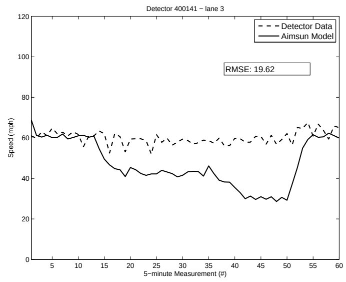
Figure D.8 *Calibration results of detector 400662 - post mile 16.45*



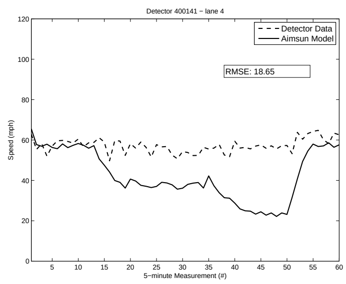
(a) Lane 1 - Speed



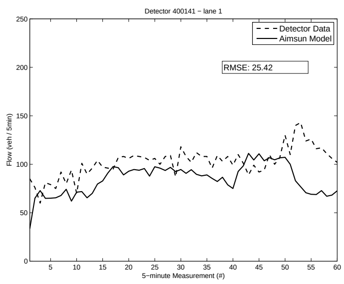
(b) Lane 2 - Speed



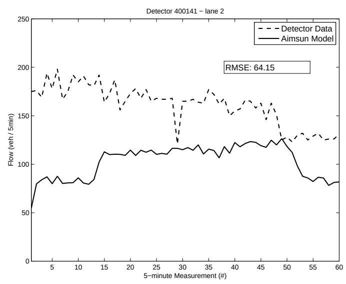
(c) Lane 3 - Speed



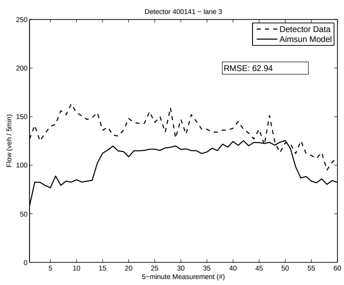
(d) Lane 4 - Speed



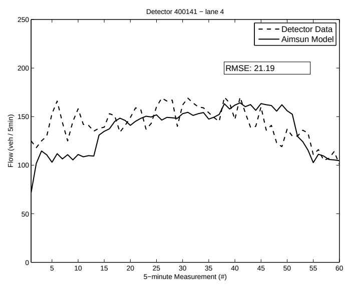
(e) Lane 1 - Flow



(f) Lane 2 - Flow

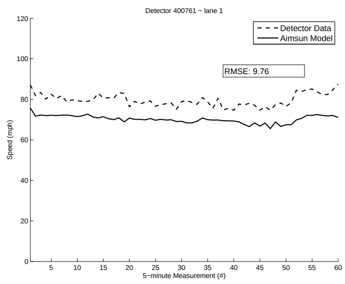


(g) Lane 3 - Flow

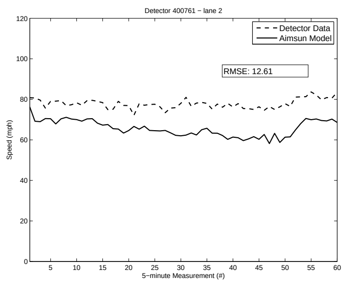


(h) Lane 4 - Flow

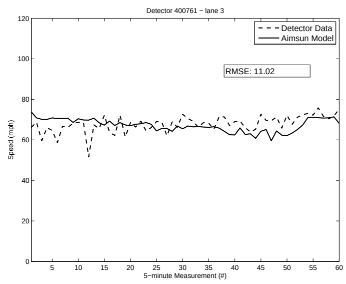
Figure D.9 *Calibration results of detector 400141 - post mile 16.60*



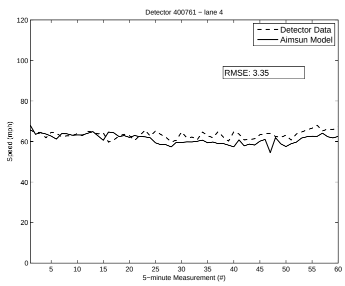
(a) Lane 1 - Speed



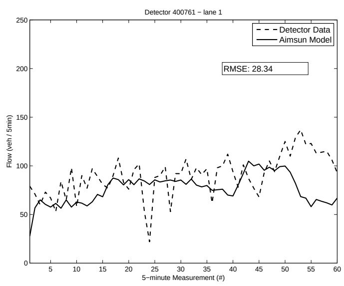
(b) Lane 2 - Speed



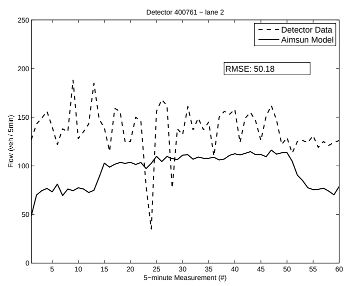
(c) Lane 3 - Speed



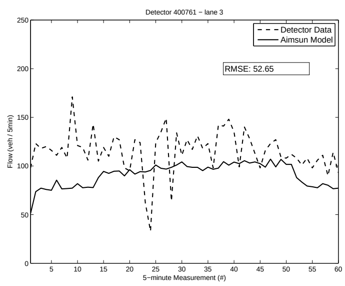
(d) Lane 4 - Speed



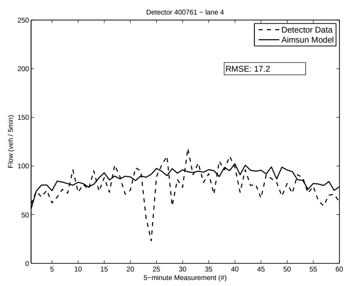
(e) Lane 1 - Flow



(f) Lane 2 - Flow

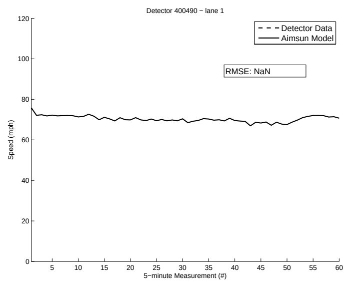


(g) Lane 3 - Flow

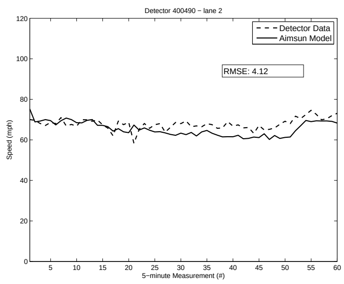


(h) Lane 4 - Flow

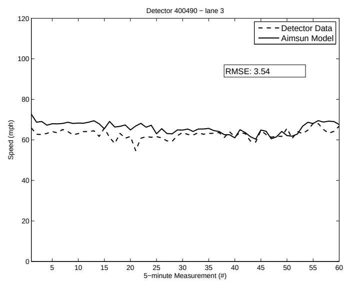
Figure D.10 *Calibration results of detector 400761 - post mile 17.36*



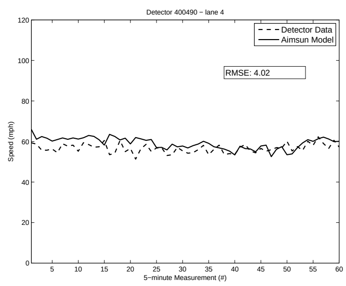
(a) Lane 1 - Speed



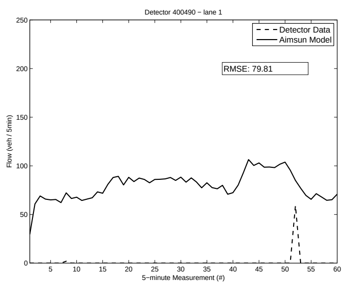
(b) Lane 2 - Speed



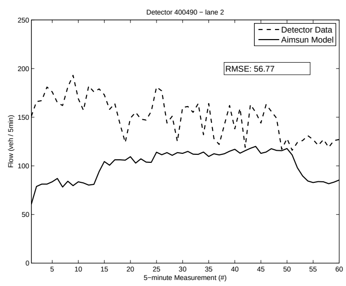
(c) Lane 3 - Speed



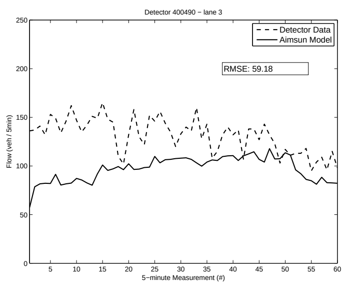
(d) Lane 4 - Speed



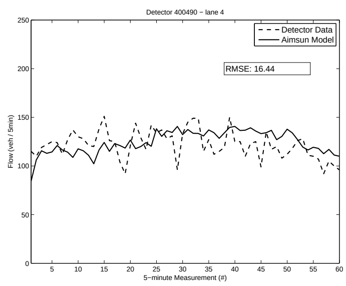
(e) Lane 1 - Flow



(f) Lane 2 - Flow



(g) Lane 3 - Flow



(h) Lane 4 - Flow

Figure D.11 *Calibration results of detector 400490 - post mile 17.59*

E Simulation Network

Overview of the simulation network in Aimsun 6.1. The colored boxes indicate detectors, green detectors are used for calibration (good data), red indicate unused detectors due to missing or no data. The detector numbers are also indicated. All detectors are assumed to be working in the Aimsun simulation environment. Here, the lanes are numbered from left to right where the HOV-lane is the most left (lane 1).

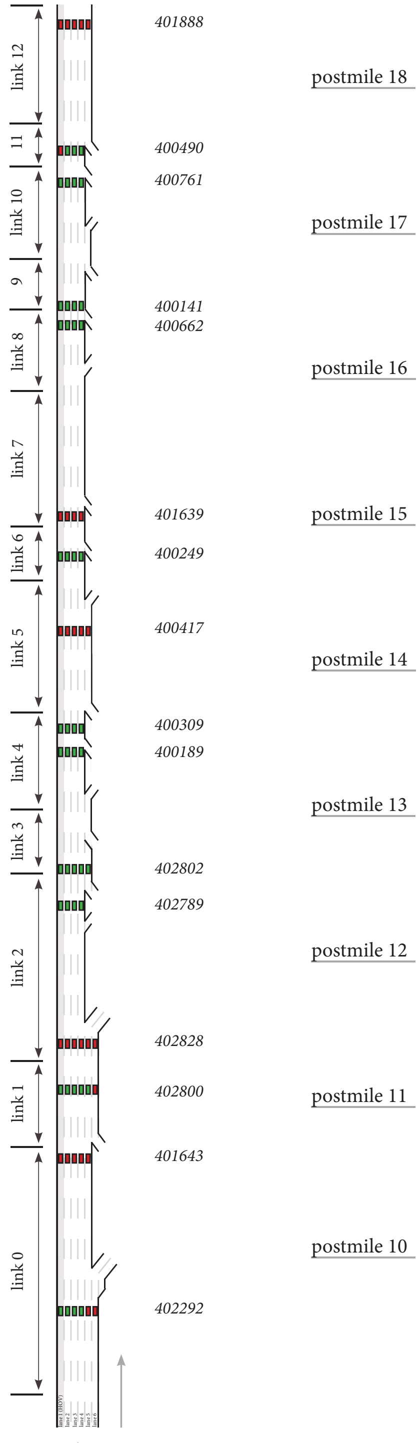


Figure E.1 *Schematic overview of the simulation network in Aimsun 6.1*