

Monitoring Travel Time Reliability on Freeways

Huizhao Tu

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Monitoring Travel Time Reliability on Freeways

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To my parents: YinLian & MuXi

Preface

In the years 2000~2003, I started out on a journey by getting involved in many transportation projects at the Tongji University under supervision of Professor Lijun Sun, who was the first to lead me into this field of science. By November 2003, I had become very enthusiastic about and interested in the topics of network reliability and data fusion. To me, it was clear that to write a thesis would be an appropriate next step in my development. I wanted to find out how the reliability model can be underpinned by using the available empirical traffic data in such a way that the model can be used in traffic assignment models and probably also in a real life traffic monitoring system. Back then, my ambition was compromised by the lack of a clear sense of direction. I set out to write a thesis without knowing what involves conducting scientific research and writing a PhD thesis.

After four years of hard work, the result now lies before you. I have reached my goal, which is an enormous relief. At the same time, I realize that especially the often difficult process of constructing this thesis now gives me the greatest satisfaction. I had to overcome a lot of obstacles and I succeeded. My work on this thesis gave me the opportunity to meet and cooperate with lots of different people. This personal contact and close cooperation with other people is what I have found most enriching.

I would like to use this opportunity to thank a great number of people for what they have meant to me in the course of writing this book. My promotor, Professor Henk van Zuylen gave me the feeling that the world was my oyster and lots of freedom to draw my own boundaries of my research project. He encouraged me to underpin my theoretical idea with empirical research. As well, I would like thank to my daily supervisor Hans van Lint. Both of you have spent the most time on reading my papers and chapters, for which I am grateful. Without their help and support I would never have gotten this far.

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And, what would life in a foreign country mean to me without my friends. Without you around me, Hao Liu, Xiao&Minxing, Xi&Wei, Bei&Jianxin, Guoping, Geping&Xueyuan, Yanxia&Xinyang, Minwei, Yi Zhou, Zhuoyu&Han, Hong, Hui&Zhuo, Fanzhong, and many more, life would not be worth a party.

I would like to thank my parents, brothers and sisters, sister-in-law, brother-in-law, and nephews who I missed so much during these eleven years (seven years in Shanghai). Thank you for the love and tolerance throughout the time I am absent. I can not find any pertinent words in English or Chinese, to express my gratitude towards them. Special thanks for my uncles, MuOu, MuXi, MuKui. Over the past decade, you always support and take care of my parents. Without your supports, I would not go abroad to pursue my career.

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Delft, April 2008

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Notations

The main symbols, variables, parameters and abbreviations that are used in this dissertation are presented as follows:

Statistical symbols and parameters

MRE	Mean Relative Error
$MARE$	Mean Absolute Relative Error
MSE	Mean Squared Error
$RMSE$	Root Mean Squared Error
$RMSEP$	Root Mean Squared Error Proportional
LSE	Least Squared Error Method

Symbols, variables and parameters

$q(x, t)$	lane and class specific mean flow rate (veh/h)
$v(x, t)$	lane and class specific mean speed (km/h)
$\rho(x, t)$	lane and class specific density (veh/km)
t_a	travel time on link a
t_a^f	free flow travel time on link a
v_a	traffic flow on link a
C_a	capacity on link a
q_{in}	inflow
p_k^{br}	probability of traffic breakdown on section k
p_r^{br}	probability of traffic breakdown on route r
TTV^f	travel time variability before breakdown
TTV^j	travel time variability after breakdown
$TT10th$	10th percentile travel time
$TT50th$	50th percentile travel time (median travel time)
$TT90th$	90th percentile travel time
λ_t	critical transition inflow
λ_c	critical capacity inflow
λ_{itr}	critical travel time unreliability inflow
L_d	length of deceleration lane
L_a	length of deceleration lane
L_w	length of weaving section
β	number of ramps per unit road length

General abbreviations

BR	Break Down
DTS	Degradable Transportation System
DTA	Dynamic Traffic Assignment
FD	Fundamental Diagram (of traffic flow)
SL	Speed Limits
TTV	Travel Time Variability
TTR	Travel Time Reliability
TTUR	Travel Time UnReliability
CTTR	Conceptual Travel Time Reliability Model
TLZ	<i>Tu, van Lint and van Zuylen Reliability Function</i>

Chapter 1

Introduction

1.1 Research Background

The economy of a nation or region depends heavily upon an efficient and reliable transportation system to provide accessibility and promote the safe and efficient movement of people and goods. In fact, the transportation system has been identified by (Nicholson & Du 1997) as the most important lifeline in the event of natural disasters such as earthquakes, floods, hurricanes, and others. Restoration of other lifelines (e.g. water supply, electrical power system, sewer system, communication, and many others) depends strongly on the ability to transport people and equipment to damaged sites. This is illustrated by the sudden snow and ice conditions in the East of The Netherlands on 25 November 2005. The snow and frost made the roads inaccessible. Trucks blocked the roads and could not be moved. Furthermore, the ice on the cables of the high voltage electricity system made them break so that electricity supply was interrupted. The restoration of the situation was very difficult because electricity generators could not be transported to the effected region, workmen could not go to the spots where the cables were damaged and towing vehicles could not drive to the spots where they were needed. A vulnerable transportation system would hinder the restoration process and increase not only economic loss but also fatalities. A reliable transportation system should also consider everyday disturbances. The actual travel demands and road capacity do vary over time, thereby contributing to the uncertainty of travel times. With the increased value of time, great loss is incurred by the drivers due to the unexpected schedule (either early or late) delay. A stable transportation system would provide a competitive edge in the global economy. Thus, the importance of the reliability of a transportation system can not be overemphasized.

Travel time reliability, a performance indicator of road networks, has significant effects on route choice, particularly for trips, such as journey-to-work, where time constraints (e.g. arrival time) may impose significant penalties on an individual (Abdel-Aty et al. 1996). Using Repeated Measurement Stated Preference Data, Abdel-Aty et al. (1996) indicated that travel time reliability is either the most or the second most important reason for choosing primary commute routes. Bates et al. (2001) also found that one minute reduction of standard deviation of travel time and two minutes of actual travel time are equally valued. While travel time reliability in the first place is a perception of travellers, it is also becoming more and more a measurable indicator of traffic and network performances.

Travellers and commercial vehicle operators, for instance, conceive travel time reliability as an attribute of (planned) trips and hence as integral part of the trip decision-making process in terms of, for example, route, departure time, and mode. Recent empirical studies (Small et al. (1999), Kazimi et al. (2000), König (2000), Lam (2000), Ghosh (2001), Bogers & van Zuylen (2004), van Lint & van Zuylen (2005a)) support this and suggest that travellers are interested in not just travel time saving but also in a reduction of travel time unreliability. For example, a decrease in travel time variability may lead to better fleet management and may significantly reduce scheduling mismatches and directly lead to time and cost savings (Small et al. 1999).

Travel time reliability as a performance indicator of mobility has also entered the political arena. A typical example is a recent announcement by the Beijing Municipality, which claims that 50% of the trips from hotels and to major sports of the Beijing Olympic Game 2008 should be made within 30 minutes (Beijing Municipality 2005). Another example, in 2004 the Dutch government presented a policy document to the parliament dealing with the national policy with respect to traffic, transport and infrastructure. The subtitle of the document is "*Towards a Reliable and Predictable Accessibility*" and shows the new emphasis that is placed on the concept of travel time reliability (VW 2005). In the document, the quantitative target for the transport policy with respect to reliability is that for trips longer than 50 km over freeways 95% of the trips arrive within the time interval of the median travel time plus or minus 20%. For shorter trips the target is that 95% of the trips will be between the median travel time plus and minus 10 minutes. Furthermore, the policy document has the target to have average travel times on freeways in the peak hours that are no more than 50% higher than in the off-peak. Given the already vast problems of traffic congestions due to the high demand and insufficient capacity on both cities, the goals in terms of reliability are ambitious (van Lint et al. 2005).

Over the past two decades a rich body of research has been developed, in terms of the definition of travel time reliability from different road participants (travellers, traffic managers and policy-makers) and the measures which can be defined to quantify the travel time reliability. This dissertation will continue addressing the definition and the measures of travel time reliability. Furthermore, knowledge about the causes of travel time reliability can be helpful to improve travel time reliability and to derive an explanatory model with which the travel time reliability can be predicted. Thus, this dissertation also tries to set up a travel time reliability model.

In this introductory chapter we describe the problem tackled and briefly define the area of research covered by this dissertation. Next, we present the main objectives of this dissertation and narrow down the scope of the research presented. Particularly, this dissertation concentrates on empirical analysis for investigating the causes of travel time reliability of freeway corridors. Subsequently, we review the main scientific and practical challenges and the contribution of this dissertation to the solution of these. The final part of this introduction then briefly outlines which subjects are covered in each chapter of this dissertation.

1.2 Problem Formulation

With the increasing importance of travel time reliability, many different definitions for travel time reliability emerged, and subsequently also different quantifiable measures for travel time reliability in a transportation network or corridor have been proposed (refer to, for example, Bell et al. (1999), van Lint et al. (2008)). What these measures have in common is that in general they all relate to properties of the (day-to-day) travel time distribution, and particularly to the shape of this distribution. There is, however, no undisputed opinion on what travel time reliability precisely entails or how it should be made operational and consistent. Before addressing this, one needs to know the elements of reliability. This puts forward the first research question in this dissertation:

Research question one: *What is the travel time reliability and which attributes can be assigned to this concept?*

Travel time reliability with the specified attributes should be measurable and quantifiable in practice. This motivates the second research question in this dissertation:

Research question two: *Which measures can be defined to quantify travel time reliability and how can these measures be monitored?*

Traffic assignment models have been used for many years as a traffic planning, designing and managing tool for transportation networks. Traffic assignment has four functions in traffic planning (Bliemer 2001):

- Gaining insight in the characteristics of the network
- Forecasting about future traffic conditions on transport networks
- Comparing scenarios of different infrastructure investments
- Estimating effects of traffic management measures

Route and departure time choices play a crucial role in a traffic assignment model and it is known that these choices do not only depend on expected travel time, but also on travel time reliability. Therefore, a good reliability model is needed to describe (link or route) travel utility in such a way that it can be used for traffic assignments (Tu, van Lint & van Zuylen 2007b). However, there is no empirically underpinned model which can be used in DTA. To this end, this dissertation puts forward an elaborate travel time reliability model that expresses this as a function of several factors and is able to predict travel time reliability. Consequently, it comes up with the third research question:

Research question three: *Which causes of travel time unreliability can be identified and how can these be used to derive descriptive models with which travel time unreliability can be predicted.*

1.3 Research Objectives and Scope

1.3.1 Research objectives

This thesis aims at proposing an appropriate definition of travel time reliability and a good travel time reliability model that can be used in traffic assignments. The following objectives have been pursued in this study:

- To investigate the elements inside the travel time reliability and propose a definition of travel time reliability that can capture these elements;
- To gain insight into the potential causes (factors) which influence travel time reliability;
- To come up with a conceptual travel time reliability that is logically related to the characteristics of traffic flows;
- To derive a mathematical formula that can quantify the conceptual travel time reliability;
- To develop a travel time reliability function (model) with the mathematical formula that is suitable for Dynamic Traffic Assignment;
- To test whether the model is generic to monitor the travel time reliability under different circumstance (e.g. weather condition, traffic control, etc.).

1.3.2 Research scope

We will limit the scope of our research efforts. First of all, this dissertation will address travel time reliability on uninterrupted roadway facilities such as *freeways*. We adhere to the definition for freeways given in the Highway Capacity Manual 2000: "*A freeway is a multilane, divided highway with a minimum of two lanes for the exclusive use of (motorized) traffic in each direction and full control of access without traffic interruption*"(TRB 2000). In the Netherlands, 6.2 billion kilometers, one-third of the total traveled kilometers, were on freeways in 2006 (Rijkswaterstaat 2007). This is a 1.9% increase compared to the year of 2005. Given the important role of freeways in road networks, the first step of this dissertation focuses on freeways.

Secondly, this dissertation tries to setup an explanatory reliability model which expresses this as a function of several factors. Therefore, the traffic data like flows, travel times, road geometry etc should be obtained. Here some traffic data collection system is installed. Historically, most deployed traffic data collection systems in practical consist of local detection equipment (e.g. inductive loops, pneumatic tubes), resulting in local aggregate characteristics (e.g. flows, local mean speeds) of traffic streams. Since these systems do not measure travel times directly, a so-called offline travel time estimation algorithm is used to enable translation of for example local speeds into (route) travel times. This will be explained in more detail in Appendix B.

Thirdly, different vehicle classes, such as trucks and passenger cars, will have a different value of time and reliability, and the mix of these varies over the day. However, the description of this research is limited to motorized vehicles while no attention is given to different vehicle classes and to the effect of one specific class on another. This is partly because of the lack of available data, but also because the traffic mix is to a certain extent constant variable for a specific roadway.

Finally, traffic flow used in this research exclusively denotes vehicles entering the studied freeway stretch at the upstream entry of the main carriageway, which does not include the flow of on- or off-ramps along the roadway section. This obvious limitation is due to first of all the fact that no data were available for this study from loops located on these ramps. Another reason to ignore these parameters is that in general in and out flow are strongly correlated with the flow entering at the beginning of the corridor (see an example in Figure 1.1). In the remainder, hence, traffic (in)flow refers to the flow at the upstream boundary of the considered road stretches. In this context, traffic (in)flow could be an indicator of traffic demand for the beginning of the corridor.

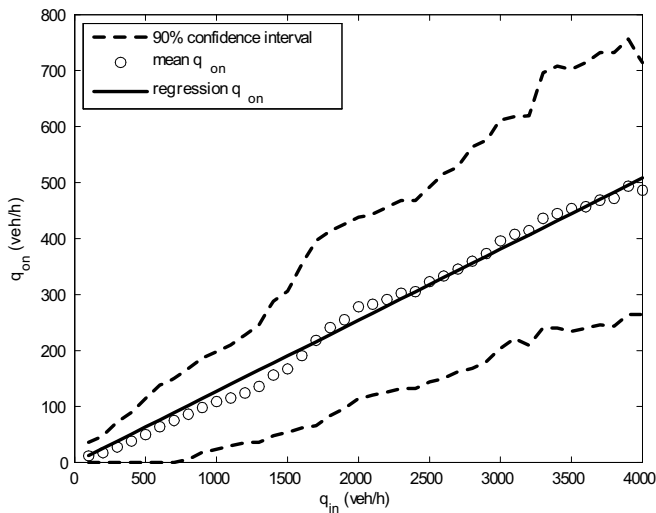


Figure 1.1: Empirical relation between inflow and on-ramp flows on A12 freeways in 2004

1.4 Contributions and Scientific Relevance

1.4.1 Summary of contributions

This thesis contributes to the State-of-the-Art of travel time reliability on freeways in various ways:

1. It gives a new definition of travel time reliability which include both the variability (uncertainty) in travel times and the stability of travel times (Chapter 3).
2. It develops an analytical formulation to set up the relationship of the probability of traffic breakdown on section level and route level (Chapter 3).
3. It develops an (*static*) inflow-travel time variability function (Chapter 4).
4. It proposes an inflow-travel time reliability function (so-called TLZ (Tu, van Lint, van Zuylen) function) (Chapter 4).
5. It expresses travel time reliability as a function of several factors such as inflow, road geometry, adverse weather conditions, Speed Limits, traffic accidents (Chapter 4 and Chapter 5).
6. It provides an analytical formulation to setup the relationship on the probability of traffic breakdown among different measurement intervals (Chapter 4).
7. It proposes a framework to setup the travel time reliability model. Accordingly, a data collection procedure for travel time reliability modeling is established and the measurement time interval is proposed for the reliable empirical analysis (Chapter 4).

1.4.2 Theoretical and scientific relevance

In this dissertation a number of issues are discussed which in our view are theoretically and scientifically relevant:

1. The definition of travel time reliability proposed in this dissertation is a dynamic (or consistent) process, which takes both the variability in travel times and the stability of travel times into account. The benefit of such an approach is that it can give a consistent and intuitive reliability definition.
2. The developed analytical formulation on the relationship between the probabilities of traffic breakdown on various sections along a route can account for the interactions between traffic streams among route sections.
3. The developed travel time reliability model is expressed as a function of several factors such as inflow, road geometry, adverse weather, traffic control, and traffic accidents. The benefits of such an approach are threefold, namely it provides insight and enable analyses in the following ways:
 - The developed reliability gives insights into the impact of road geometry (for instance, weaving length, the average distance between off/on ramps etc.) on travel time reliability.
 - It also gives insights into the effects of other factors (e.g. adverse weather, traffic control measures) on travel time reliability.
 - It provides a way to analyze reliability in dynamic traffic assignment models.

1.4.3 Practical relevance

Since travel time reliability is one of the most important factors for travelers while making their route choices and departure time choices, there is a very clear need for building up the travel time reliability model in practice such that the model can be used in traffic assignments in which the route and departure time choice depends on the reliability of the travel time on the various routes over time.

It is clear that freeway characteristics have an impact on travel time reliability. Based on empirical data analysis, the relationship between travel time reliability and freeway characteristics (i.e. length of deceleration lane, length of acceleration lane, weaving length, the number of ramps per unit road length) indicates that, at least in terms of practical design guidelines, there exist certain threshold values L for the length of ramp sections and weaving sections and travel time reliability is strongly affected by the number of ramps per unit of road length. For instance, for an unreliable freeway corridor, traffic managers can choose to increase the distance between ramps by closing one of the on/off ramps and can extend a short weaving section to improve reliability.

Travel time varies with different external circumstances (e.g. weather conditions, (dynamic) traffic control measures). A precise estimation of the variety of travel time reliability due to the adverse weather can be useful in developing a management strategy for freeway systems using control, advisory, and road treatment strategies.

Speed Limits, one of the traffic control measures, reduce the travel time uncertainty, but increase the probability of traffic breakdown. Therefore, the influences of Speed Limits on travel time reliability depend on the value taken in Speed Limits. In practice, the implementation of Speed Limits should take the effects on travel time reliability into account.

The effects of traffic accidents on travel time reliability may be expected to be important, however the analysis shows otherwise. It is true that traffic accidents result in extreme long travel times, yet the (day-to-day) recurrent-congestion makes the largest contribution to unreliable travel times. This may imply that reducing traffic accidents may not improve travel time reliability significantly, at least in the case of the Dutch freeways.

1.5 Thesis Outline

This section briefly describes the contents of each chapter of this dissertation and the connection between them.

Chapter 2 presents the fundamental notions of travel time reliability. It discusses the different definitions of travel time reliability, presents the proposed travel time reliability measure and outlines the current State-of-the-Art in the analysis on travel time reliability. Thereafter, it explores the freeway travel time reliability modeling problem and introduces the proposed approach to set up a travel time reliability model.

Chapter 3 discusses the main empirical characteristics of both the speed-flow and travel time-flow relations. It comes up with two main components in the definition of travel

time reliability: variability in travel times and stability of travel times from which the conceptual travel time reliability model is derived.

Chapter 4 expresses travel time reliability as a function of the principal input factor: inflow. To investigate the model, it sets up an empirical framework which is applied and tested in the Regiolab-Delft test-bed (Appendix A (van Zuylen & Muller 2002)). Thereafter, the inflow-travel time reliability has been calibrated and validated.

Chapter 5 outlines how the proposed travel time reliability model could be extended. Amongst other things, the extended model has been investigated by taking road geometry, adverse weather conditions, Speed Limits, and traffic accidents as the input factors (parameters).

Chapter 6 illustrates the travel time reliability model applied in traffic assignment models.

Chapter 7 concludes the thesis and gives some research directions for the future.

Figure 1.2 shows the structure of the main body of this thesis.

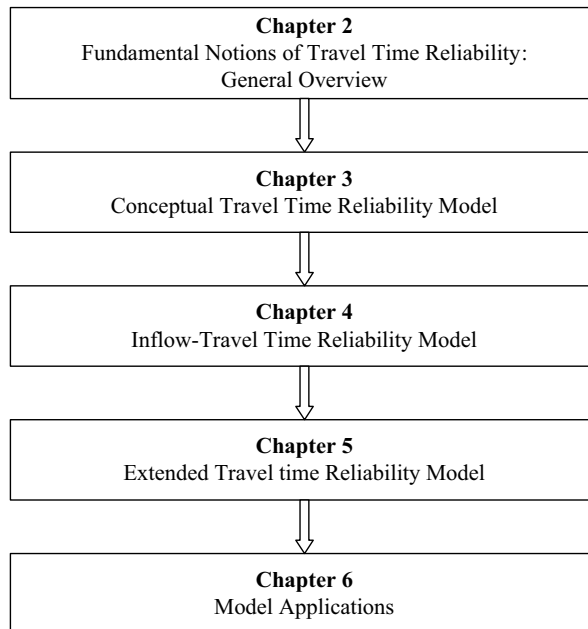


Figure 1.2: Schematic overview of the main body of this dissertation thesis

Chapter 2

Fundamental Notions of Travel Time Reliability: General Overview

2.1 Introduction

The sustained growth of the economy and the continued improvements to the quality of life lead to an increase in the value of time, especially in the developed countries. Travel time reliability has thereby become an increasingly important attribute for assessing the performance of road networks (Bell et al. 1999). Consequently, many efforts have been undertaken to define and measure travel time reliability. This dissertation will provide a new and empirically underpinned definition of travel time reliability and will derive a mathematical model which operationalizes and quantifies this definition. In turn, with this mathematical model transportation performance in terms of travel time reliability can be assessed both ex-ante (e.g. with traffic assignment models), ex-post (on the basis of archived data) as well as in real time (reliability monitoring). Before addressing these issues in later chapters, this chapter will first discuss some reliability related notions and outline and discuss existing reliability measures. Next the new travel time reliability measure is proposed and discussed in section 2.2. Thereafter, in section 2.3 we identify factors influencing travel time (un)reliability or more precisely the traffic conditions that influence travel time reliability. Furthermore, in section 2.4 we give an overview of the complexity of travel time reliability problem and outline the approaches to analyze travel time reliability. Finally, we summarize the main conclusions in section 2.5.

2.2 Definitions of Travel Time Reliability

This following sections present clear definitions of travel time, reliability, and travel time reliability, respectively.

2.2.1 Travel time

Before addressing the term travel time reliability, this section briefly addresses some definitions of travel times. Figure 2.1 shows some often used classifications of travel times,

for more detailed descriptions see (Thijs 2000) and (van Lint 2004).

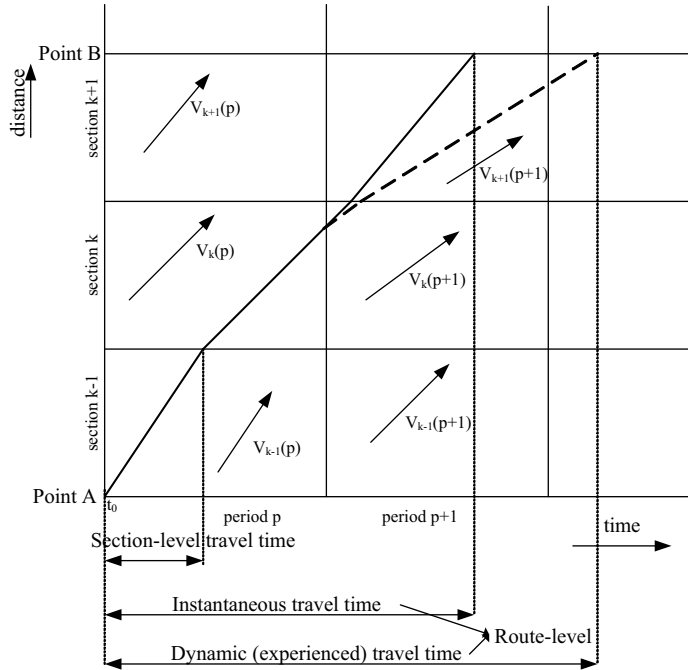


Figure 2.1: Examples of travel times. Sources (Thijs 2000)

Depending on the spaces vehicles experienced, travel times are classified into:

- Section-level travel time
- Route-level travel time

Depending on the travel time vehicles could experience, travel times are categorized into:

- Instantaneous travel time *The instantaneous travel time is the travel time a vehicle would experience on a particular section k or route r departing at time instant t_0 if the traffic conditions on k or r remain stationary for time instant ($t \geq t_0$). Instantaneous travel time hence reflects an approximation of the actual travel time which particularly in congested traffic conditions may deviate substantially from the actually experienced travel time.*
- Dynamic (experienced or realized) travel time *This is the travel time a vehicle actually experiences on a particular section k or route r departing in at time instant t_0 given the (possibly) non-stationary traffic conditions on k or r for time instant ($t \geq t_0$). Figure 2.1 illustrates the differences between instantaneous and dynamic travel time.*

Travel times may furthermore be classified as individual and mean (average) travel time according to the level of detail with which the number of vehicles they account for

- **Individual travel time** *The individual travel time on a section k or a route r for a vehicle departing at time instant t_0 is the time it takes for that individual traveller to traverse that particular section or route.*
- **Mean travel time** *The mean travel time on a section k or a route r for vehicles departing at time instant t_0 is the average time it takes these vehicles to traverse the specific section or route.*

In this dissertation, the term travel time will denote *route-level dynamic mean* travel time, unless specifically stated otherwise.

2.2.2 Reliability

The main concept of this dissertation is *reliability*. In the preceding sections the concept of *reliability* has been used without a precise definition. It is, however, very important that all main concepts are defined in an unambiguous way. A precise definition of reliability and some associated concepts like variability, robustness, and vulnerability are given below.

- **Variability** relates to the degree of variation in the outcome (e.g. travel time) of a certain process (e.g. traffic), or put differently to how "spread out" a (statistical) distribution is. Variability does not necessarily imply unpredictability, but it does reflect the uncertainty around a prediction.
- **Reliability** Reliability has been studied within many fields, dealing with, for instance, electronic systems, power systems, transportation systems etc. The type of reliability analysis done in, say, an electronic system differs from that done in an transportation system. The former tends to emphasize the physics of failure and tends to downplay the mathematics of uncertainty; the latter tends to do the opposite (Singpurwalla 2006). In a transportation field, reliability is defined as the probability that a road network can perform a required function under given environmental and operational conditions and for a stated period of time (Iida & Wakabayashi 1989). A required function may be a single function or a combination of functions that is necessary to provide a specified service.
- **Robustness** In the field of transportation, robustness is defined as the insusceptibility of a road network to disturbing incidents, opposed to network vulnerability. In other words, a robust road network is able to operate within the design specifications also for conditions that are outside the range of the design specifications. Furthermore, the performance of a robust road network deteriorates "gratefully" under increasingly adverse circumstances.
- **Vulnerability** can be defined as the susceptibility to incidents that result in performance deterioration. Vulnerability increases as the probability and/or consequence of failing to meet user expectations increases (Nicholson & Du (1994) and Berdica

(2000)). In this context, vulnerability is essentially the opposite of robustness. On the other hand, D'Este & Taylor (2001) define vulnerability as the likelihood of severe adverse consequences if a small number of links (or possibly a single link) is degraded. They distinguish between 'connective vulnerability' and 'access vulnerability'. The former considers a pair of nodes and the generalized cost of travel between them; if the loss or substantial degradation of one or more network links leads to a substantial increase in that cost, then the connection between those nodes is deemed vulnerable. Access vulnerability considers a single node and the overall quality of access from that node to all other parts of the network; a node is vulnerable if the loss or substantial degradation of a small number of links results in a significant reduction in the accessibility of that node.

In short, robustness or vulnerability is a characteristic of the system itself. Reliability or variability, on the other hand, provides a measure of the stability of the quality of service, which the transport system offers to its users (Immers & Jansen 2005). Since this dissertation addresses reliability, it hence focuses on the measures of road network performance rather than on inherent transportation system characteristics. A more detailed elaboration on the differences between reliability and variability will be given in section 2.2.4.

2.2.3 Overview measures for reliability

Prior researches on transport network reliability focuses on four main aspects: connectivity reliability, capacity reliability, encountered reliability, and travel time reliability. Here we summarize these reliability indicators in Table 2.1 and briefly describe them below.

Table 2.1: Principal Characteristics of Definitions of Road Network Reliability (Modified from Chen and Recker, 2001)

Reliability Index	Definition	Sources of Unreliability	Performance Indicator
Connectivity	Connected and disconnected network	Disruption of road links	1 if connected and 0 if disconnected
Network Capacity	Network reserve capacity greater than a specified value	Degradable link capacity	Required demand level
Encountered Reliability	Not encountering a link degradation	Disruption or degradation of road links	Least costs
Travel Time (threshold based)	Travel time less than a specified value	Fluctuation of daily flow	Specified travel time
Travel Time (level of service based)	Service level less than a specified value	Degradable link capacity	Specified network service

- *Connectivity reliability*, which is concerned with the probability that the network nodes can be reached. A special case of the connectivity reliability is the terminal reliability, which relates to the existence of a path between a specific origin-destination (O-D) pair (Iida & Wakabayashi 1989). A path consists of a set of roadways or links which are characterized by a binary variable denoting the state of each link (operating or failing). Capacity constraints on the links are not accounted for when determining connectivity reliability. This type of reliability analysis might be suitable for abnormal situations, such as earthquakes, but there is an inherent deficiency in the sense that it only allows for two operating states: operating at full capacity or complete failure with zero capacity. This binary state approach prevents application to everyday situations where links are operating in between these two extremes. Furthermore, it is only applicable to special network structures without redundancy in connections.
- *Capacity reliability*, which is defined as the probability that the network capacity can successfully accommodate a certain level of O-D demand at an acceptable service quality (Chen et al. 2002). Link capacities in a road network are considered as random variables and can change from time to time due to e.g. the blockage of one or more lanes due to traffic accidents. The joint distribution of random link capacities can be experimentally obtained or theoretically specified. Capacity reliability explicitly considers the uncertainties associated with link capacities by treating roadway capacities as continuous quantities subject to routine degradation due to physical and operational factors. The capacity reliability of the network should be considered an important and meaningful measure of overall system performance that is of interest to system managers.
- *Encountered reliability* is concerned with the probability that a trip can be made successfully without encountering link degradation on the least (expected) cost path (Bell & Schmöcker 2002). In assessing encountered reliability, the level of information to the users is important since users will often try to avoid degraded links or links which may be degraded. In addition, different users may behave differently. Risk averse users are inclined to avoid a disruption while they are willing to travel longer, while risk neutral users will still travel on their preferred routes based on expected cost considerations regardless of the probability of encountering disruptions along their preferred routes (Chen & Recker 2001).
- *Travel time reliability* is the subject of this dissertation and will be defined and discussed more elaborately in the ensuing sections.

2.2.4 Measures for travel time reliability

As discussed in section 1, the reliability of travel time is an important attribute of a trip. If a trip is unreliable, the travelers will experience unexpected travel conditions and they will be unsatisfied about that. If, on the contrary, trip conditions are predictable and travel times are reliable, the reaction of travelers is not so much that they are satisfied by this characteristic of the trip, but the best that can be achieved is that they are not unsatisfied. Of course, reliability is a relative concept. What we consider as unreliable in our days might have been very reliable in the time of our grand parents. Furthermore, as Bonsall

(2000) noted, "*reliability implies a degree of predictability but not necessarily complete invariance or stability*". The reliability of a trip depends on the variability as well as the predictability of the travel time. A trip that shows little variation in travel times is quite reliable. A large variability in travel times and the expectation of travelers are not necessarily a problem, as long as the actual travel time can be well predicted and the prediction is made before certain decisions have to be made. The reliability is poor when the variability is large and the predictability is small. This results in a major problem in assessing travel time reliability. The question is: what is predictable? It is a common concept that travel times are longer on a Monday morning at 8:00AM in comparison to the next morning at 11:00AM. However, the extent of this difference will only be known to those making the trip regularly at different times. Traffic gridlocks due to road accidents will be a surprise for all travelers, yet local jams due to events that attract a lot of visitors or road works will be known to some but not to all travelers. In short, travel time reliability comprises both predictability and invariance. For the predictability aspect of travel time reliability, uncertainty or instability implies a degree of unpredictability. In this dissertation, we propose instability is related to unpredictability. Therefore, the requirements in terms of reliable travel times are listed as follows:

- Low variability in travel times
- High stability in traffic flows

In this context, travel time reliability is a function of both variability in travel times and instability of traffic flow:

$$TTR = \{TTV, Instability\} \quad (2.1)$$

where TTR denotes travel time reliability; TTV denotes travel time variability.

Many different definitions for travel time reliability exist, and subsequently also different quantifiable measures for travel time reliability in a transportation network or corridor have been proposed (refer to, for example, (Bell et al. (1999), van Lint et al. (2008))). What these measures have in common is that in general they all relate to properties of the (day-to-day) travel time distribution, and particularly to the shape of this distribution and to the variability in travel times. Two distinctive approaches exist for modeling these effects of travel time variability on travelers' behavior in a way that can lead to economic analysis (Hollander 2006). The first approach claims that travelers see travel time variability per se as a direct source of inconvenience, similarly to the way they treat the mean travel time, often using a variable that stands for the standard variation of the travel time. The alternative approach argues that the entire cost attribute to travel time variability can be captured indirectly, by modeling travelers' earliness and lateness considerations when choosing at what time to depart for their journey. This is based on the idea that the main means travelers have of coping with unpredictable travel times is moving their departure time from home backward or forward. By doing this they trade-off between their chances of arriving too early or too late to the destination; the indirect approach to modeling the effects of travel time variability on travelers claims that this trade-off fully explains the disutility associated with travel time variability. In literature, the indirect approach is

commonly referred to as the *scheduling approach* (e.g. Small et al. (1999), Bates et al. (2001), Noland & Polak (2002)). Small et al. (1999) noted that “in models with a fully specified set of scheduling costs, it is unnecessary to add an additional cost for unreliability.” Bates et al. (2001) reject this argument, and state that there is an additional cost attributed to travel time variability itself. Hollander (2006) reported that monetizing the effects of travel time variability should use both direct and indirect approaches. Since the schedule day approach is more like the way how travelers react on travel time variability, it is necessary to take both variability and schedule delay into account in the choice behavior. Therefore, we only review the measures for travel time variability (unreliability) in this section. A comprehensive overview of travel time reliability measures can be found in (Lomax et al. 2003) and (van Lint et al. 2008), a brief overview is as follows:

1. *Statistical range methods*, which reflect measures related to the shape of the travel time distribution. These usually appear as variability measures (Bates et al. 2001) (Lomax et al. 2003). Examples are Travel Time Window (e.g. average travel time \pm standard deviation. This “plus or minus” type expression indicates the possible spread of travel time around some expected value, while implicitly assuming travel times to be symmetrically (e.g. normally) distributed). In case of normally distributed travel times, using a window of one standard deviation will encompass 68% of the days, peak periods or whatever time period is chosen for analysis.), the Percent Variation (i.e. the ratio of standard deviation and travel time), the Variability Index (the ratio between 95% travel time during peak hours and 95% during off peak), or measures for the skewness of the travel time distribution, which encompasses information regarding predictable functions like day-to-day fluctuations. In the 1998 California Transportation Plan (Booz-Allen & Hamilton 1998), travel time reliability is defined as the level of difference between the expected travel time and the actual travel time experienced. In that definition, the expected travel time is based on scheduled travel time or mean travel times experienced, while the actual travel time incorporates the effects of non-recurrent congestion.
2. *Buffer time methods*, which consider the extra time a traveller has to depart earlier to have less than X% chance to miss an appointment (buffer time index), or which gives the minutes of extra time needed to guarantee a statistically minimum number of arrivals within the preferred arrival time at destination (buffer time) (Pearce 2001) (Lomax et al. 2003). Usually buffer time is expressed as the ratio of the difference between 90th or 95th percentile travel time and the mean travel time over the mean travel time, as shown in Eq.2.2.

$$BI_{90} = \frac{TT_{90th} - M}{M} \quad (2.2)$$

where BI_{90} is buffer time index while TT_{90th} is the 90th percentile travel time and M is the mean travel time.

3. *So-called "tardy-trip" measures*, such as the 'misery index', which is the difference between the average travel time of the 20% worst trips with the overall travel time average (Pearce 2001) (Lomax et al. 2003). Another example of a measure in this class is the On-Time Arrival, which indicates the percentage of trip times that are

within an arrival time window, as shown in Eq.2.3. The arrival time window is defined by the travelers' characteristics (e.g. importance of the activity at destination) and the expected duration of the trip.

$$MI = \frac{M_{(TT_i > TT80th)} - M}{M} \quad (2.3)$$

where MI is the misery index. $TT80th$ is the 80th percentile travel time while TT_i is a travel time observation and M is the mean travel time.

4. *Probabilistic measures*, which consider the probability that a trip will be made within the nominal travel time multiplied by a factor, set by the analyst (Asakura & Kashiwadani (1991), Bell et al. (1999), Yang et al. (2000), Tu et al. (2005), Tu, van Lint & van Zuylen (2006a), Tu, van Zuylen & van Lint (2006)). Some probabilistic measures have in common that they are used as measure for travel time unreliability. Probabilistic measures are hence parameterized, in the sense that they use either a threshold travel time or a predefined time window to differentiate between reliable or unreliable travel times. This implies that their usefulness greatly depends on properly setting these parameters, which of course is application and context specific. The Florida Department of Transportation, for example, defined travel time reliability on a highway segment as the percentage of trips that takes no longer than the expected travel time plus a certain acceptable additional time (Shaw 2000) (Florida-DOT 2000), as shown in Eq.2.4:

$$PR(\alpha) = P(T_i \leq \alpha \cdot TT50th) \quad (2.4)$$

where PR is the probability of reliability while parameter α may be chosen at the discretion of the analyst and $TT50th$ is the median travel time. Tu, van Lint & van Zuylen (2006a) and Tu, van Zuylen & van Lint (2006) defined travel time reliability as a function of departure time (e.g. time-of-day, day-of-week):

$$PR(\alpha) = P(T_i \leq \alpha \cdot T_r^* |_{TOD, DOW}) \quad (2.5)$$

where $\alpha \cdot T_r^*$ is the threshold travel time with $\alpha \geq 1$ and T_r^* equal to for example the free flow travel time. TOD stands for "Time-of-Day"; DOW is short for "Day-of-Week". In this sense, probabilistic measures focus more on whether travel times meet the expected travel time of travellers; statistical measures, on the other hand, consider the travel time distribution

5. *So-called "skew-width" measures*, in which skewness of travel times λ^{skew} is defined as the ratio of the distance between the $TT90th$ and $TT50th$ percentile travel time and the distance between the $TT50th$ and $TT10th$ percentile travel time (Eq.2.6) while width of travel times λ^{var} is defined as the distance between $TT90th$ and $TT50th$ percentile travel time relative to the median travel time $TT50th$ (Eq.2.7) (van Lint et al. 2004) and (van Lint & van Zuylen 2005b). In general, a larger λ^{skew} means a higher probability for extreme travel times (relative to the median) to occur. Large λ^{var} values indicate the width of the travel time distribution is large relative to its median value. Furthermore, van Lint & van Zuylen (2005b) proposed the following indicator for travel time unreliability (Eq.2.8) that can be derived based on the percentile-based definitions of width and skew (Eq.2.6 and Eq.2.7). L_r is

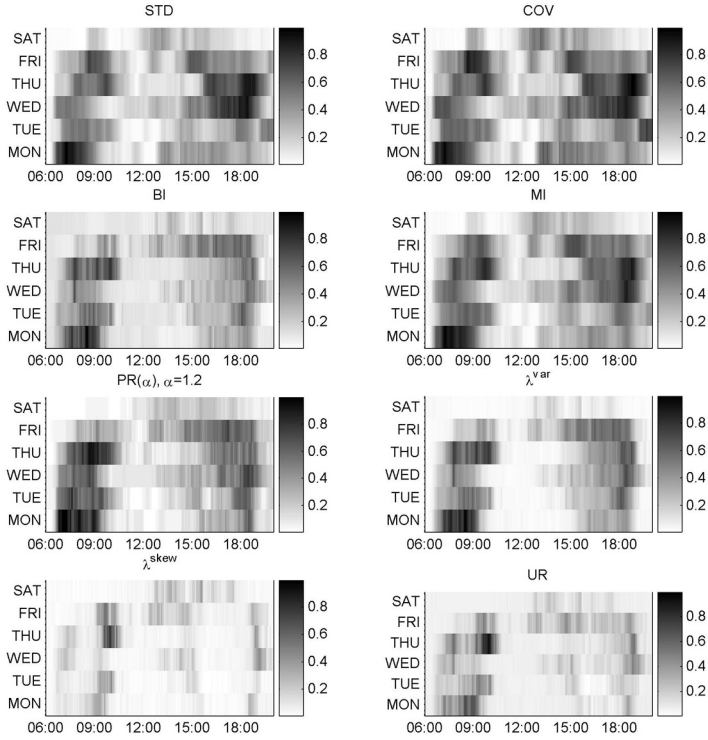


Figure 2.2: Reliability maps on the basis of 8 different travel time reliability measures. In all graphs dark areas depict unreliable travel times. Note that all measures have been scaled to the same interval (0,1). STD: standard deviation; COV: coefficient of variation (standard deviation over the mean value); BI: buffer time index; MI: misery index; UI: unreliable index. *Source: (van Lint et al. 2008)*

the route length. This means that it is large for unreliable periods.

$$\lambda^{skew} = \frac{TT90th - TT50th}{TT50th - TT10th} \quad (2.6)$$

$$\lambda^{var} = \frac{TT90th - TT10th}{TT50th} \quad (2.7)$$

$$UI_r = \begin{cases} \frac{\lambda^{var} \ln(\lambda^{skew})}{L_r} & \lambda^{skew} > 1 \\ \frac{\lambda^{var}}{L_r} & otherwise \end{cases} \quad (2.8)$$

van Lint et al. (2008) compare these five different measures by looking at empirical data from a much used freeway in The Netherlands. Figure 2.2 shows eight reliability maps

for the A20 freeway in The Netherlands. In all graphs dark areas depict values close to 1 and hence unreliable travel times. It is found that there are large differences between the measures. According to for example COV (coefficient of variation, Figure 2.2 top right graph), travel times are considered unreliable on 41% of all TOD(time of day)/DOW(day of week) periods between 6:00AM and 8:00PM, and on 58% of all TOD periods on Wednesdays, based on a threshold of 0.25. In contrast, according to the UI_r measure not more than 12% of the TOD periods on all days and 17% of the TOD periods on Wednesdays are unreliable (also at a threshold of 0.25). COV hence assigns 4 to 5 times more TOD periods unreliable than UI_r , and still over 1.5 times more often than for example BI. Different measures therefore give very different answers. van Lint et al. (2008) concluded that none of the measures presented here provides undisputable arguments in favor of either reliable or unreliable travel times in a particular time period.

2.2.5 Discussions

Above it is argued that reliable travel times are characterized by two important aspects: low variability and high stability in travel times. However, the currently available travel time reliability measures focus solely on the first of these aspects, that is, the variability (or uncertainty) of travel times. In the next chapter we will develop a travel time unreliability measure which incorporates both variability and instability. To this end, the next section will discuss some of the factors which influence both travel time variability and instability, and hence reliability.

2.3 Factors Influencing Travel Time Reliability

Since travel times are the result of traffic flow operations, which in turn are governed by the interplay between traffic demand (the amount of travelers entering a network) and traffic supply characteristics (e.g. the available capacity on the infrastructure) the distribution of travel times is a result of fluctuations in both traffic demand and supply characteristics, which is schematically outlined in Figure 2.3.

For readability purposes, not all (inter)relationships between these demand and supply factors have been drawn in Figure 2.3. Although each class of factors is discussed separately, this does not imply that they are independent. On the contrary, most of these factors strongly overlap and depend (non-linearly and dynamically) upon each other. For example, adverse weather conditions may (locally) reduce the capacity in a traffic network, but may at the same time yield (global) changes in traffic demand, due to people changing routes, departure time, mode or even reconsidering taking a trip altogether. Similarly, particular traffic management measures (peak-hour shoulder lane usage for example) may increase capacity (locally) but at the same time induce demand (globally and locally). Travel time reliability hence results from the interaction of many different factors - or sources of unreliability. Nonetheless, we subdivide the factors influencing travel time reliability into two groups: factors causing travel time unreliability due to demand variations and factors causing travel time unreliability due to capacity (supply features) variations. The following sections describe qualitatively how these factors affect the reliability of travel times.

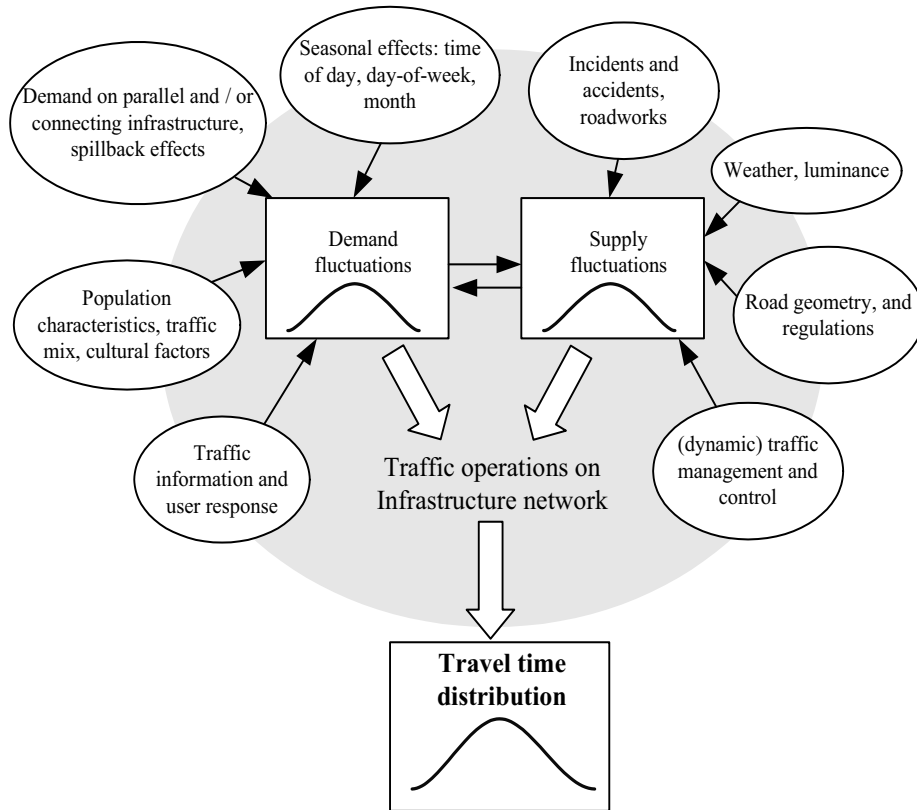


Figure 2.3: Schematic overview (not exhaustive!) of factors influencing the distribution of travel times

2.3.1 Variations in traffic demand

The main group of factors that fall into this category is what we will refer to as *Temporal effects*. It is well known that during holiday periods roads are less busy. Figure 2.4 illustrates for example, that in July and August present less congestion occurs than in October in The Netherlands. Similarly some days of the week are more congested than others (e.g. Tuesday morning peaks are rather busy, while Friday morning attracts much less traffic (van Eck 2004)). All of these seasonal effects result in a certain bandwidth of travel demand during a typical peak hour (for example, morning peak hours between 6:30Am and 9:30Am; afternoon peak hours between 4:00PM and 7:00PM) or a typical off-peak hour. In the Dutch case, on an average working day 5.5 million trips are made by car during the peak hours (van Eck 2004). On the 5% quietest working days this number is less than 4 million, while the 5% busiest working days have to cope with more than 6.9 million trips: a bandwidth in demand of more that 25% on both sides of the median. In the off-peak hours an average working day results in 10.5 million trips. On the 5% quietest days this is less than 7.7 million and on the busiest days this is more than 13 million: a similar bandwidth.

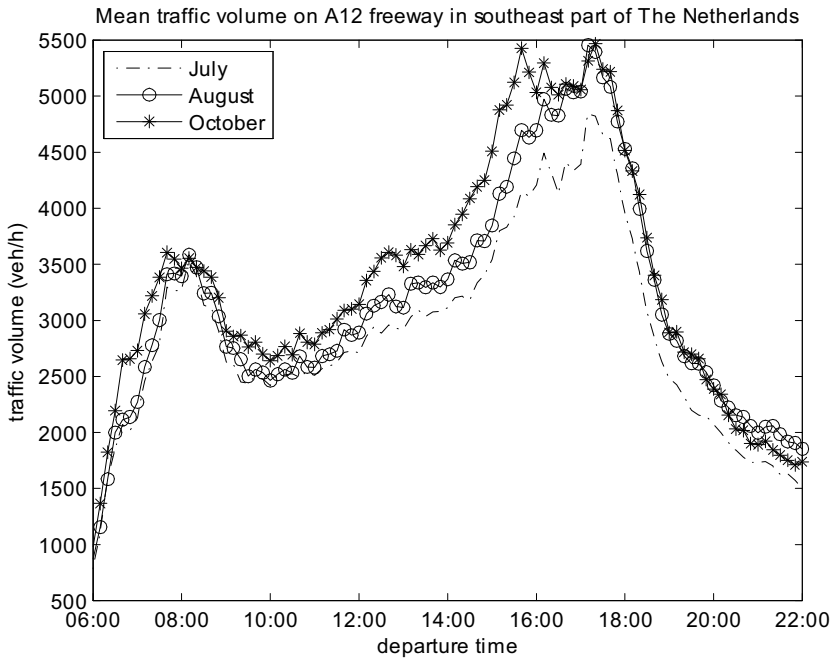


Figure 2.4: Example of traffic volume as a function of departure time under different month of year on A12 freeway in the southwest part of The Netherlands

Figure 2.5 shows clearly identifiable traffic flow patterns as a function of departure time on working days and weekend days on a freeway stretch in The Netherlands. It is found that there is a morning and afternoon peak during working days, while no clear peak periods can be distinguished during weekend days. Strong differences in flow rates are measured between working and weekend days because of the concentration of commuters during working days. On the other hand, in terms of variance (inter-percentile range: 90th percentile traffic flow minus 10th percentile traffic flow) the traffic flow patterns as a function of departure time differ substantially. Travelers in peak hours face much more uncertainty than travelers on off-peak hours. Thus, temporal effects do not only affect mean traffic flow but also variance in traffic flow, implying that also fluctuation of the demand during the day makes road capacities inefficiently utilized.

The second group of factors that are categorized as demand factors are what we refer to as *network effects* (Figure 2.3), that is the effect of traffic on adjacent, connecting or parallel links and on- and off-ramps on the traffic conditions on the link of interest. A typical example of how network effects influence travel time distribution is the phenomenon of queue spillback. Knoop et al. (2006), for instance, compared the influence of route information in a simulation with spillback and without spillback modeling. The results indicate that the model without spill back overestimates the network performance so that route information does not seem to be effective and hence the travel time gaining from route information is only 1.8%, while the simulation with spillback effects shows that

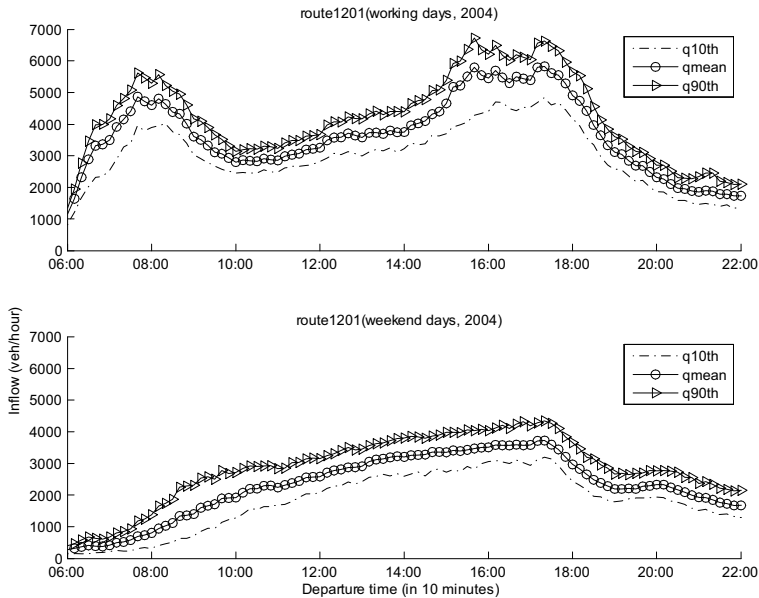


Figure 2.5: Example of temporal effects on traffic flow. The graph shows 10th percentile, mean and 90th percentile values of traffic flow profiles as a function of departure time on workdays and weekend days in the whole year of 2004 on the A12 freeway (southern part of the A12 freeway in the Netherlands).

rerouting is effective and gives 17% reduction of travel time.

The third group of demand factors are so-called "*population characteristics*" (Figure 2.3). These factors include regional and temporal differences in traffic composition (e.g. percentage of trucks, commercial vehicles), regional and temporal differences in driving attitude, driving style and socioeconomic characteristics, all of which affect not only average demand patterns but also demand variations. Note that these population characteristics may also impact variations in supply characteristics (capacities, desired speeds, etc.) due to differences in driving skills (e.g. beginners vs. experienced drivers), age, gender, fatigue level, concentration level and so on.

Finally, an important class of demand factors are the (potential) effects of *traffic information and user response* (Figure 2.3). Providing road users with traffic information may yield cost-benefits for the individual, and potentially also more stable and less congested traffic conditions for all road users. From a traveller perspective, it has been shown that particularly travel time unreliability is valued negatively (Bogers & van Zuylen 2004). The average travel time and the variance of travel time on a specific route are important to travelers. There have been numerous efforts during the last decade to evaluate the benefits of ATIS (see, for example, (Ben-Akiva et al. 1991) (Habib 2004)). The results to date conclude that, by and large, the benefits of route guidance are marginal under conditions of congestion (Habib 2004). Experienced travelers, who make up the major portion of traf-

fic in congested urban networks, have sufficient information to manage their route choice under conditions of recurring congestion. This has often been reflected in the estimation of potential benefits from ATIS in the vicinity of 10% savings in total travel time.

2.3.2 Variations in capacity

In the Highway Capacity Manual 2000, capacity is defined as "*the maximum hourly rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions*" (TRB 2000). In this context, the capacity is treated as a constant value. Doubts about this nature of capacities as constant values were raised by (Ponzlet 1996) and (Brilon 2005) who demonstrated that capacities are probabilistic characteristics, also varying according to factors like adverse weather conditions, traffic control, etc.

In general, capacity is strongly related to the *geometry* and lay-out of the infrastructure facility of interest and the (traffic) regulations that apply to it. Weaving traffic, for instance, can strongly affect the traffic operation. The length of a weaving section has a larger impact on the capacity of weaving sections for the case of short lengths and high demands (Zhang & Rakha 2005). Another example, a single lane on a basic 3-lane freeway stretch has a capacity of about 2100 vehicles per hour, while the capacity of a single lane on a freeway stretch with a weaving section may drop well below 2000 vehicles per hour (TRB 2000). Thereby, the capacity variation may be quite significant, which may induce higher travel time reliability at weaving sections. On the other hand, instead of a constant value capacity can show a significant degree of variability at the same location of along a freeway Bovy (1998) and Brilon (2005). Figure 2.6 demonstrates capacity measured on different time instants on a 2-lane section of the A15 freeway in The Netherlands. It is clear that the capacity varied over time within the specified section.

Secondly, *environmental conditions* such as weather and luminance affect the capacity of road infrastructure. For example, the impact of adverse weather conditions on freeway capacity has been investigated by (Maze et al. 2006). They reported that heavy rains (more than 6.4 mm/h) reduce freeway capacity by an average of 14 percent and heavy snows (more than 12.7 mm/h) reduces freeway capacity by an average of 22 percent. Note that weather and luminance do not so much physically change the infrastructure but rather complicate the driving task given certain headways yielding suboptimal use of the available space and thus lower capacity. Furthermore, the adverse weather not only results in lower capacity but also in higher variations in capacity (Brilon 2005).

Thirdly, (*dynamic*) *traffic management* influences the capacity of road infrastructure. Examples of these include dynamic speed limits, ramp metering, compulsory route guidance, lane segregation and traffic lights at intersections. Papageorgiou et al. (1997) presented results of ramp metering at three on-ramps along a 6-km section of the southern part of the Boulevard Périphérique, Paris. ALINEA (a local feedback ramp-metering strategy) local control results in a 5% increase in motorway throughput.

Finally, *incidents and accidents* are the most common ones causing non-recurring congestion and hence lower capacity. In case of accidents or incidents, freeways are partly or fully blocked and so have a decreased capacity. Studies have indicated that incidents

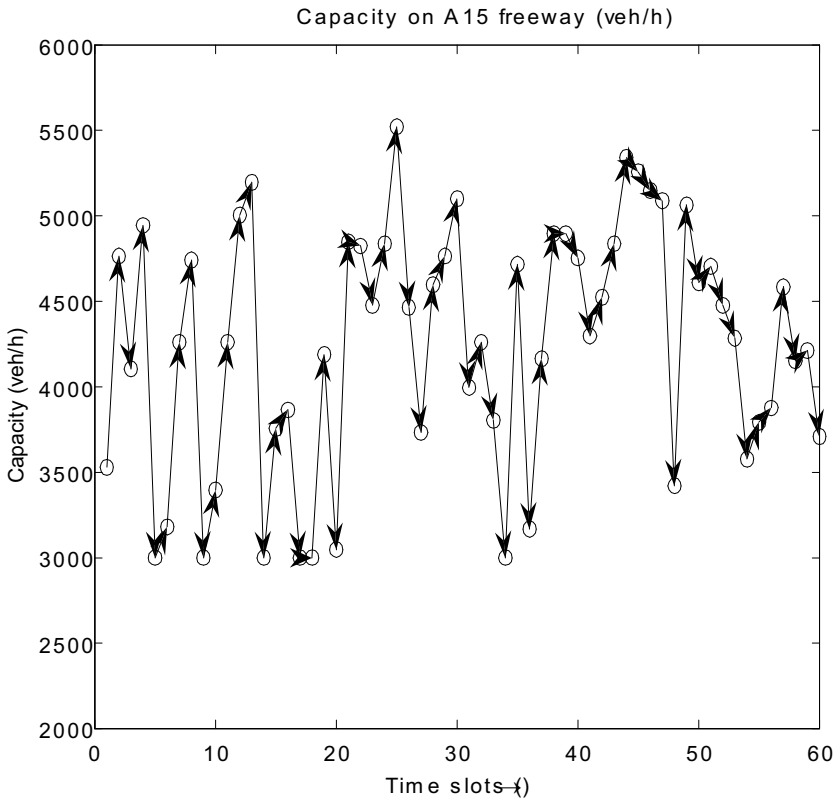


Figure 2.6: Example of capacity measured on different time instant (time slots) for a 2-lane freeway section on A15 freeway in the Netherlands. 10-minute aggregated traffic volume and speed data are collected from inductive loop detectors in the months May and June in the year of 2005. The capacity is estimated by PLM (Product Limit Method, Bovy(1998) and Brilon (2005)).

contribute nearly to 20% of the freeway delay in the Netherlands (Knoop et al. 2008). It is clear that incidents on freeways interrupt traffic flows unexpectedly, and thus are a major cause of freeway capacity-drop. More seriously, they can cause bottlenecks or even secondary accidents. Another obvious factor is the occurrence of *roadworks* since roadworks temporarily (partly or fully) block the freeway.

To conclude there are many sources of travel time unreliability which can be roughly categorized into demand and supply related factors Travel time variability and instability, and hence (un)reliability must be viewed as the combined result of these interrelated factors. In literature most studies devoted to unravel the relationship these demand and supply factors (in almost all cases defined in terms of travel time variability) are simulation-based, rather than based on empirical observations. In the next section, an overview of a number of travel time reliability studies will be presented by a critical discussion..

2.4 Review of Travel Time Reliability Analyses

As discussed in the previous section, transportation networks are exposed to various sources of unreliability in the real world. With all these uncertain environments, the travelers and network planners face unreliability in the network performance or level of service. The impacts of unreliability of the transport network can be listed from those related to our day-to-day life (late arrival or higher level of extra in the departure time), major loss in economic efficiency of the logistics and transport system, or even a threat to national security and safety.

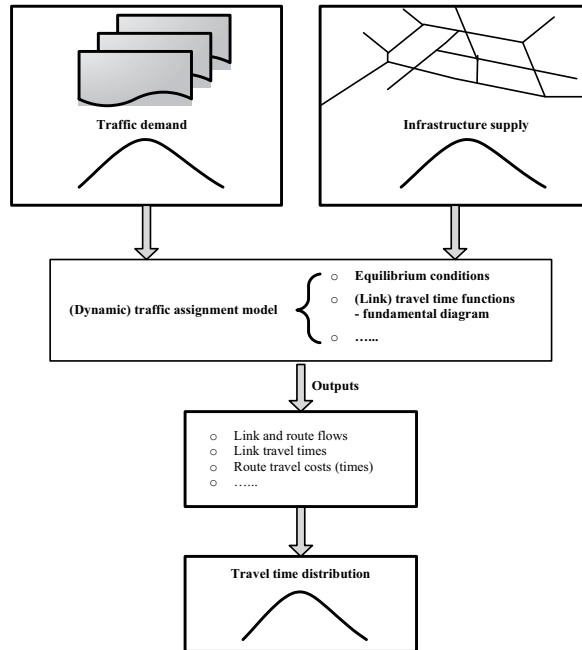


Figure 2.7: Basic structure of travel time variability analysis from the literature (simulation-based)

In order to cope with these problems, one needs to understand how the various factors impact the travel time reliability. Figure 2.7 briefly outlines the basic structure of most travel time reliability studies from the literatures for example (Nicholson & Du 1994), (Nicholson & Du 1997), (Chen et al. 2002), and (Clark & Watling 2005). The dynamic (time-dependent) traffic demand is given by a multidimensional origin-destination matrix with travelers' departure rates from each origin node to each destination node for each time instant (or time interval). The infrastructure supply (capacity) is given by a network consisting of nodes, links and link parameters. For example, it can describe a network of freeway links. Given transport network supply and traffic demand, a traffic assignment model is usually employed to determine an optimal trade-off between supply and demand, based on some given decision rules. These decision rules include the route choice behavior and the departure time choice behavior of the travelers. Two important components of the assignment model are the route cost functions and the link travel time functions. The

route cost functions mathematically describe the (un)attractiveness of a route in terms of (dis)utility (cost, travel time). Link travel time functions, which depend upon the link flows, are used for the propagation of the flows through the network. These link travel times are usually part of the costs. Normally, the travel time function is the relationship between speed (travel time is the reciprocal of speed) and flow, the so-called fundamental diagram. The outputs of the traffic assignment model are given by time-dependent variables, such as time-dependent link and route flows, time-dependent link and route travel times etc. Thus, the travel time distribution for a specific route or network can be obtained from the outputs of a traffic assignment model.

The traffic assignment model can either be stated using an analytical approach or a heuristic approach. Heuristic models normally offer a higher degree of flexibility in traffic modeling (such as including if-then constructions), which enables engineers to establish more realistic models by adding more rules to which the traffic flows should comply (Bliemer 2001). The available methods to study travel time reliability mainly focus on heuristic models, see, for example, Nicholson & Du (1997), Chen et al. (2002), Clark & Watling (2005) and Immers & Jansen (2005). Either the scenario-based method (e.g. sensitivity analysis), the random generation method (e.g. Monte Carlo simulations) or the analytical technique is used to generate the uncertainty both in demand and capacity within heuristic models. An overview of these methods and their main conclusions on the primary causes of travel time reliability are given below:

2.4.1 Sensitivity analysis

Nicholson & Du (1997) used the sensitivity analysis method to investigate the sources of travel time reliability of a Degradable Transportation System (DTS). In their model, the travel time is related to the flow and capacity according to the (Davidson 1966) travel time function

$$t_a = t_a^f \frac{(1 - j_a) \frac{v_a}{C_a}}{1 - \frac{v_a}{C_a}} \quad (2.9)$$

where j_a is a constant ($0 < j_a \leq 1$) on link a and t_a^f is the free flow travel time on link a while v_a is the flow on link a and C_a is the capacity on link a . The value of j_a controls the shape of the travel time function: as j_a decreases, link travel time t_a becomes less sensitive to changes in the flow/capacity ratio at low values of the ratio, and more sensitive at high values. The value of j_a may vary between arcs, according to the characteristics of the travel mode associated with the arcs.

It is assumed that an explicit demand function can be formulated for each OD pair, as follows:

$$v_w = f(c_w) \quad (2.10)$$

where v_w and c_w are the flow and generalized cost between OD pair w , respectively. The demand function could be satisfied by various forms, for instance, logit function, power function, exponential function, and elastic exponential function.

Nicholson and Du used sensitivity analysis to identify the important components in a DTS (i.e. components whose degradation would have a large impact on the performance of the whole system). A weak component is one with a probability of degradation which is markedly higher than for other components, whereas a critical component is one which is both important and weak. Once the important components have been identified, their degradation probabilities can be assessed to identify the critical components. Then these should be the prime candidates for strengthening, rather than those that are merely weak.

According to this approach, they concluded that travel time reliability is proportional to both capacity and inflow variability. For a given (fixed) link capacity, the variability in link travel time is due to link flow variation, while for a given (fixed) link flow, the variability in link travel time is due to variation in the link capacity. They note that travel time variability, in reality, can arise from both sources, and that it is not always an easy matter to identify the separate effects of flow and capacity variations.

Immers & Jansen (2005) also used the sensitivity analysis to minimize the consequences of varying network loads and (large) incidents by an adequate design of the network. Assuming that traffic load (OD table) is increased in 8 consecutive steps starting with free flow conditions (base load) and ending with serious congestion in the network (8 times the base load), while local variations in supply are modeled as an incident on one (heavily loaded) link, they found that the location of a link or a node is important in the sense that in certain cases congestion and associated unreliability are confined to the concerned link or a small part of the network. In other cases congestion at a centrally located link or node may cause a series of cascading failures disrupting traffic on large parts of the network. It is also found that higher traffic loads make the network more sensitive to small variations in demand or supply.

2.4.2 Monte Carlo simulation

Roadway capacities are random in nature with continuous degrees of degradation and possible correlation among the components. A reasonable way to capture these variations and their is to model roadway capacities using probability distributions (Brilon 2005). In this context, Chen et al. (2002) developed an assessment methodology, which integrates reliability and uncertainty analysis, network equilibrium models, sensitivity analysis of equilibrium network flow and expected performance measure, and Monte Carlo methods, to evaluate the performance (both capacity reliability and travel time reliability) of a degradable road network. The random capacities are proposed to be non-normal and correlated with the vector of means \bar{C} and a covariance matrix $Cov(C)$ of random capacities. Thus, the travel time reliability is defined as the probability $\tau_w(\theta)$ for the ratio between $\mu_w(C)$ and $\mu_w(C_0)$ to be within an acceptable level θ . That is,

$$\tau_w(\theta) = P\left(\frac{\mu_w(C)}{\mu_w(C_0)} \leq \theta\right) \quad (2.11)$$

where,

C and C_0 are the vectors of degraded and non-degraded capacities

$\mu_w(C)$ and $\mu_w(C_0)$ are the travel times between the OD pair w under degraded and non-degraded capacities

θ can be interpreted as the level of service that should be maintained despite the capacity degradation that has occurred on certain arc in the network.

On a small test network, Chen et al. (2002) use Monte Carlo methods to analyze amongst other things the sensitivity of travel time reliability to fluctuations in link capacities. They conclude that travel time reliability (at least their definition of it) decreases as the demand level increase, which *'is no surprise since traffic congestion grows as a result of higher demand'*. They also show that the sensitivity of path travel time reliability to individual link capacity fluctuations differs largely. Capacity variations on one link may have a huge impact on path travel time reliability, while capacity variations on other links may affect travel time reliability only marginally.

A similar research was carried out by (Chen & Recker 2001). In their Monte Carlo simulation framework, link capacities and OD demands are treated as random variables in which they assume a probability distribution between some upper and a lower bound value. Their procedure is to simulate risk perceptions and preferences in making route choice decisions in an uncertain environment. The link travel time function used in the route choice model is the standard Bureau of Public Road (BPR) function (Bureau of Public Roads 1964) given below:

$$t_a = t_a^f \left(1 + 0.5 \left(\frac{v_a}{C_a} \right)^4 \right) \quad (2.12)$$

where,

t_a and t_a^f are the travel time and free flow travel time on link a , respectively.

v_a and C_a are the flow and capacity on link a , respectively.

Numerical results were also presented to examine what the aggregate impact of changes in variability caused by demand and supply variations might be on travel time reliability. They found that the uncertainty due to variations in both demand and supply (capacity) can significantly contribute to the travel time reliability.

2.4.3 Analytical techniques

Clark & Watling (2005) aimed to reconstruct a full probability distribution for the network performance (total network travel time) to examine the impact of variable OD demand flows on the total network travel time. Basic assumptions in their approach are:

- Each OD pair w is distributed as a stationary Poisson random variable with constant mean $q_w > 0$.
- Link travel time function is proposed as a polynomial form (Eq. 2.13)

$$t_a(v_a) = \sum_{j=0}^m b_{ja} v_a^j \quad (2.13)$$

- Total link travel time is the sum of link flow and link travel time

$$W_a = v_a t_a(v_a) = \sum_{j=0}^m b_{ja} v_a^{j+1} \quad (2.14)$$

where W_a is the total link travel time on link a .

- Total network travel time is the sum of total link travel times

$$T = \sum_{a=1}^A W_a \quad (2.15)$$

where T is the total network travel time

After the moments of the total network travel time distribution are computed by the analytic method, a flexible family of density functions is fitted to these moments. Based on the numerical results from a five-link test network, Clark & Watling (2005) noted that network travel time reliability decreases as capacity decreases for a given demand level and 'spare capacity' allows a network to deal better with unexpected variation.

2.4.4 Discussion

Although these studies provide valuable insight into the causes of travel time reliability, and yield intuitive results that both demand variation and supply (capacity) variation can significantly contribute to the travel time reliability, several issues related to the previous studies are questionable. More specifically

- *Approaches*

1. *Static network equilibrium.* Prior studies are in most cases simulation-based using a static network equilibrium model. However, path travel times are the result of highly stochastic dynamic traffic processes along this path which are not captured adequately by the static approaches. Bliemer & Bovy (2003) note that static assignment essentially predicts congestion at the wrong places and wrong times. Consequently, the resulting path travel time distributions are also not very likely to be realistic. As Peeta & Ziliaskopoulos (2001) put it: '*debating on whether dynamic models are better than static ones is hardly the issue; dynamic models are obviously superior, since they relax more assumptions and capture more realities than the static approaches*'.

2. *Link (or section) capacities are assumed to be independent.* The capacity on a certain link has significant impact on that on the adjacent link due to, for instance, the queue spillback effects. It is well known that spillback can lead to grid lock on ring roads and that, as a result, the time-dependent output flows (by OD pair) for networks susceptible to gridlock may be very sensitive to the time-dependent input flows; e.g., a slight increase in the feasible OD flows of a gridlock-susceptible road can result in no output flow at all (Daganzo 1998).
3. *Travel time function.* The travel time functions, for instance, Davidson travel time function and Polynomial flow-travel time function (BPR travel time function is a special case), are on a basis of the flow-travel time (or flow-speed) plane following a one-dimensional area. It is well known, however, that the flow-speed diagram, especially in the congested part is widely scattered within a two-dimensional area (Helbing 2001).

- *Factors*

4. *Factors influencing travel time reliability.* Previous studies focus mostly on two aspects influencing travel time reliability: demand and supply. However, as shown in Figure 2.3, demand and supply are the result of how other factors affect traffic (e.g. traffic information, adverse weather, and road geometry etc.). The impacts of these specific factors on travel time reliability have not been deliberated by previous studies.

In order to cope with these problems, in this dissertation, we follow a completely different approach. We use empirical travel times obtained from dual loop detectors on freeways instead of simulation-outcomes. Thus, travel time reliability as it can be measured from the empirical data, is analyzed and plausible causes for the reliability are determined. The approach is shown in Figure 2.8. In this framework, there are no assumptions on the demand and capacity distribution, travel time functions etc. Since we use the empirical data, traffic assignment models can be avoided to investigate the causes of travel time unreliability. In this context, an explanatory travel time model can be constructed, which can then be applied in a (dynamic) traffic assignment model.

2.5 Summary

This dissertation focuses on travel time reliability modeling. To this end, this chapter presented definitions for travel times, reliability and travel time reliability. We also illustrated the key differences between travel time variability and travel time reliability. The latter depends on the variability in travel times as well as the instability of the travel times. As we will see in the next chapter, this implies that travel time reliability includes in fact these two elements.

Travel times are the results of the highly complex and dynamic interplay of various factors. These factors can be categorized as factors influencing travel time unreliability through the traffic demand (e.g. daily, weekly and monthly activity patterns, composition of the population) and those influencing travel time unreliability through the traffic

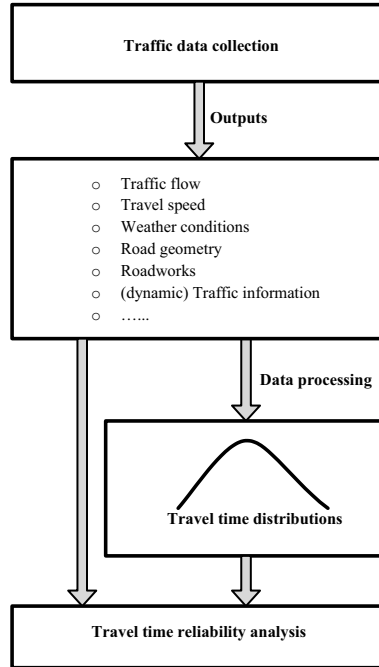


Figure 2.8: General framework of empirically analyzing travel time reliability

supply (i.e. infrastructure capacity), (e.g. road works and weather conditions). In the following chapters, we are going to address some of the factors which influence travel time reliability.

The second part of this chapter gave an overview of travel time reliability studies. Two conclusion can be drawn here. First, the approach from literature to analyze travel time reliability focuses on (static) network equilibrium, which is unrealistic. Second, the analysis mostly concentrates on two main aspects (demand and capacity), which does not go into the detailed factors. In this context, the travel time reliability model will be derived from a large dataset of empirical data from inductive loop detectors without any assumption on the assignment. The latter will be addressed in the remainder of this dissertation.

Chapter 3

Conceptual Travel Time Reliability Model

3.1 Introduction

In order to be able to make forecasts about future traffic conditions on transport networks, to compare scenarios of different infrastructure investments, or to estimate effects of traffic management measures, policy makers and transport planners rely on tools such as traffic assignment models. The outcomes of a traffic assignment model include the route split proportions, the traffic flows, and the travel times and costs on the network. The interdependence among travel costs, travel times, travel time reliability, flows, and the processes in the traffic assignment models are illustrated in Figure 3.1. Route and departure time choices play a crucial role in a traffic assignment model and it is known that these choices do not only depend on expected travel time, but also on travel time reliability. In this respect, travel time reliability not only contributes to the utility function or travel costs, but also plays an important role in route choice (Bogers & van Zuylen 2004) and departure time choice (Li et al. (2007), Li et al. (2008)). Therefore, a good model is needed to describe travel time reliability in such a way it can be used for traffic assignments. Furthermore, the policy makers and transport planners like to know what the influences of traffic control measures or road geometry are on travel time reliability. A model that describes the influences of certain conditions on travel time reliability can help making decisions on measures to improve travel time reliability.

Such a travel time reliability model should have sufficient credibility (or validity) (van Zuylen et al. 2007) (see details in section 6.1). Therefore, the parameters in the model should be logically related to the characteristics of the system, called content validity. Travel time unreliability is the result of traffic flow operations, which in turn is governed by the fluctuations in both traffic demand and supply characteristics, which is schematically outlined in Figure 2.3. In this respect, a travel time reliability model should have parameters with all the factors in Figure 2.3.

Since in the traffic assignment process (Figure 3.1) the result of route and departure time choices becomes visible as flows on the links, the travel time reliability model should have parameters that include at least the inflow of a path. It is known that if there is no

flow on the road, there are no travel times and no travel time unreliability problem. In this sense, traffic flow is one of the most important factors (probably the most important one) influencing travel time reliability. Thus, the (in)flow can be considered as the principal parameter in the travel time reliability model. Furthermore, The geometry of the road, the traffic control measures, the conditions that have a considerable impact on travel time reliability but are not controllable, like the weather conditions (Tu, van Lint & van Zuylen 2007a), should be included in the model as well. Therefore, the reliability model can be regarded as a function of inflow levels given certain conditions, as following:

$$TTR = f(q_{in} | f_r, f_w, f_c, f_a, f_o) \quad (3.1)$$

in which

TTR	= Travel Time Reliability
q_{in}	= Traffic flow (inflow)
f_r	= Road geometry
f_w	= Weather conditions
f_c	= Traffic control measures
f_a	= Accidents
f_o	= Other factors

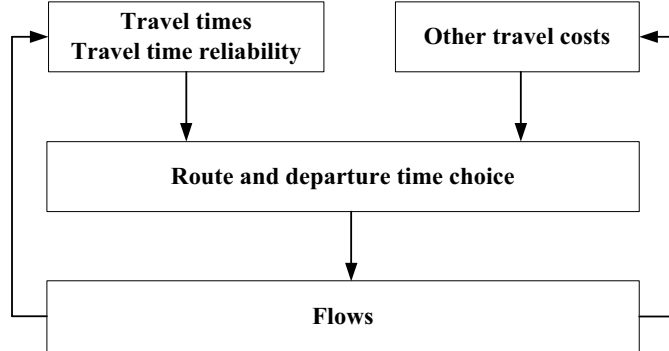


Figure 3.1: Interdependence between traffic processes and choice variables in the traffic assignment model

The relationship between flow and travel time (reliability) is essential to build up the travel time reliability model. In this dissertation, such a model is derived based on empirical (loop detector) data. Since travel time is a reciprocal value of travel speed, knowledge of the traffic states and transitions in the speed-flow plane can help better understand the flow-travel time relationship. Here the speed-flow plane is not just a simple fundamental relationship, but is viewed as a statistical relationship and is characterized by a wide scatter (see also section 3.2.1 and Figure 3.4). In this study, the inflow q_{in} is proposed as the indicator of traffic flow and the measurement interval of inflow is on 10-minute aggregate. Further information will be addressed in section 4.2.2.

In Figure 3.2, the structure to build up a travel time reliability model is depicted. Firstly, the characteristics of speed-flow and travel time-flow planes are discussed in section 3.2. After a short overview of the fundamental diagram, some features in the fundamental diagram are taken as a starting point to discuss some basic characteristics of the speed-flow plane in section 3.2.1. Next we focus on the characteristics of speed-flow plane and travel time-flow from the empirical observations in section 3.2.2. Thereafter, we turn to present the components of travel time reliability in section 3.3. That is, the instability of travel times is discussed in section 3.3.1 and the severity of congestion due to traffic breakdown is discussed in section 3.3.2. Then in section 3.4 we come up with the conceptual travel time reliability model in which the elements of travel time reliability are combined. Finally, in section 3.5 a summary of some important conclusions is presented.

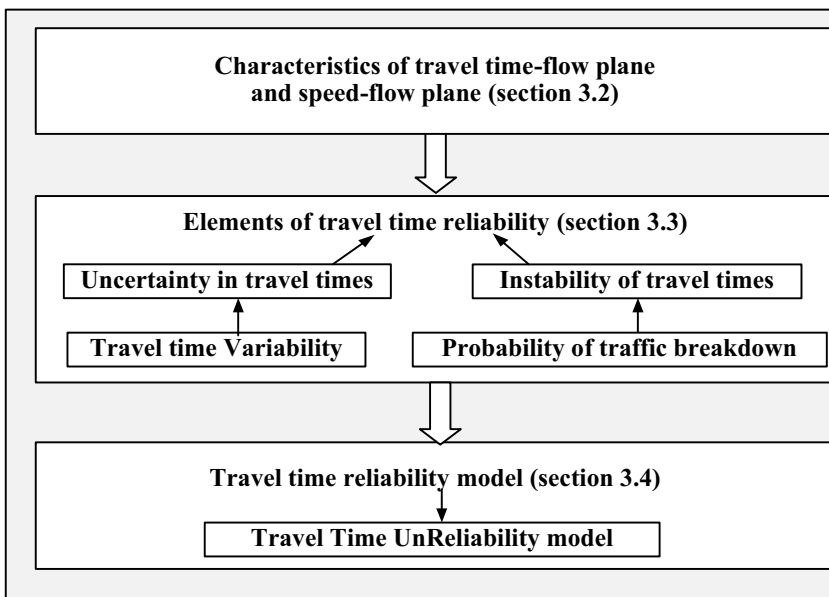


Figure 3.2: A diagram to set up a travel time reliability model

3.2 Characteristics of Travel Time-Flow Plane and Speed-Flow Plane

In order to investigate the travel time reliability, there is a need to clarify what the elements of travel time reliability are. In this section, the characteristics of travel time-flow and speed-flow relations are discussed, from which the elements of the travel time reliability model are derived. The next subsection briefly outlines the relationship between speed and flow.

3.2.1 Fundamental relations

The interest in the subject of traffic flow characteristics is not new. Greenshields (1935) and Greenshields et al. (1947) carried out early studies of vehicular traffic: the study of models relating volume and speed and the investigation of performance of traffic at intersections. After WWII, with the tremendous increase in use of automobiles and the expansion of the highway system, there was also a surge in the study of traffic characteristics and the development of traffic flow theories. In the 1950s, there was considerable publication activity in journals on operations research physics and engineering based on a variety of approaches, such as car-following, traffic wave theory (hydrodynamic analogy) and queuing theory. Some of the works of that period include the studies by (Reuschel (1950a), Reuschel (1950b), Reuschel (1950c), Wardrop (1952), Pipes (1953), Lighthill & Whitham (1955), Newell (1955), Webster (1958), Edie & Foote (1958), Chandler et al. (1958)). These papers introduced the fundamental diagram showing the relation between traffic flow and vehicle density or the instability of traffic flow, which are still relevant. The detailed review of traffic flow theory can be referred to, for instance, (Hoogendoorn & Bovy 2001) and (Helbing 2001). The motivation for such studies was self-evident. As Greenberg wrote in 1959, "*The volume of vehicular traffic in the past several years has rapidly outstripped the capacities of the nation's highways. It has become increasingly necessary to understand the dynamics of traffic flow and obtain a mathematical description of the process.*"

Functional relations between the vehicle flow $q(x, t)$, the average velocity $v(x, t)$, and the vehicle density $\rho(x, t)$ or occupancy $o(x, t)$ have been measured for decades, beginning with (Greenshields 1935), who found a linear velocity-density relationship. The name *fundamental diagram* is mostly used for some empirical relations, such as speed-flow relation

$$V = V(q) \tag{3.2}$$

where $V(q)$ stands for the fitted empirical speed-flow relation. We will address neither the speed-density relation $V(k)$ nor the flow-density relation $Q(k)$, but the speed-flow relation only because of the fact that density is not directly measurable from the empirical loop detector data. Density is defined in space while detectors measure on point. In the fundamental diagram, a multitude of the steady-state model solutions cover a one-dimensional region in the speed-flow plane either in free-flow region or in congested flow region (Figure 3.3). The following features (e.g. see Helbing (2001), Kerner (2004), and Papageorgiou et al. (2007)) can be observed:

1. In the free-flow part, speed slightly decreases with flow increasing (at low densities ρ). The speed is under free flow speed conditions, which can be sustained at many densities as long as there are sufficient possibilities for overtaking.
2. In the congested flow part, with decreasing speed, the traffic volume decreases monotonically, and it vanishes together with the speed at zero.
3. The vehicle flow has one maximum q_{\max} (capacity).

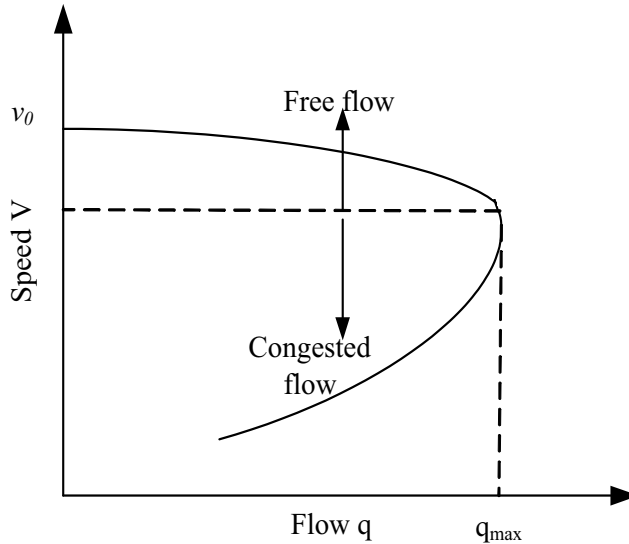


Figure 3.3: Fundamental diagram, flow-density plane in one-dimensional region for both free-flow and congested flow parts

However, some studies like Banks (1991) and Brilon & Ponzlet (1996) claimed the so-called *capacity drop* or the *two-capacity* phenomenon: *free flow capacity*, the absolutely highest flow rates are in general observed shortly before traffic breakdown, and *queue discharge rate*, the highest flow rates that occur at the head of the queue once traffic has broken down. In the literature the differences between two capacities ranges from 5-10% (Cassidy & Bertini 1999) (Bertini & Cassidy 2002), to 20% (Smulders et al. 2000) up to 50% (Kerner 1999). More recent investigations come to the conclusion that the capacity of a freeway shows a significant degree of variability. To some extent, observed differences in capacity are due to systematic external influences like daylight/darkness or weather conditions (Brilon & Ponzlet 1996). There are, however, other and even larger fluctuations that cannot be explained by any external or traffic-flow parameters which must, therefore, be regarded as a random variable. This random nature of capacity on freeways has been studied by several authors (among others: (Minderhoud et al. 1997) (Lorenz & Elefteriadou 2000) (Brilon 2005)). This fundamental relation, accordingly, can not reflect the random nature of traffic flow characteristics. In the next subsection, we focus on the empirical speed (travel time)-flow relations.

3.2.2 Empirical relations

In recent studies, it has been shown that the speed-flow data in both free-flow and congested flow parts are widely scattered within a two-dimensional area (see, for example, (Koshi et al. 1981), (Hall et al. 1986), (Kerner & Rehborn 1996)). Figure 3.4 shows an example of the empirical fundamental diagrams of (a) speed-flow relation and (b) travel time-flow relation (obtained through local 10-minute aggregated measurements of time-

mean flow and time-mean speed on the A20 freeway in the whole year of 2004 in The Netherlands). Note that Figure 3.4(a) has a particular property that is not typical for fundamental diagrams: for very low flows, the speed decreases. This effect is site-specific and is an effect of changing traffic composition: these data points are related to night traffic, which - due to the presence of a large harbour in Rotterdam - has a very high fraction of trucks, driving at lower speeds than cars. This site-specific observation shows how the characteristics of the fundamental diagram not only depend on the measurement location with respect to bottlenecks but also on the composition and characteristics of traffic. It is important to realize that such a diagram reflects a *local relationship* between the macroscopic traffic variables, which should not be perceived as a causal relationship. It has been found (also from the example) that there is no one-on-one relationship of speed and flow or travel time and flow, neither in the free-flow, nor in the congested branch of the fundamental diagram. The measurements of speed-flow plane and travel time-flow plane are both widely scattered. In general, the diagram only suggests relations between speed (travel time) and flow. To some extent, the observed scattered speed-flow plane is due to internal and external factors like vehicle composition, weather conditions, traffic control measures, etc. The impacts of the internal and external factors on traffic flow lead to the instability of flows.

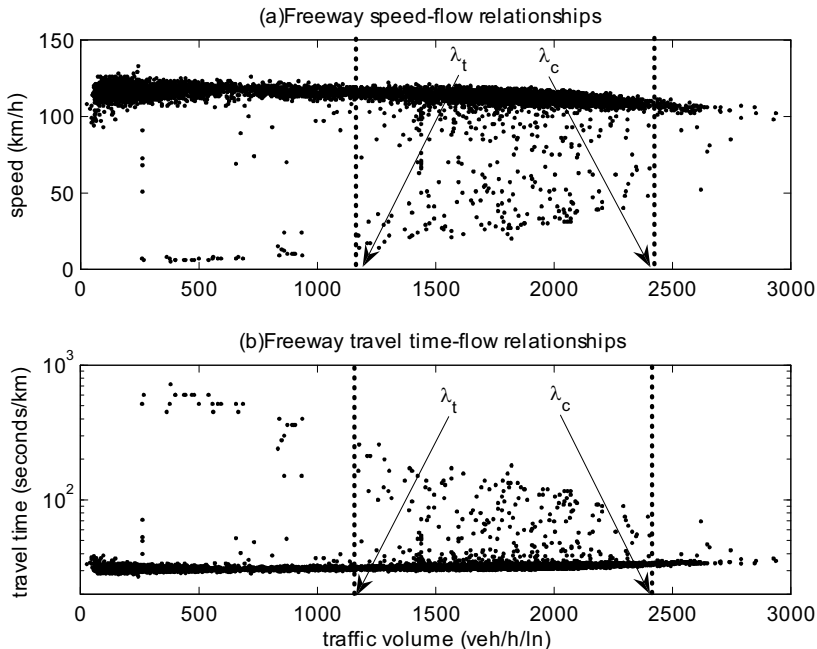


Figure 3.4: Example of empirical fundamental diagram: (a) speed-flow relations and (b) travel time-flow relations on A20 freeway in the whole year of 2004 (speed and flow are both measured in 10-minute aggregate) in The Netherlands. Note that travel time unit here is the travel time (or seconds) per km, the reciprocal value of speed.

If we take a close look at the travel time-flow plane (Figure 3.4), there appears to be two

critical inflow levels, critical *transition* inflow λ_t and critical *capacity* inflow λ_c . In case inflow below λ_t , it is nearly certain that we deal with free flow condition; between λ_t and λ_c , flows are uncertain, either in free flow conditions or in congested conditions; above λ_c (the flow is close to capacity) it is also certain that flows have a free flow condition because such high flow rates are only possible in uncongested conditions. However, when the flow is close to capacity, it is very likely that some minutes later the flow will break down and congestion will set in with the consequence that the flow and speeds become lower and the travel time will become longer. Therefore, we have three regions:

- Below λ_t , certain and stable flows
- Between λ_t and λ_c , uncertain and unstable flows
- Above λ_c , certain but instable flows.

Figure 3.5 illustrates these three regions and the representation of the relationship between uncertainty and inflow and the relationship between instability and inflow (Tu et al. 2008).

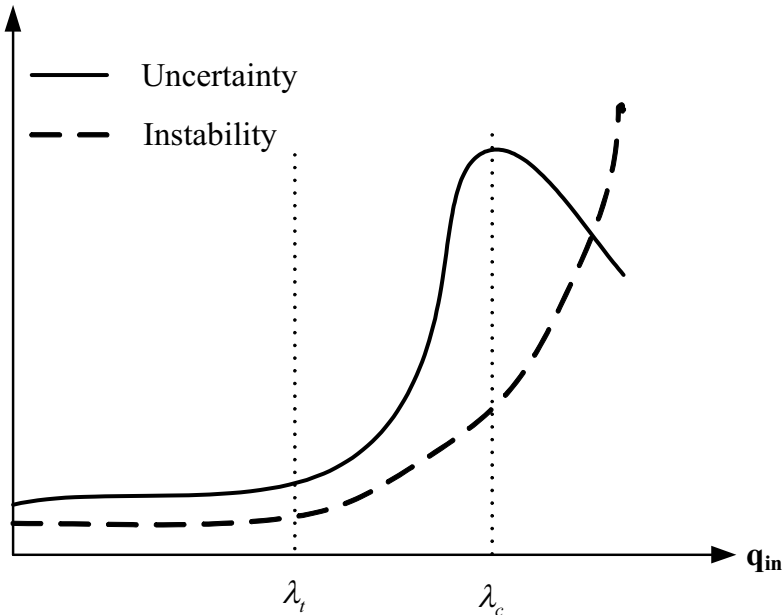


Figure 3.5: Representation of the static relationship between uncertainty and inflow (q_{in}) and the static relationship between instability and inflow. (Y-axis: the higher value, the higher uncertainty or instability)

3.3 Elements of Travel Time Reliability

Based on these properties of the speed-flow and travel time-flow relationships, we argue that travel time reliability includes two parts: travel time uncertainty and instability of

travel times, as shown in Eq. 2.1. Therefore, travel times are unreliable in case they are either unstable, uncertain or both:

$$\textit{Travel time unreliability} = \{\textit{Instability}, \textit{Uncertainty}\} \quad (3.3)$$

More specifically, we argue that

1. Travel times are unreliable if they are uncertain (so if they vary largely, either congested or free flowing), that is, if there is a wide range of possible travel times given the prevailing inflow rate
2. Travel times are unreliable if they are certain at a moment but unstable, that is, if there is a large probability of traffic breakdown under the prevailing circumstances at a next moment
3. Travel times are unreliable if the consequences of the breakdown are severe, in terms of long delays (increasing travel time)

In terms of the elements of travel time (un)reliability, the following questions need to be answered:

1. What is the travel time variability?
2. What is the travel time uncertainty? How to quantify travel time uncertainty?
3. What is the definition of traffic breakdown? And how to quantify the probability of traffic breakdown?
4. How to quantify the impact of traffic breakdown on travel time?

The first question has been addressed in section 2.2.4. The next two subsections will answer the latter three questions.

3.3.1 Traffic breakdown and congestion patterns

Traffic breakdown of a section

As shown in Figure 3.4, both the speed-flow plots and the travel time-flow plots are widely scattered. The data presented here clearly demonstrate that flows between λ_t and λ_c are either in free flow conditions or in the congested conditions. Thus, in this region, flows are instable. Traffic consists of vehicles, controlled by drivers, who are all assumed to have the intention to travel safely at their desired maximum speed. Then how can a high speed free flowing traffic state change into a low speed congested traffic state (traffic breakdown)? Basically two views on the causes of traffic breakdown can be found from the literature, for instance (Daganzo et al. 1999) and (Kerner & Rehborn 1996). The first argues that traffic breakdown is always deterministically associated with the presence of a bottleneck (Daganzo et al. 1999), whereas the second claims traffic can break down

in absence of any apparent bottleneck, but the probability of the traffic breakdown will be much higher in the presence of a bottleneck. However, both state that the probability of traffic breakdown increase with increasing density. Furthermore, traffic breakdown does not necessarily occur at maximum flow and it can occur over a range of flow rates including flows lower or higher than those traditionally accepted as capacity (Lorenz & Elefteriadou 2000). The breakdown of a section is a probabilistic variable and not deterministic (Elefteriadou et al. 1995) and the probability of traffic breakdown is an increasing function of inflow (Persaud et al. 1998). Here we used the same definition of traffic breakdown as (Brilon 2005):

Section traffic breakdown: A reduction of average speed of a section within one time interval from a high level down to below a threshold of 70km/h is treated as a section breakdown.

Observations on both Dutch and German freeways clearly show that most of the breakdowns occurred as a sharp reduction of average speeds (70km/h) within a short time interval. This speed threshold is also used to distinguish the free flow condition and congested condition. The size of time interval also affects the measurement of the probability of traffic breakdown. Here 10-minute is chosen as the time interval (see details in section 4.2.2). Therefore, the probability that traffic flows in a congested condition given a certain inflow q_{in} equals:

$$p_k^{jam}(q_{in}) = \frac{n_k^{jam}(q_{in})}{N_k(q_{in})} \quad (3.4)$$

where

- $p_k^{jam}(q_{in})$ = the probability that traffic flows in congested condition for a given inflow level q_{in} on section k
- $n_k^{jam}(q_{in})$ = the number of intervals with traffic are under congested condition for a given inflow level q_{in} on section k
- $N_k(q_{in})$ = the total number intervals in a given inflow level q_{in} on section k

The probability of traffic breakdown on a section is defined as:

$$p_k^{br}(q_{in}) = \frac{n_k^{br}(q_{in})}{N_k^f(q_{in})} \quad (3.5)$$

where

- $p_k^{br}(q_{in})$ = the probability that traffic breaks down for a given inflow level q_{in} on section k
- $n_k^{br}(q_{in})$ = the number of time intervals that traffic flows in free flow conditions but followed by a traffic breakdown for a given inflow level q_{in} on section k
- $N_k^f(q_{in})$ = the total number intervals that traffic flows in free flow conditions for a given inflow level q_{in} on section k (include $n_k^{br}(q_{in})$)

Probability of traffic breakdown on a section can be categorized into two types:

- Spontaneous breakdown $p_k^{brs}(q_{in})$: A local spontaneous breakdown is caused by an *internal* local disturbance in traffic flow. This internal local disturbance can be associated with a deterministic local perturbation at a freeway bottleneck or to the occurrence of a *random* local perturbation in traffic flow.
- Induced breakdown $p_k^{bri}(q_{in})$: A local induced breakdown is caused by an *external* short-time local disturbance in traffic flow. This external disturbance can be associated with the propagation of a queue that has initially occurred at a different freeway location in comparison with the location of the induced breakdown.

Attention should be paid that: firstly, the spontaneous breakdown and the induced breakdown are independent due to the different local disturbance; secondly, spontaneous breakdown and induced breakdown can not occur at the same time, that is if spontaneous breakdown occurs in a section at current time step, the induced breakdown can not occur at the next time steps even the propagation of a queue comes to the section. Thus, the probability of traffic breakdown on a section includes both the probability of spontaneous breakdown and the probability of induced breakdown.

$$p_k^{br}(q_{in}) = p_k^{brs}(q_{in}) + p_k^{bri}(q_{in}) \quad (3.6)$$

A section, for instance a basic freeway section, with a high value of $p_k^{bri}(q_{in})$ has a low value of $p_k^{brs}(q_{in})$ since breakdown of the section, in this case, is mostly due to the propagation of a queue from downstream bottleneck section like on-ramp section. That is, induced breakdown probably occurs before spontaneous breakdown on the basic freeway section. Therefore, in Eq. 3.6, $p_k^{br}(q_{in})$ is always smaller than 1.

So far, we have only briefly discussed traffic breakdown on a section-level basis. This is however not sufficient to analyze the route-level traffic breakdown since the adjacent sections are highly dependent. Traffic flows on the downstream section probably have significant influences on upstream flows. In the next subsection, we will address the probability of traffic breakdown on a route level.

Traffic breakdown of a route

The adjacent sections along a route have strong interconnections. One of the common phenomena is the effect of the spillback on the sections. In the queue spillback, vehicles decelerate from a phase with higher speed to a phase with lower speed. The motion of the upstream jam front is determined by the flow rate and density in the fast region upstream of the front, and by the flow rate and density inside the slow congested region (for instance, Kerner & Rehborn (1996), Daganzo et al. (1999), Helbing (2001), Brilon (2005)). A route of a freeway is combined of several dependent adjacent sections. If traffic breaks down on a section, queues propagate upstream. Figure 3.6 presents an empirical example of congestion propagate upstream on A13 freeway on January 8 in 2004. As can be seen in Figure 3.6, one phenomenon is found that some of the congestions continuously propagate upstream over long time while the other congestions only last a short time. The phenomenon is due to different traffic conditions, such as traffic flow, weather conditions,

traffic control, road geometry etc. On the other hand, there are strong temporal and spatial interactions between sections along the route. The probability of traffic breakdown of a route is defined as:

Route traffic breakdown: Traffic breakdown of a route occurs in case of at least one section along the route breaks down.

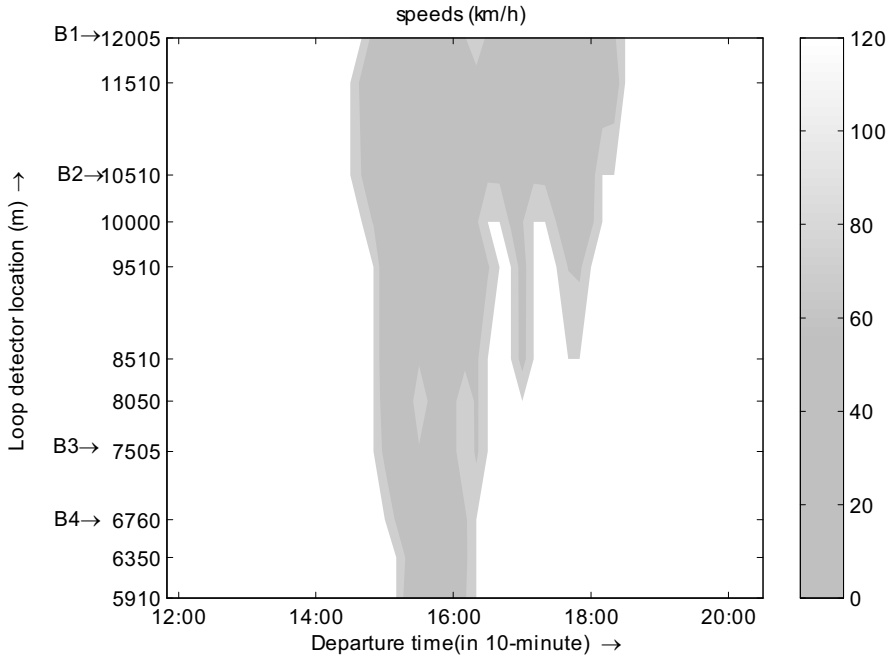


Figure 3.6: Empirical example of queue spillback on A13 freeway (8 January, 2004). Free flow (above 70km/h, in white) and congested flow (below 70km/h, in grey). BX represents the bottlenecks along the freeway, such as B1-4 are the bottlenecks around the detector location 12005m, 10510m, 7505m, and 6760m, respectively. Traffic direction is from small number to large number.

Consequently, the probability of traffic breakdown on a route can be formulated as:

$$p_r^{br}(q_{in}) = \frac{n_r^{br}(q_{in})}{N_r^f(q_{in})} \quad (3.7)$$

where

- $p_r^{br}(q_{in})$ = the probability of traffic breakdown for a given inflow level q_{in} on route r
- $n_r^{br}(q_{in})$ = the number of time intervals that traffic in free flow conditions but followed by a traffic breakdown for a given inflow level q_{in} on route r
- $N_r^f(q_{in})$ = the total number intervals that traffic in free flow conditions for a given inflow level q_{in} on route r (include $n_r^{br}(q_{in})$)

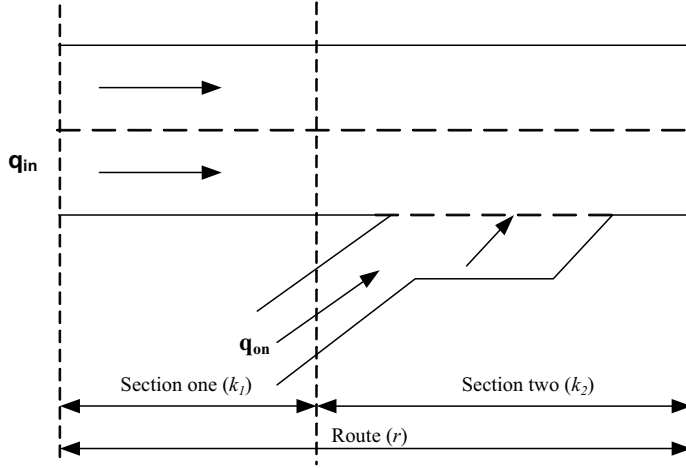


Figure 3.7: Scheme of a route with one basic freeway section (k_1) and one on-ramp section (k_2)

Eq. 3.7 only illustrates the general approach to measure $p_r^{br}(q_{in})$, but without addressing the relationship between $p_r^{br}(q_{in})$ and each section $p_k^{br}(q_{in})$ along a route. A route is combined of several dependent adjacent sections which have strong interconnections. The probability of traffic breakdown of a route, therefore, has a strong correlation with the probability of traffic breakdown of the sections along the route. Since the spontaneous breakdown is due to the internal (local) factors, the spontaneous breakdowns of adjacent sections along a route are considered to be independent. Thus, the probability of (spontaneous) traffic breakdown of a route is one minus the product of the probability of non-breakdown of each section, which can be formulated as:

$$p_r^{brs}(q_{in}) = 1 - \prod_{k=1}^n (1 - p_k^{brs}(q_{in})) \quad (3.8)$$

in which

- $p_r^{brs}(q_{in})$ = the probability of (spontaneous) traffic breakdown for a given inflow q_{in} on route r . Note that the spontaneous traffic breakdown of a route is not affected by the queue propagation of the downstream adjacent route
- $p_k^{brs}(q_{in})$ = the probability of spontaneous traffic breakdown for a given inflow level q_{in} on section k
- n = the number of sections along the route

From a network level, a route does have a correlation with adjacent route, especially the downstream adjacent route. Therefore, the probability of (induced) traffic breakdown of a route is caused by an external disturbance in traffic flow. This external disturbance can be associated with the propagation of a queue that has initially occurred at a downstream adjacent route. Thus, the total probability of traffic breakdown on route r can be formulated as following:

$$p_r^{br}(q_{in}) = p_r^{brs}(q_{in}) + p_r^{bri}(q_{in}) \tag{3.9}$$

where $p_r^{bri}(q_{in})$ = the probability of (induced) traffic breakdown of route r for a given inflow level q_{in} due to the propagation of a queue that has initially occurred at a downstream adjacent route. Analogously to section $p_k^{br}(q_{in})$, $p_r^{br}(q_{in})$ is also smaller than or equals 1.

To illustrate the relationship between $p_r^{br}(q_{in})$ of a route and each section $p_k^{br}(q_{in})$ along the route, a route r with one basic freeway section k_1 and one on-ramp section k_2 on A12 freeway in The Netherlands is presented in Figure 3.7. Figure 3.8 demonstrates the relationship between traffic breakdown of a route and traffic breakdown of each section along the route on A12 freeway in The Netherlands. Traffic disturbance increases with rising inflows due to the effects of weaving vehicles on section two (k_2). At inflows above a certain value (1680 veh/h/ln), the probability of traffic breakdown on both section two k_2 and route r sharply increases with rising inflows. The graph also shows that $p_r^{br}(q_{in})$ has much higher correlation with $p_{k_2}^{br}(q_{in})$ of the bottleneck section k_2 than the basic freeway section k_1 . Furthermore, the estimated $p_r^{br}(q_{in})$ (circles in the graph) is very close to the observed $p_r^{br}(q_{in})$ (solid line in the graph). That is, Eq. 3.8 can approximately estimate $p_r^{br}(q_{in})$ by combining each section $p_k^{br}(q_{in})$ along the route.

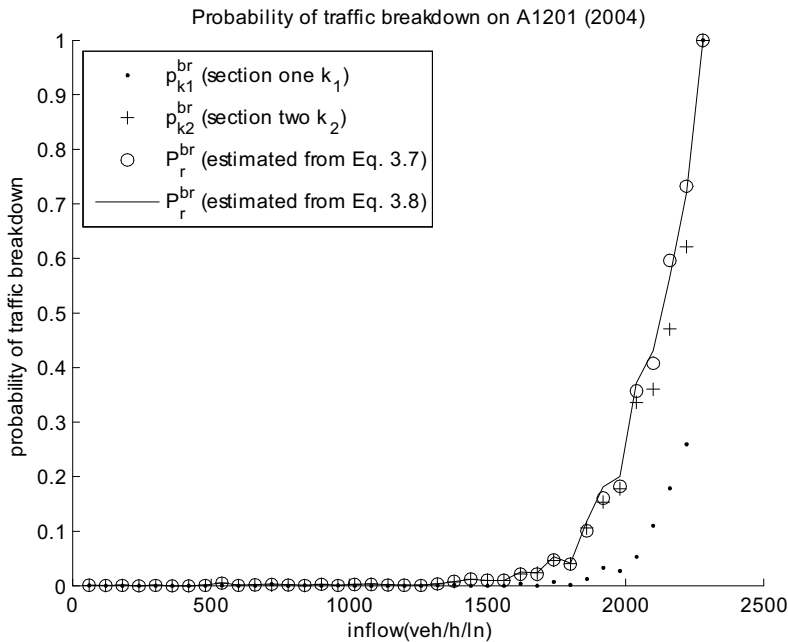


Figure 3.8: Example relationship between traffic breakdown on a route and traffic breakdown on a section of the route (A12 freeway in The Netherlands in the year of 2004, 10-minute aggregate). Traffic breakdown on section one without spillback of section two. p_r^{br} (observed) is calculated by Eq. 3.7 and P_r^{br} (estimated) is measured by Eq. 3.8.

Congestion patterns

Once traffic breaks down, the flows are in congested conditions. Several studies presented the theory that the congested regime may not be a single dynamic phase but rather a collection of multiple phases (see Kerner & Rehborn (1996), Helbing & Tilch (1998), Helbing & Treiber (1999), Lee et al. (1999), Kerner (1999), Lee et al. (2000), Kerner (2004)). Not only the theory was formulated based on simulation results, but also empirical evidence for the theory was reported. For instance, Kerner (2004) investigated the spatiotemporal features of traffic flows on a basis of empirical data from German freeways and proposed three phase traffic theory, free flow, synchronized traffic flow and the wide moving jam. At least in Kerner's empirical findings, two phases exist within congested regime. Furthermore Kerner claimed that there are three main types of synchronized patterns (SPs) in the synchronized traffic flow: Localized SP (LSP), Widening SP (WSP), and Moving SP (MSP). LSP is fixed at the bottleneck and does not continuously propagate upstream over time while WSP also is fixed at the bottleneck but continuously propagate upstream over time. In contrast to the LSP and WSP, an MSP propagates as a whole pattern on the freeway over time. It is suggested by Kerner (2004) that steady states of synchronized flow cover a two-dimensional region in the speed-flow plane. In short, the amount of simulation results and empirical observations forms strong evidence that traffic in congested conditions are in a collection of multiple phases and hence instable.

3.3.2 Travel time uncertainty

In Figure 3.5, the relationship between inflow and travel time uncertainty is *stationary* which does not take the time steps into account. However, traffic breakdown plays an important role in travel time reliability since when the probability of traffic breakdown at current time step under the prevailing circumstances is higher, at a next moment the consequences due to breakdown is much worse (much more unstable travel times). Thus, the instability of traffic flow for a given inflow level is directly associated with the probability of traffic breakdown. Figure 3.9 illustrates the travel time-flow relations before and after traffic breakdown. In the graph, travel times before breakdown are quite certain since traffic flows are in free-flow conditions while travel times after breakdown are uncertain, especially in case high volumes. Accordingly, uncertainty can be categorized into: (a) the travel time uncertainty in case of flows in free flow conditions (before breakdown) and (b) the travel time uncertainty after breakdown (the consequence of the traffic breakdown). Under this circumstance, the relationship between travel time uncertainty and inflow is so-called *dynamic*, which can be represented in Figure 3.10. The empirical data from Figure 3.9 provide evidences that the travel time uncertainty does not sharply increase with the increasing inflows in case of high inflows. This is probably due to the fact that travel times after traffic breakdown depend more on how the queue builds up and propagates upstream rather than on inflow. It can be seen that the travel time-flow relation in stationary situations (Figure 3.4(b)) and the travel time-flow relation in dynamic situations (Figure 3.9) are quite different. Therefore, the associated inflow and travel times should be clearly defined.

In case inflows are low, the congestion due to breakdown may have a short dissolution time. Travel time uncertainty is low. But in case of high inflows, the congestion due to breakdown may have a long dissolution time and hence travel time uncertainty is also

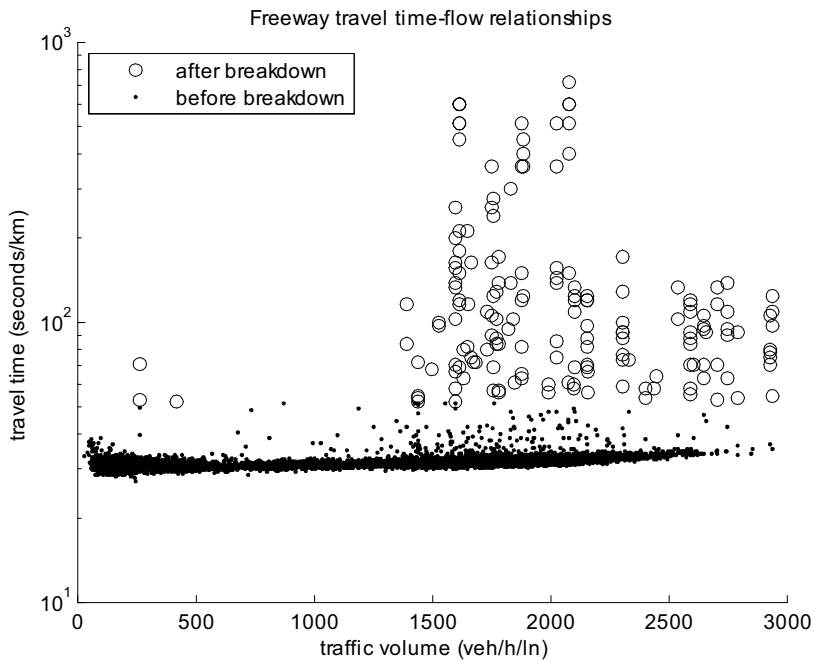


Figure 3.9: Example of empirical relation between travel time-flow before and after traffic breakdown on A20 freeway. This figure is derived from Figure 3.4.

relative high. Then the travel time uncertainty before or after breakdown needs to be defined and quantified. To this end, the associated inflow for a travel time given a certain time step needs to be clarified.

Let us define if, for a given inflow level, (a) *state one*: traffic does not break down, the flows are in *stationary* conditions (the flows are in free flow conditions); (b) *state two*: if traffic does break down, the flows are in *transition* conditions (the flows transits from free flow conditions to congested conditions); and (c) *state three*: after traffic breaks down at current time period, the flows in the next several time periods are in *congested* conditions (the congestion will last for several time intervals until the congestion dissolved).

Figure 3.11 schematically represents these three states to illustrate the correlations between travel times and inflows. Once the flows are in a congested condition, the congested flows will last several time periods until traffic congestion resolves. The congested flows within these time periods mostly are due to the inflow broken down at a previous time period and induced the congested flows. Consequently, the corresponding travel times with respect to inflow can be classified as:

- $TT^s(q_{in})$: Travel times in *stationary* conditions for a given inflow q_{in}
- $TT^t(q_{in})$: Travel times in *transition* conditions which is followed by a traffic breakdown for a given inflow q_{in}

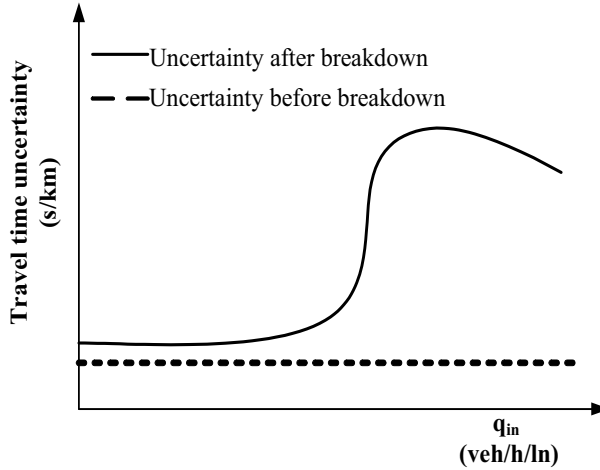


Figure 3.10: Representation of the dynamic relationships between uncertainty and inflow (q_{in}) before and after traffic breakdown (Y-axis: the higher value, the higher uncertainty)

- $TT^j(q_{in})$: Travel times in *congested* conditions for a given inflow q_{in} . Here the associated inflow q_{in} with the travel times under congested conditions (state three) is regarded as the inflow at time period $p - 1$ which induces traffic breakdown in state two (Figure 3.11). This is due to the fact that travel times in state three is caused by the inflow which induces traffic breakdown.

Once the one-to-one associated travel time-inflow relation has been setup, the travel times can be categorized into two types: $TT^f(q_{in})$ (travel times before breakdown), include both stationary travel times $TT^s(q_{in})$ and transitions travel times $TT^t(q_{in})$; $TT^j(q_{in})$ (travel times after breakdown).

As can be seen in Figure 3.9, (a) if travel times are uncertain, travel time variability is large, and (b) if travel times are certain, travel time variability is low. Therefore, travel time variability is regarded as a quantitative indicator of travel time uncertainty. In this dissertation, we use a single indicator as a measure for travel time variability, a so-called statistical range method i.e. the difference between the 90th and 10th percentile travel time, as shown in Eq.(3.10). The idea behind this is twofold. Firstly, the more travel time varied, the more unreliable travel time can be considered. Secondly, percentile travel time is a better performance indicator in terms of travel time reliability than standard deviation or variance does, as Chen et al. (2003) putted: "The 90th percentile travel time is a meaningful way of combining the effect of both average travel time and its variability (standard deviation) into one number." (see further empirical analysis in section 4.5.2).

$$TTV = TT90th - TT10th \quad (3.10)$$

where TTV denotes travel time variability while $TT90th$ and $TT10th$ denote 90th percentile travel time and 10th percentile travel time, respectively.

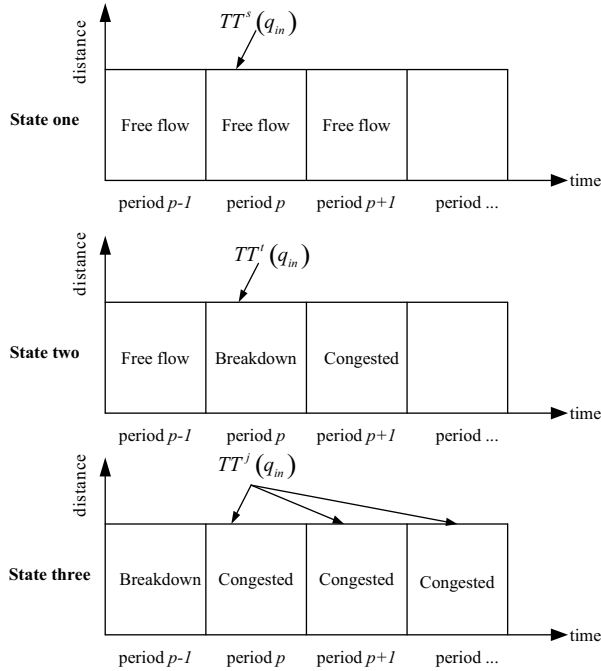


Figure 3.11: Schematic representations of the states of traffic under a given inflow q_{in}

Travel time variability before (include the transition travel times since these travel times are still in free flow conditions) and after breakdown can be defined as Eq. 3.11:

$$\begin{aligned} TTV^f &= TT_{90th} - TT_{10th}, TT \subset TT^f \\ TTV^j &= TT_{90th} - TT_{10th}, TT \subset TT^j \end{aligned} \quad (3.11)$$

in which,

$$\begin{aligned} TTV^f &= \text{Travel time variability in free flow conditions (before breakdown)} \\ TTV^j &= \text{Travel time variability due to transitions (after breakdown)} \\ TT^f &= \text{Travel times in free flow conditions (before breakdown)} \\ TT^j &= \text{Travel times in congested conditions (after breakdown)} \end{aligned}$$

3.4 Conceptual Travel Time Reliability Model

Based on the previous considerations, the travel time unreliability includes two elements: instability and uncertainty. The investigation of travel time unreliability is the process of quantifying these two elements. Consider a driver departing at current time step given a certain inflow level. Whether this driver will experience the traffic breakdown (or congestion) can be seen as a risk. Risk is then characterized by two quantities:

1. the consequence (magnitude) of a possible breakdown, and
2. the probability (likelihood) of occurrence of each breakdown

Consequences are expressed numerically (e.g. travel time uncertainty) and the likelihood of occurrence are expressed as probabilities (e.g. the probability of breakdown). Then the total risk is computed as the sum of the products of the consequences multiplied by corresponding probabilities (Singpurwalla 2006). Thus, travel time unreliability includes the perceived or estimated (non)breakdown probability and the associated travel time variability. The travel time unreliability (i.e. risk) can be represented by the following conceptual travel time (un)reliability model:

$$TTUR(q_{in}^r) = (1 - p_r^{br}(q_{in}^r)) \times TTV^f(q_{in}^r) + p_r^{br}(q_{in}^r) \times TTV^j(q_{in}^r) \quad (3.12)$$

in which

$$\begin{aligned} TTUR(q_{in}^r) &= \text{travel Time UnReliability given a inflow level } q_{in} \text{ on route } r \\ p_r^{br}(q_{in}^r) &= \text{probability of traffic breakdown given a inflow level } q_{in} \text{ on} \\ &\quad \text{route } r \\ TTV^f(q_{in}^r) &= \text{travel time variability before breakdown given a inflow le-} \\ &\quad \text{vel } q_{in} \text{ on route } r \\ TTV^j(q_{in}^r) &= \text{travel time variability after breakdown given a inflow level} \\ &\quad q_{in} \text{ on route } r \end{aligned}$$

Since this reliability model is conceptual which include two elements of travel time reliability, the model is named as CTTR (Conceptual Travel Time Reliability model). The idea in Eq. 3.12 is that travel time unreliability depends not only on the consequence of traffic breakdown, but also on the probability of traffic breakdown. The consequence in this case relates to the variability of travel times conditional to internal and external factors like road geometry, weather conditions, traffic control etc. For instance, the congestion due to breakdown probably lasts a short time period under good weather conditions, but it lasts a long time period under heavy rainy conditions.

3.5 Summary

This dissertation aims to build up a travel time reliability model which can be used in traffic assignment models. To this end, this chapter presented a conceptual travel time reliability model. Firstly, the characteristics of the speed-flow and travel time-flow planes are described on a basis of empirical data and provide evidences that both speed-flow relation and travel time-flow relation are widely scattered. It turns out that travel times are unreliable in case they are either unstable, uncertain or both. Travel times are determined by two critical flows (Critical Transition Flow λ_t and Critical Capacity Flow λ_c). Below λ_t , travel times are certain and stable; between λ_t and λ_c , travel times are uncertain and instable; above λ_c , travel times are relatively certain but highly instable. Therefore, travel

times have three regions: (a) certain and stable travel times, (b) uncertain and instable travel times, and (c) certain and very instable travel times. In this context, travel time unreliability is computed as the sum over the products of the consequences (uncertainty) and corresponding probabilities of breakdown. Traffic breakdown is a stochastic process and the probability of breakdown is an increasing function of inflow. The travel time variability is proposed as the indicator of the travel time uncertainty.

Chapter 4

Inflow-Travel Time Reliability Model

4.1 Introduction

As shown in Eq. 3.1, a generic travel time reliability model is regarded as a function of a variety of factors. In essence, these factors are conditionals, that is, the function expresses travel time reliability for a certain inflow level, *given* certain road characteristics, and *given* all other relevant factors such as traffic control measures, the prevailing traffic state (congested or not), and possibly external factors like weather, luminance, etc. In this chapter, we focus solely on the principal input factor: inflow.

In order to measure the travel time variability for a given inflow level, in section 4.2, we will analyze flow-travel time functions in detail and postulate some hypotheses of the inflow-travel time variability relations. Next, in section 4.3, the inflow-travel time unreliability model is built up. In order to test the inflow-travel time variability relations and the inflow-travel time unreliability model, an experiment will be setup in section 4.4. Thereafter, we analyze some empirical data to define and evaluate the inflow-travel time reliability model in section 4.5. In section 4.6, the validity of travel time reliability model will be discussed and in section 4.7 the effects of the measurement time interval on reliability analysis will be addressed. This chapter closes with some important conclusions and related implications in section 4.8.

4.2 Inflow-Travel Time Variability

4.2.1 Existing flow-travel time functions

The travel time functions describe how long it takes for vehicles that enter link k or route r to reach the end of link k or route r . A nonexhaustive overview of travel time functions proposed in the past for traffic assignment models is listed in Table 4.1. A dated review of travel time functions is given in (Branston 1976).

Table 4.1: Overview of flow-travel time functions

Name	Functions *)
Irwin <i>et al.</i> (1961)	$t_a(v_a) = \begin{cases} t_a^f + \alpha_a v_a, & v_a < C_a \\ t_a^f + \beta_a v_a + (\alpha_a - \beta_a) C_a, & v_a \geq C_a \end{cases}$
Davidson (1966)	$t_a(v_a) = t_a^f \left(1 + \alpha_a \frac{v_a/C_a}{1 - v_a/C_a} \right)$
Akcelik (1991)	$t_a(v_a) = \begin{cases} t_a^f \left(1 + \alpha_a \frac{v_a/C_a}{1 - v_a/C_a} \right), & v_a < \rho C_a \\ t_a^f \left(1 + \alpha_a \frac{\rho}{1 - \rho} \right) + \frac{\alpha_a}{(1 - \rho)^2} \left(\frac{v_a}{C_a} - \rho \right), & v_a \geq \rho C_a \end{cases}$
BPR (1964)	$t_a(v_a) = t_a^f \left(1 + \alpha_a \left(\frac{v_a}{C_a} \right)^{\beta_a} \right)$
Smock (1962)	$t_a(v_a) = t_a^f \exp \left(\frac{v_a}{C_a} \right)$
Soltman (1965)	$t_a(v_a) = t_a^f 2^{v_a/C_a}$
Overgaard (1967)	$t_a(v_a) = t_a^f \alpha_a \left(\frac{v_a}{C_a} \right)^{\beta_a}$
Mosher(1) (1963)	$t_a(v_a) = t_a^f + \ln(C_a) - \ln(C_a - v_a)$
Mosher(2) (1963)	$t_a(v_a) = \alpha_a - \frac{C_a(t_a^f - \alpha_a)}{v_a - C_a}$

*) t_a = travel time on link a , t_a^f = free-flow travel time on link a , v_a = flow on link a , C_a = capacity of link a . All other unknowns are nonnegative parameters of the functions, mostly link type-specific.

One of the earliest travel time functions to be used was the model proposed by (Irwin et al. 1961). The travel time-flow linear relationships are relatively simple. Some studies proposed the curvilinear travel time-flow relationship such as (Smock 1962) (Soltman 1965) (Overgaard 1967). Meanwhile, Mosher (1963) suggested two approaches for the travel time functions, namely the logarithmic, and hyperbolic functions, and Davidson (1966) and Akcelik (1991) presented two non-linear travel time functions for urban road. One of the best known and most widely used travel time-flow function to have been developed is that often referred to as the BPR function, (Bureau of Public Roads 1964). Most of the proposed travel time-flow functions are monotonically increasing functions of the flow, also for flows larger than the capacity C_a . The above linear and nonlinear travel time functions (Table 4.1), however, do not produce very realistic travel time curves because the travel times given a certain inflow vary with different circumstance (see also Figure

3.4(b)). In this study, on a basis of empirical dataset, the (route-level) inflow- percentile travel time curve (function) will be investigated.

4.2.2 Inflow - percentile travel time functions

Before addressing flow-travel time relationship, there is a need to clarify what the traffic flow is. As mentioned in section 3.1, inflow is proposed as the indicator of traffic flow. The idea behind this is twofold. Firstly, the inflow q_{in} is the assigned flow in each step of the traffic assignment model; Secondly, the inflow q_{in} is a local measurement so that it can be easily collected in reality. Thus, in this and the remaining chapters flow is proposed as a inflow (q_{in}) which exclusively denotes vehicles entering the studied freeway section or corridor in vehicles per hour per lane.

Furthermore, attention should be paid to the definition of inflow since it is based on the volume of vehicles that pass over a point for a particular time interval and the size of this measurement interval affects reliability analysis. In order to integrate variability in travel times and instability (probability of traffic breakdown), in this dissertation, a balance of measurement interval is taken between a very small value (e.g. 1 minute) and a very large value (e.g. 1 hour) (see section 4.7). Thus, a inflow measurement interval is proposed: *Inflows (q_{in}) are measured on 10-minute aggregate. This interval duration has also been applied in the probability of traffic breakdown.*

Clearly Figure 3.4 provides evidences that the flows have three regions which is determined by two critical inflow levels (section level). Several successive sections are combined on a route. As far as the speed-flow plane on a route-level basis is concerned, this point essentially leads to the following issue: are there also two critical flows in the journey speed-flow relation? To illustrate this, Figure 4.1 shows the journey (route) speed probability density function (dark area depict high probability density) on the A12 freeway in the year of 2004 between 6:00AM and 20:00PM with 10-minute aggregate data in The Netherlands. Note that these are not local or section speeds, but journey speeds (the length of this route is 17.28 km). Therefore, traffic flows on a route level also have three regions:

1. Region: $q_{in} < \lambda_t$. In this region the probability that traffic along an entire route mostly is a free flow state, in which only high speeds occur. Although there are some data points in the fluent traffic region (Figure 4.1, left bottom part), the probability that traffic is a congested state is very low (less than 2%). In this region, travel speeds are certain and stable.
2. Region: $\lambda_t \leq q_{in} \leq \lambda_c$. In this region different traffic states along a route can exist, either free flow, synchronized (given a certain flow, speed varies from a low value to a high value) or congested. The transition traffic region here may consist of data from both synchronized and free flow. As can be seen in Figure 4.1, the transition region covers both free flow traffic states (the top part of the region) as well as "synchronized flow" (lower part) areas. Therefore, travel speeds are uncertain and instable.

3. Region: $q_{in} > \lambda_c$. In the capacity traffic region, both speeds and flows are relatively high. Although there are some data points in the fluent traffic region (Figure 4.1, right middle part), the probability that traffic is in congested conditions is low (less than 2%). The travel speeds are relative certain. However, the probability of traffic breakdown inside this region is high and thereby the flows are instable.

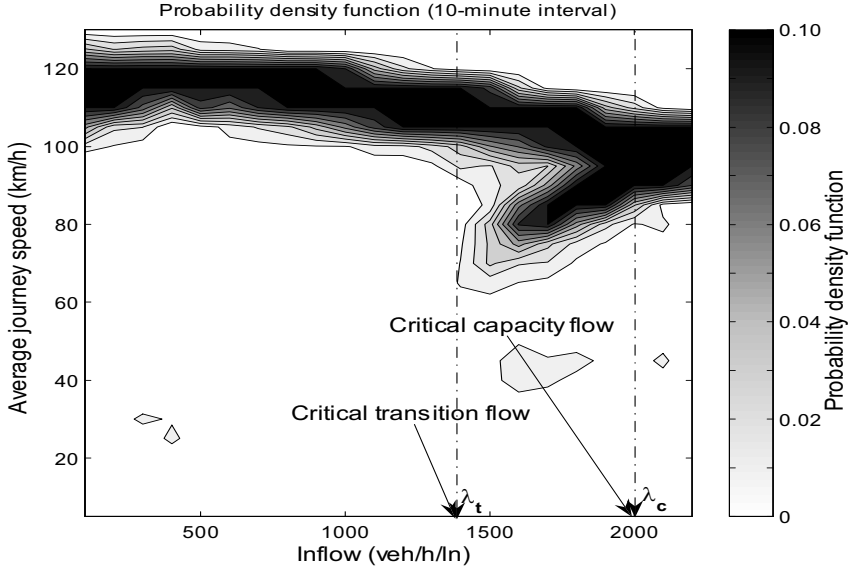


Figure 4.1: Journey (route) speed probability density function as a function of inflow in a 17.28 km corridor on A12 freeway (2004, between 6:00AM and 20:00PM, 10-minute aggregate). Dark areas depict high probability density.

Consequently, the inflow-travel time (static) relation highly relies on these two critical inflow levels. Therefore, an alternative way of parameterizing the inflow-percentile travel time functions is as follows:

$$TT_{XX} = \begin{cases} f_1(q_{in}) = t_{XX}^f \left(1 + \alpha_1 \left(\frac{q_{in}}{\lambda_t} \right)^{\beta_1} \right), & q_{in} \leq \lambda_c \\ f_2(q_{in}) = t_{XX}^f \left(\alpha_2 \left(\frac{q_{in} - \lambda_c}{\lambda_c} \right) + \gamma_1 \right), & q_{in} \geq \lambda_c \end{cases} \quad (4.1)$$

subject to

$$f_1(q_{in}) \Big|_{q_{in}=\lambda_c} = f_2(q_{in}) \Big|_{q_{in}=\lambda_c} \quad (4.2)$$

$$\begin{aligned} 0 &< \alpha_1 < 1 \\ \beta_1 &> 1 \\ \alpha_2 &< 0 \end{aligned}$$

where $f_1(q_{in})$, percentile travel time, is a function of inflow for inflows below λ_c ; $f_2(q_{in})$, percentile travel time, is a function of inflow for inflows above λ_c ; and parameter γ_1 can be derived from combining Eq. 4.1 and Eq. 4.2,

$$\gamma_1 = 1 + \alpha_1 \left(\frac{\lambda_c}{\lambda_t} \right)^{\beta_1}$$

in which q_{in} = inflows; t_{xx}^f = free flow travel time (the length of road unit over the speed limits long the route); TT_{xx} , denotes percentile travel time, such as $TT10th$, $TT50th$, $TT90th$ are 10th, 50th, 90th percentile travel time, respectively. λ_t and λ_c are two critical flows governing the scale of travel time functions, and α_i and β_i are parameters governing the shape of the relationship, which (like λ_t and λ_c) need to be estimated from data.

Note that our primary interest is not to derive an alternative BPR functions (although this is a useful side effect), but to gain more insights into the inflow-travel time reliability relationships, and particularly whether these critical inflow levels explain its relationship. All parameters can be estimated by fitting the estimated travel time TT to the observed travel time t by minimizing the squared errors, as shown in Eq. 4.3.

$$\min \sum_{i=1}^n (t_{xx}^i - TT_{xx}^i)^2 \quad (4.3)$$

where

$$\begin{aligned} t_{xx}^i &= \text{the } i^{\text{th}} \text{ observed } XX \text{ percentile travel time} \\ TT_{xx}^i &= \text{the } i^{\text{th}} \text{ estimated } XX \text{ percentile travel time} \\ n &= \text{number of flow bins (flow interval)} \end{aligned}$$

To estimate the parameters, we apply a following procedure to minimize the square error between estimated travel times and observed travel times using the **Least Square Error Method**. The procedure can be described as follows:

Algorithm 1 Parameter of inflow-travel time model estimator

1. Data preparation

Step 0: Divide the inflow-travel times into discrete inflow bins (e.g. inflow interval, $\Delta q = 60$) and determine the percentiles (of travel time) for all these separate inflow bins using the observations (percentile travel time t_{xx}^i under an inflow bin q_{in}^i ($q_{in}^i = i \times \Delta q$)) in which $i = 1, 2, \dots, n$; n : number of inflow bins).

2. Determination of λ_c

Step 1: Determine λ_c by finding the maximum travel time percentile t_{xx}^m ($m \leq n$) of all inflow bins and set λ_c equals the corresponding flow q_{in}^m ; Initial $k=1$;

3. Determination of λ_t , α_1 , α_2 , and β_1 using an iterative procedure described below:

Step 2: Determine α_1 , α_2 , and β_1 by minimizing the sum of square errors between t_{xx}^i and TT_{xx}^i for given values of λ_t and λ_c . Parameter λ_c is equal to the one found in step 1, while parameters $\lambda_t = k \cdot \Delta q$ store the Square Errors $SE(k)$ ($SE(k) = (t_{xx}^k - TT_{xx}^k)^2$);

Step 3: Set $k = k + 1$. If $k < m$, go to Step 2, else, go to Step 4.

Step 4: Find the minimum value SE_{\min} in the vector SE and the corresponding k equals k_{\min} , in which set $\lambda_t = k_{\min} \cdot \Delta q$.

Here the inflow interval Δq affects the precision of the estimated parameters of Eq. 4.1 in **Algorithm 1**. The larger numbers of travel times in each inflow bin, the more sure you can be that the samples reflect the real travel times for that inflow bin. The sample size can be calculated (Robertson 1994):

$$ss = \frac{Z^2 \times p \times (1 - p)}{c^2} \quad (4.4)$$

where:

- ss = Sample Size
- Z = Z value (e.g. 1.96 for 95% confidence level)
- p = percentage picking a choice, expressed as decimal (0.5 used for sample size needed)
- c = confidence interval, expressed as decimal (e.g., 0.02)

In this study, 95% confidence level and 2% confidence interval was chosen to determine the sample size. Therefore, the sample size for each inflow bin should be large than 1200. If one year traffic data (365 days, 52560 records) is used to estimate Eq. 4.1, the total number of inflow bins should be lower than 43. Thus, the inflow bin size should be large than 56 veh/h/ln if the freeway capacity is 2400 veh/h/ln. Here we choose inflow bin size $\Delta q = 60$.

4.3 Inflow Travel Time Unreliability Model

As discussed in Section 2.2.4, the travel time reliability is a function of the travel time variability and the instability of traffic flow. Once we have the inflow-percentile travel time functions, travel time variability can be depicted as Eq. 4.5 (see also Eq. 3.10). And, the travel time variability is determined by two critical inflows λ_t and λ_c (see Eq. 4.1).

$$TTV(q_{in}) = TT90th(q_{in}) - TT10th(q_{in}) \quad (4.5)$$

in which,

$$\begin{aligned}
TTV(q_{in}) &= \text{Travel time variability for a given inflow } q_{in} \\
TT90th(q_{in}) &= 90th \text{ percentile travel time for a given inflow } q_{in} \\
TT10th(q_{in}) &= 10th \text{ percentile travel time for a given inflow } q_{in}
\end{aligned}$$

In this chapter it is proposed that the conceptual travel time (un)reliability model (CTTR) (Eq. 3.12) can be formulated as:

$$\begin{aligned}
TTUR(q_{in}) = & (TT90th^f(q_{in}) - TT10th^f(q_{in})) \times (1 - p_r^{br}(q_{in})) \\
& + (TT90th^j(q_{in}) - TT10th^j(q_{in})) \times p_r^{br}(q_{in})
\end{aligned} \quad (4.6)$$

in which,

$$\begin{aligned}
TTUR(q_{in}) &= \text{travel time unreliability for a given inflow } q_{in} \\
TT90th^f(q_{in}) &= 90th \text{ percentile travel time in free flow conditions for a} \\
& \quad \text{given inflow } q_{in} \\
TT10th^f(q_{in}) &= 10th \text{ percentile travel time in free flow conditions for a} \\
& \quad \text{given inflow } q_{in} \\
TT90th^j(q_{in}) &= 90th \text{ percentile travel time in congested conditions for} \\
& \quad \text{a given inflow } q_{in} \\
TT10th^j(q_{in}) &= 10th \text{ percentile travel time in congested conditions for} \\
& \quad \text{a given inflow } q_{in} \\
p_r^{br}(q_{in}) &= \text{the probability of traffic breakdown on route } r \text{ for a given} \\
& \quad \text{inflow } q_{in}
\end{aligned}$$

4.4 Experimental Setup

In order to investigate the above travel time variability and travel time reliability model from the empirical data, this section presents a general framework for empirically setting up a travel time reliability model. As shown in Figure 4.2, there are several key elements such as data sources, data cleaning, historical database etc. which are introduced in the following subsections.

4.4.1 Traffic data collection system and other data sources

The sources of traffic data collection system are among others:

- Roadside detection
 - *Induction loop detectors*, which detect vehicles entering a created electromagnetic field by induction of Foucault currents. With two induction loops placed closely together (commonly 1 meter apart) not only the vehicle but also its speed can be detected. Usually gathered information: aggregated flows and aggregated speeds in one minute intervals. Vehicle types can be obtained by induction patterns.

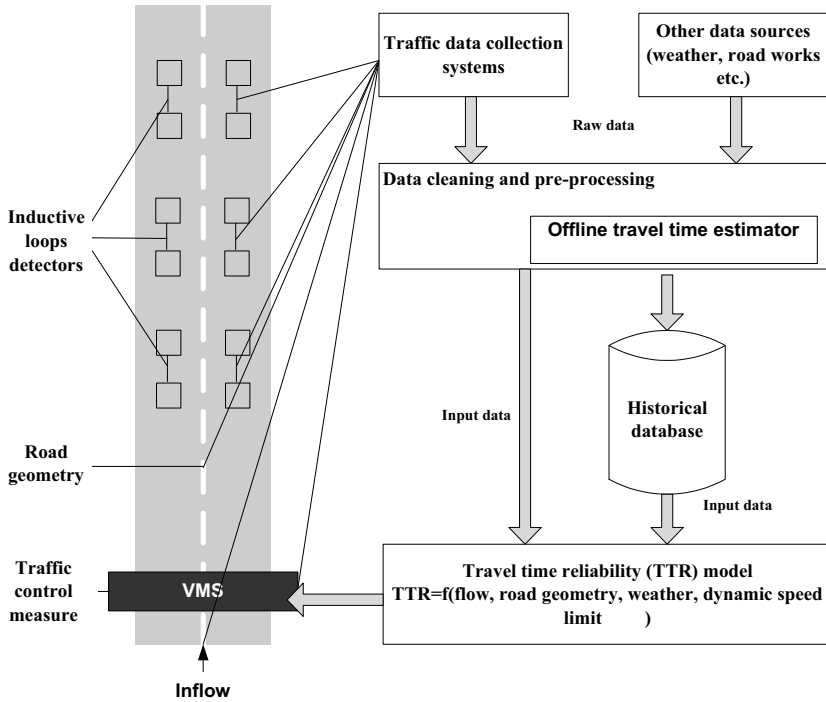


Figure 4.2: General framework for empirical travel time reliability model setup

- – *Infrared detectors*, which detect passing vehicles when a beam of light is interrupted. Active infrared detectors are additionally able to recognize temperature differences (engine heat, body warms). Usually gathered information: aggregated flows and aggregated speeds in one minute interval.
- *Radar detectors*, which measure the presence and the speed of vehicles using the Doppler Effect. Usually gathered information: aggregated flows and aggregated speeds in one minute interval. Further they can measure the height of the passing vehicle.
- *Ultrasonic detectors*, which transmit ultrasonic sound waves instead of electromagnetic radar waves. Usually gathered information: aggregated flows in one minute intervals plus a record of vehicle types, distinguished by their height.
- *Video cameras*, which detect vehicles when entering and existing a road stretch. Usually gathered information: aggregated flows and aggregated speeds in one minute intervals plus individual travel time data if a license plate recognition is used.
- Floating car data
 - *Probe vehicles* transmitting traffic messages containing location, speed, and others at regular time intervals.

- *GSM and GPS* data, which is recently used to gather travel time data of the road network, for instance (Faghri & Hamad 2002).

As can be seen in Eq. 4.6, only the travel times and inflow data are needed in order to build up the inflow-travel time reliability model. Therefore, the traffic data from above all traffic monitoring systems can be used to develop such a reliability model. Our analysis is based on the *induction loop detectors* data obtained from the Regiolab-Delft traffic monitoring system (van Zuylen & Muller (2002), van Lint (2004), Muller et al. (2005)). This dissertation builds up travel time reliability modeling framework on a typical example of such a traffic data collection system, the MONItoring CAasco (MONICA) system operation on the larger part of the Dutch Highway Network maintained by the Dutch Ministry of Transport, Public Works and Water Management (for details, see (van Lint 2004)).

In Figure 4.2 two data sources are obtained to build up travel time reliability model: traffic data collection systems, which measure the actual traffic conditions and data collection systems measuring "ambient factors" influencing these traffic conditions, such as weather, road works, etc. A more specific taxonomy of data source is given below.

Traffic flow Volume and flow rate are two measures that quantify the amount of traffic passing a point on a lane or roadway during a given time interval. The volume is the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; volumes can be expressed in terms of annual, daily, hourly, or sub hourly periods. The flow rate is the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 hour, usually 5~15 minutes. In this dissertation, flow rate is used as the measure of traffic flow.

Speed Speed is defined as a rate of motion expressed as distance per unit of time, generally as kilometers per hour (km/h). Several different speed parameters can be applied to a traffic stream, such as space mean speed and time mean speed (TRB 2000). Space mean speed is a statistical term denoting an average speed based on the average travel time of vehicles to traverse a segment of roadway while time mean speed is the arithmetic average of speeds of vehicles observed passing a point on a roadway, also referred to as the average spot speed. In this research, the speed data is only obtained from the inductive loop detectors and thereby time mean speed is collected.

Road geometry The characteristics of freeway, such as number of lanes, location of on-ramps, location of off-ramps, location of weaving sections, length of deceleration or acceleration lanes weaving length etc. are registered in Regiolab-Delft traffic monitoring system. Furthermore, the location of inductive loop detectors have also been registered in Regiolab-Delft traffic monitoring system.

Traffic control measure Freeway traffic control management is the implementation of strategies to improve freeway performance, especially when the number of vehicles desiring to use a portion of the freeway at a particular time exceeds its capacity. Dynamic speed limits, static speed limits, and ramp metering information are also obtained from traffic data collection systems.

Weather condition The weather data are collected from 37 weather stations of the Dutch Meteorological Institute KNMI spread across The Netherlands. The weather measurement took places each hour during one year which measured several weather variables, such as rain, fog, ice, storm, snow etc.

4.4.2 Data cleaning and offline travel time estimation tool

The input data in a real-time situation, collected by a real-time traffic monitoring systems, will often have corrupted or missing values (e.g. on average 15% of the inductive loops of the Dutch freeway monitoring system (MONICA) may be out of operation or producing unreliable measurement) (van Lint 2004). This happens when a measurement device produces data that is (either by the modeler or the device itself) dubbed unreliable, or when it produces no data at all. There are three types of detection failure: incidental failure due to, for example, temporal power or communication failures; structural failure due to physical damage or maintenance backlogs to the inductive loops or roadside equipment; intrinsic failure, measurement noise and bias, is inherent to detection devices and averaging measurements over time in general. Consequently, data cleaning is imperative to correct and/or replace missing or corrupt data. In general, such a data cleaning module performs three tasks:

1. **Data Checking:** before possible problems (e.g. missing data) can be adequately tackled, they need to be identified first.
2. **Data Completion:** filling the possible gaps in the data with a reasonable replacement.
3. **Data Correction:** recheck the now complete data set for validity and consistency and replace/adjust data if required.

Various approaches can be undertaken to tackle the missing or unreliable data: Null replacement, Simple imputation, Model based imputation, Multiple imputation (see Appendix C). In this study, the Simple imputation is used to correct the unreliable data.

Since there are no actual travel times measured in Regiolab-Delft monitoring systems, one needs to resort to tools that can convert local measurements (e.g. flow rate, speed) into travel times in order to build up the travel time reliability model. The estimation of travel times on freeways has been studied intensively in the field of transportation engineering. Generally travel time estimation refers to the reconstruction of the travel times for historical trips. In this dissertation, we use the so-called *Piece-wise Linear Speed Based Algorithm* (PLSB) to estimate travel times on routes of adjacent freeway sections (van Lint & van der Zijpp 2003). The PLSB method reconstructs vehicle trajectories and hence mean travel times based on time series of speed and volume measurements on consecutive detector locations along a route (see Appendix B). The combination of PLSB algorithm and a simple imputation method provide a robust framework for offline travel time estimation (van Lint 2004).

4.4.3 Historical database

The historical database is a collection of past observations of the traffic system under consideration. Given the offline estimated travel times, measurements from a traffic data collection system along the routes and a set of data from other sources (weather etc.), a large scale historical database was built for a large number of departure time periods.

In such a database, for each freeway route and time period a record was obtained containing the offline estimated travel time for vehicles departing in that time period and measurements from all connected traffic data collection systems and other data sources (such as inflows, speed etc.) in the same time period. The travel time reliability model was estimated from the historical database.

4.5 Empirical Analysis and Results

4.5.1 Test case description

In this section, we describe the data for the calibration and validation of the travel time reliability model. We also collect the traffic data from Beijing urban freeway for the construct validity of the reliability model.

Traffic data for travel time reliability model setup

We have collected data from 6 freeway stretches from a network of freeways, provincial roads and an urban network in the South-West of The Netherlands, which includes two of the major Dutch cities, The Hague and Rotterdam (see Figure 4.3). Table 4.2 overviews the 6 freeway corridors and their associated on- and off-ramps, and weaving sections.

Table 4.2: Overview of freeway corridors in the Regiolab-Delft

Code	Freeway	Direction	Number of ramps*) per ten kilometers	Route length (meter)
1201	A12	East	4.1	17280
1211	A12	West	7.1	15520
1301	A13	Southeast	8.5	10590
1501	A15	East	7.2	9725
2001	A20	Northeast	6.7	10500
2011	A20	Southwest	7.0	17215

*) ramps include on-ramps, off-ramps, and weaving sections. One weaving section is two ramps (one on-ramp and one off-ramp)

We briefly outline the main features of the 6 freeway corridors in Regiolab-Delft:

1201: the east direction of A12 freeway with a length of 17.28 km contains four on- and three off-ramps. This stretch is between The Hague and Gouda.

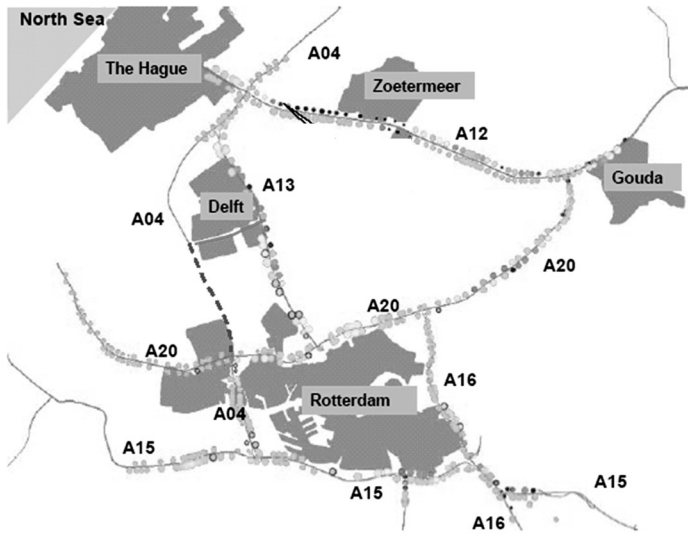


Figure 4.3: Regiolab-Delft traffic monitoring map

1211: the west direction of A12 freeway with a length of 15.52 km contains six on- and five off-ramps. This stretch is between Gouda and The Hague.

1301: the southeast direction of A13 freeway with a length of 10.59 km contains two on- and three off-ramps, and two weaving sections. This stretch is between The Hague and Rotterdam.

1501: the east direction of A15 freeway with a length of 9.73 km contains two on- and one off-ramps, and two weaving sections. The Harbour of Rotterdam is connected to the hinterland by waterways, railways, pipelines and one freeway, the A15 freeway. This stretch links the Harbour of Rotterdam and Rotterdam city.

2001: the northeast direction of A20 freeway with a length of 10.50 km contains four on- and three off-ramps. This stretch is between Rotterdam city and Gouda.

2011: the southwest direction of A20 freeway with a length of 17.22 km contains five on- and five off-ramps, and one weaving section. This stretch is between Gouda and Rotterdam city.

Traffic data for validation

The data used to build up the travel time reliability model were measured in the year of 2004. Traffic data in the year of 2005 on A12 freeway are collected to validate the travel time reliability model. Note that, only the principal factor, inflow, will be validated due to the insufficient data for the other factors (e.g. adverse weather conditions, etc.).

Traffic data from Beijing urban freeways

Beijing has an area of 16,807 square kilometers and a population of more than 13 million permanent residents and five million temporary and seasonal workers. The road network for Beijing central districts expands from six rings. Traffic congestion and air pollution have been the two major problems facing Beijing for decades and are the two central concerns for the successful execution of the 2008 Olympics. In a recent announcement the Beijing Municipality claims that travel time between hotels and major sports locations of the Beijing Olympic Game 2008 should be within 30 minutes (Beijing Municipality 2005). So, travel time reliability as a performance indicator of mobility has entered the Beijing political arena. For the application of the proposed travel time reliability model, detailed real-time traffic data is collected from a mixed freeway and urban network in the North of Beijing city (Beijing Olympic area). The data for this study come from inductive loops detectors on a 7.1 km corridor of the second ring of Beijing from the Dongzhimen Bridge to the Xizhimen Bridge (from east to west), as shown in Figure 4.4. The data consists of 10-minute aggregate time-average speed and flow observations, measured within three months in 2006 (between September 1st and November 30th).

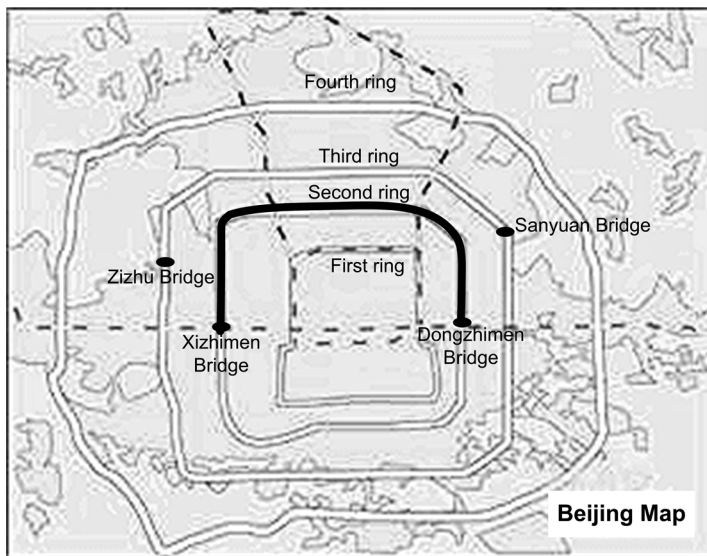


Figure 4.4: Map of Beijing traffic monitoring system. The solid black line represents the road stretch for which the traffic data was analyzed.

The data obtained from Beijing urban freeways will often have corrupted or missing values. The *Simple Imputation* method (see Appendix C) is used to replace the missing values by using ad-hoc (statistical) procedures.

Since actual travel times are not available in Beijing case, all travel times used in this study are estimated with the 'Piecewise Linear Speed Based' (PLSB) travel time estimation model (van Lint & van der Zijpp 2003) (see also Appendix B).

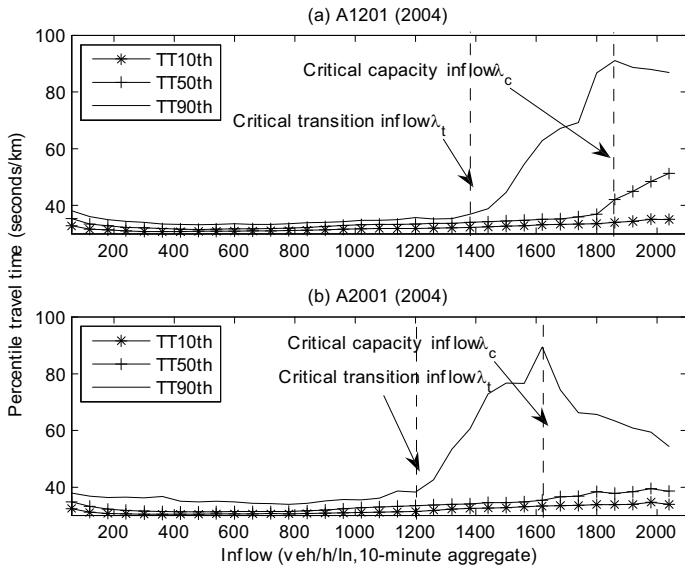


Figure 4.5: Percentile travel time as a function of inflow level in Regiolab-Delft on both (a) A12 and (b) A20 freeways in The Netherlands (10-minute aggregate, 2004)

4.5.2 Results

Before we illustrate the results of travel time unreliability as a function of inflows, the findings from a static travel time variability-inflow analysis will be discussed to provide evidences that reliability measures based on variability tell only half the story.

Variability in travel times

Figure 4.5 illustrates the empirical relationship of 10th, 50th, and 90th percentile travel time as function of inflow on both A12 and A20 freeways. Clearly, the empirical data provide evidence for the existence of such critical inflow values.

In Figure 4.5, the 90th percentile travel time sharply rises when inflow is between λ_t and λ_c . The differences in critical inflow levels (Table 4.3) are probably due to two main causes. One cause may be the number of ramps per unit road length which has a significant influence on the road capacity variation (Tu, van Lint & van Zuylen 2006b) (Tu, van Lint & van Zuylen 2007c). The number of ramps per unit road length on the A20 is 7.0 per ten kilometers which is much larger than on the A12 (4.1 per ten kilometers). Another candidate explanation is the difference between the A12 and A20 in composition of vehicles. The percentage of trucks on the A20 is higher than on the A12 due to its role as main entry route from the North of the Netherlands to the Port of Rotterdam. Note that in both cases, critical transition inflow levels are far lower than capacity, which on Dutch two and three lanes freeways, varies between 2000 and 2200 veh/h/ln.

Table 4.3: Estimated critical inflow parameters for TT90

code	λ_t (critical transition inflow)	λ_c (critical capacity inflow)
A1201	1380 veh/h/ln	1800 veh/h/ln
A2001	1200 veh/h/ln	1620 veh/h/ln

Clearly, the fact that at volumes above λ_c travel time unreliability decreases with the rising inflows is not taken into account if the terms variability and unreliability are used interchangeable. Because in this region, although travel time variability is low, the probability of traffic breakdown is very high and hence traffic flows are very unstable. In this subsection, travel time unreliability which include both the variability and instability will be tested using the empirical dataset from Regiolab-Delft traffic monitoring systems.

Travel time (un)reliability

Probability of traffic breakdown

Figure 4.6 demonstrates the probability of traffic breakdown as a function of inflow levels on two freeway corridors in the Netherlands. Clearly the probability of traffic breakdown increases with rising inflows. It is also seen that there is a certain critical inflow value (A1201: about 1500 veh/h/ln and A1211: about 840 veh/h/ln): above the critical inflow, the probability of traffic breakdown is much higher than that below the critical inflow.

Travel time uncertainty (variability)

As aforementioned, the *static* relations between travel time variability and inflows were investigated. The *so-called static* relations mean that the travel times for a given inflow level comprise the travel times under free flow conditions and the travel times after breakdown. Here we distinguish these two types of travel times and investigate the *dynamic* relations between travel time variability and inflows. That is, the relation between travel time variability before breakdown (TTV^f) and inflows and the relation between travel time variability after breakdown (TTV^j) and inflows are discussed, respectively. Figure 4.7 demonstrates the dynamic relations between travel time variability and inflows on two freeway corridors in the Netherlands. The graph clearly shows that the inflow has limited or no impacts on TTV^f . However, TTV^j under high inflow levels are much larger than that under low inflow levels.

Travel time unreliability

Travel time unreliability model is proposed as Eq. 4.6, which is the result of travel time uncertainty multiplied by the probability of traffic breakdown. Figure 4.8 depicts the empirical relationship of travel time unreliability as a function of inflows on six freeways corridors. As can be seen in the graph, the travel time unreliability as a function of inflow has a similar trend as the probability of traffic breakdown (compared with Figure 4.6) Travel time unreliability increases with the rising inflows. The free flow travel time is 33.3 seconds/km (if speed limit is 120km/h). A check of the travel time unreliability (Figure

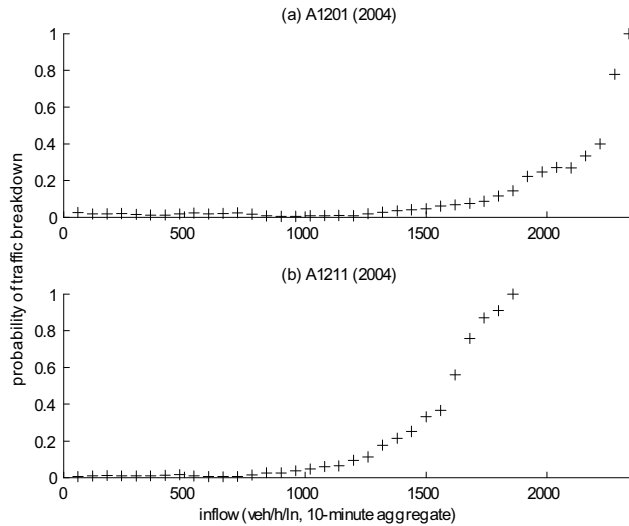


Figure 4.6: Probability of traffic breakdown as a function of inflow levels on two freeway corridors in the Netherlands

4.8 left upper figure: A1201) shows that the variability in travel times ($TT_{90th} - TT_{10th}$, 6.6 seconds/km) is 20% of the free flow travel time (30seconds/km) in case inflow below 1500 veh/h/ln; the variability in travel times (13 seconds/km) is about 40% of the free flow travel time when inflow is 1800 veh/h/ln; but the variability in travel times (100 seconds/km) is 300% of the free flow travel time when inflow is 2400 veh/h/ln.

4.5.3 Fitted travel time reliability function (TLZ reliability function)

In the graph (Figure 4.8) the travel time unreliability curve sharply rises when inflow is above a certain threshold value and hence travel times become more and more unreliable. An approximate travel time reliability function (so-called TLZ (Tu , van Lint, van Zuylen) reliability function) which fitted the measured data is as follows:

$$TTUR(q_{in}) = TTUR_0 \left(1 + \beta \left(\frac{q_{in}}{\lambda_{ttr}} \right)^\gamma \right) \quad (4.7)$$

where,

- $TTUR(q_{in})$ = fitted travel time unreliability for a given inflow q_{in}
- q_{in} = inflow
- λ_{ttr} = critical travel time unreliability inflow
- $TTUR_0$ = free flow travel time unreliability
- β, γ = parameters

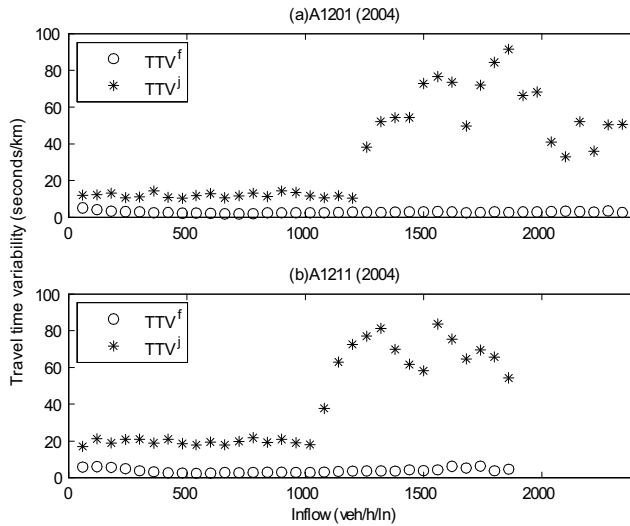


Figure 4.7: Travel time variability under free flow conditions and under congested flow conditions as a function of inflow levels on A12 freeway corridors

Below critical travel time unreliability inflow λ_{ltr} , the travel time unreliability is very small and the travel times are reliable. Above λ_{ltr} , the travel time unreliability increases with the increasing inflows. The differences of threshold inflows (see Table 4.4) are probably due to the differences in the number of ramps per unit road length (see Table 4.2) which might have a significant influence on the travel time unreliability.

Table 4.4: Estimated parameters for TLZ Reliability function

code	freeway	λ_{ltr}	MAREs*
1201	A12	1440	10%
1211	A12	1020	15%
1301	A13	900	15%
1501	A15	1020	12%
2001	A20	1380	22%
2011	A20	1260	30%

*) : Mean Absolute Relative Errors: $MAREs = 100 \frac{1}{N} \sum \left| \frac{e_n}{t_n} \right|$

N= number of data points; e_n = the differences between measured and fitted travel time unreliability; t_n = fitted travel time unreliability

On the basis of the available empirical dataset, travel time unreliability as a function of inflow was obtained (or estimated) from Eq.4.6, after which the function (TLZ function) as Eq.4.7 was fitted to the data. In this TLZ function, one does not need to estimate the

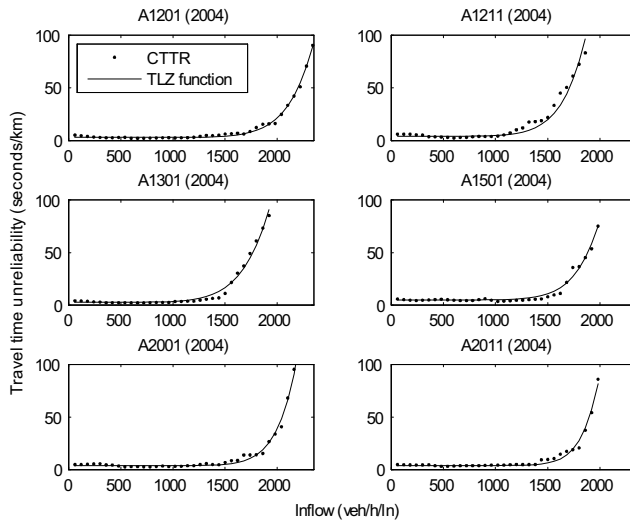


Figure 4.8: Travel time unreliability as a function of inflow level on six freeway corridors in The Netherlands (10-minute aggregate, 2004). CTTR: the Conceptual Travel Time Reliability model (Eq.3.12); TLZ function: travel time reliability function developed by Tu, van Lint and van Zuylen Eq. 4.7.

distribution of the probability of traffic breakdown and travel time variability. Therefore, the TLZ function can be easily applied in Dynamic Traffic Assignments.

4.6 Validity of Travel Time Reliability Model

A model has to be made trustworthy, i.e. the results of a model should have sufficient credibility. This credibility is called the *validity* of the model. The *validity* of a simulation model can be seen at different levels (van Zuylen et al. 2007):

- Content validity (does the model have parameters that are logically related to the characteristics of the system). In chapter 3, the empirical characteristics of travel time-flow relation demonstrate that travel time reliability include two elements: instability of travel times and uncertainty in travel times while travel times are the result of traffic flow operations which in turn are governed by multiple factors like traffic flow, road geometry etc. Therefore, the proposed conceptual travel time reliability model, a function of multiple factors, is logically related to the characteristics of travel time reliability.
- Construct validity (calibration). That is, the adjustment of the model parameters to improve the model's ability to reproduce the observed performance of the system as closely as possible. The calibration is done by observing the system, by measuring certain aspects of its performance, and then adjusting the model such that

these aspects are well represented. Afterwards one may assume that the model is calibrated.

- Predictive validity (validation). The predictions made with the model should be consistent with the observed evolution of the system. It is about two processes, the estimated and the observed, and checking how one approximates the other. New observations are made of the system behavior and the model is used to imitate the system behavior again and without further adjustment of the model parameters. If the results are sufficiently similar to the observed behavior and the error is within certain margin, the model is able to predict the performance of the system.

In this section, the proposed travel time reliability model will be calibrated and validated using the empirical traffic data.

4.6.1 Construct validity

In section 4.5, the travel time reliability model has been calibrated on the traffic data on the Dutch freeways. The proposed model will be calibrated on the traffic data on Beijing urban freeways as well. Before calibration, two items (the speed threshold to determine traffic breakdown and the inflow interval) should be re-defined. Firstly, for the Dutch freeway, a reduction of average speed on a section within one time interval from a high level down below to a threshold of 70km/h is treated as a breakdown. Figure 4.9 shows the speed-flow relations on Beijing second ring urban freeway. In this freeway, the speed limit is 60 km/h. As can be seen in the graph, traffic below 40km/h can be treated as congested conditions. Therefore, traffic breakdown on Beijing second ring urban freeway can be proposed as:

A reduction of average speed on a section within one time interval from a high level down below to a threshold of 40 km/h is treated as a section breakdown. Route traffic breakdown: traffic breakdown of a route occurs in case at least one section along the route breaks down.

Secondly, since only three months' data are available, 100 veh/h/ln is regarded as the inflow intervals for the model applications on Beijing urban freeways so that the sample size requirement can be met with (see Eq. 4.4).

Three elements have been discussed in the proposed travel time reliability model in chapter 3 and 4. That is, probability of traffic breakdown, travel time uncertainty (travel time variability), and travel time unreliability. Figure 4.10, Figure 4.11, Figure 4.12 show the results from the traffic data in Beijing urban freeways.

Figure 4.10 illustrates the probability of traffic breakdown as a function of inflow levels on Beijing second ring urban freeway. In the graph, the probability of traffic breakdown increases with the increasing inflows. Figure 4.11 demonstrates the travel time uncertainty as a function of inflows. Travel time variability before breakdown (TTV^f) does not change with the increasing inflows. TTV^f is about 25 seconds/km in this case, much

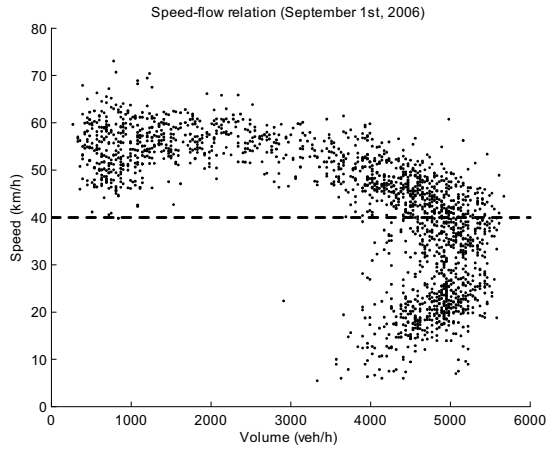


Figure 4.9: Speed-flow relations on Beijing second ring urban freeway (September 1st, 2006). Data is obtained from loop detectors (10-minute aggregate)

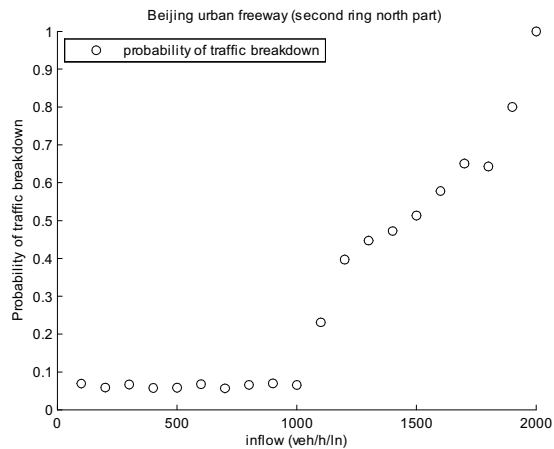


Figure 4.10: Probability of traffic breakdown as a function of inflow levels on Beijing second ring urban freeway

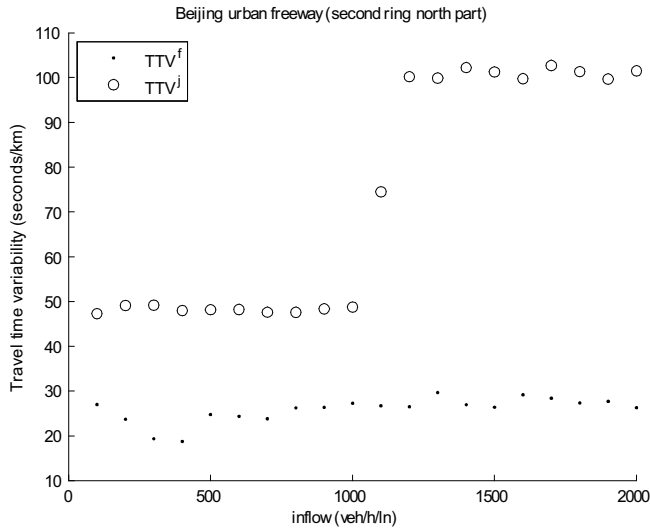


Figure 4.11: Travel time uncertainty (travel time variability) before and after breakdown as a function of inflow levels on Beijing second ring urban freeway

higher than Dutch freeways (5 seconds/km). This may due to the different driving behaviors and traffic mix between Dutch and Beijing. Travel time uncertainty (variability) after breakdown (TTV^j) increases with the increasing inflows.

Figure 4.12 illustrates the travel time unreliability as a function of inflows on Beijing urban freeway. It can be seen that the travel time unreliability increases with the increasing inflows. The travel time unreliability-inflow relation on Beijing urban freeway has the similar trend with the relation on Dutch freeways. Figure 4.12 shows the TLZ as well. Above λ_{ttr} , travel time unreliability sharply increases with the increasing inflows; below λ_{ttr} , travel times are relatively reliable.

The results from the Beijing urban freeway and the results from Dutch freeways (see Table 4.4) provide the evidence showing that the proposed fitted travel time reliability can reproduce the reliability of freeways reasonably well.

4.6.2 Predictive validity

Validation (predictive validity) is proven by showing that the predictive results of the travel time reliability model give an accurate representation of the real traffic situation. This section shows the validation results of the proposed model. Here the results obtained from the year of 2004 are considered as the *estimated* reliability performance while the results obtained from the year of 2005 are considered as the *observed* reliability performance.

Figure 4.13 shows the estimated and observed probability of traffic breakdown on freeways, for example, on A12 freeway (both directions). It is found from the graph and

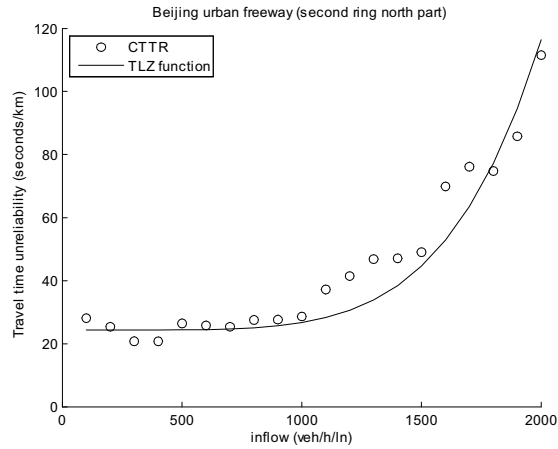


Figure 4.12: Travel time unreliability as a function of inflow levels on Beijing second ring urban freeway

Table 4.5 that the *RMSEP* are 33.8% (A12 freeway, east direction) and 21.7% (A12 freeway, west direction). However, the *MSE* are only 0.001 for both directions. Therefore, the model is able to estimate the probability of traffic breakdown reasonably well.

Table 4.5: Performance indicators of travel time reliability model

	Formula	A1201		A1211	
		p_r^{br}	<i>TTUR</i>	p_r^{br}	<i>TTUR</i>
<i>MRE</i> (%)	$100 \frac{1}{N} \sum \frac{e_n}{t_n}$	19.5	10.4	-0.1	-12.9
<i>MARE</i> (%)	$100 \frac{1}{N} \sum \left \frac{e_n}{t_n} \right $	28.1	16.9	19.5	17.3
<i>MSE</i>	$\frac{1}{N} \sum (e_n)^2$	0.001	3.4	0.001	13.5
<i>RMSE</i>	\sqrt{MSE}	0.031	1.8	0.035	3.7
<i>RMSEP</i> (%)	$100 \frac{RMSE}{t}$	33.8	21.9	21.7	23.0

*) N = number of data points; e_n = the differences between measured and observed values (e.g. travel time unreliability, probability of traffic breakdown); t_n = observed values (e.g. probability of traffic breakdown, travel time unreliability)

Figure 4.14 shows the estimated and observed travel time unreliability on A12 freeway (both directions). It is found from the graph and Table 4.5 that the *RMSEP* are 21.9% (A12 freeway, east direction) and 23.0% (A12 freeway, west direction). The large *RMSEP* value may due to the different characteristics of two years traffic data, like the effects of adverse weather conditions, traffic accidents, road works etc. on traffic data are not completely the same for two years. To conclude, for the same freeway corridor, the proposed travel time reliability model can approximately predict travel time reliability on freeways reasonably well.

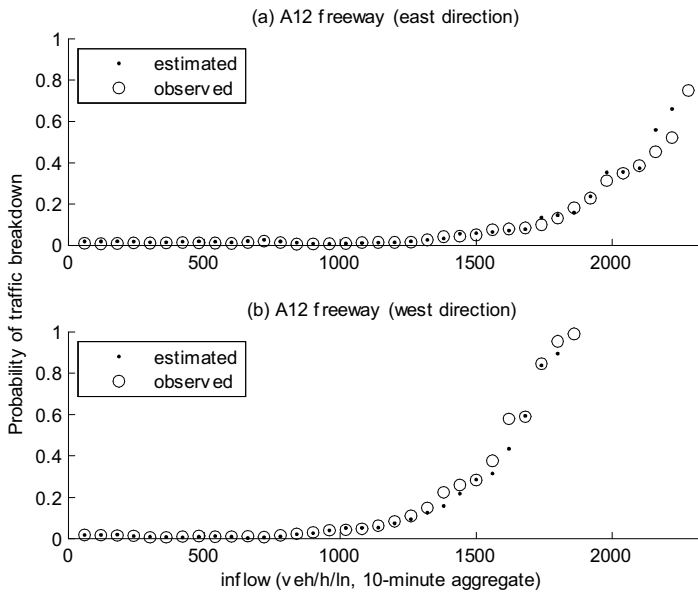


Figure 4.13: Estimated and observed probability of traffic breakdown as a function of inflow levels on A12 freeway (upper figure: A12 freeway east direction; lower figure: A12 freeway west direction)

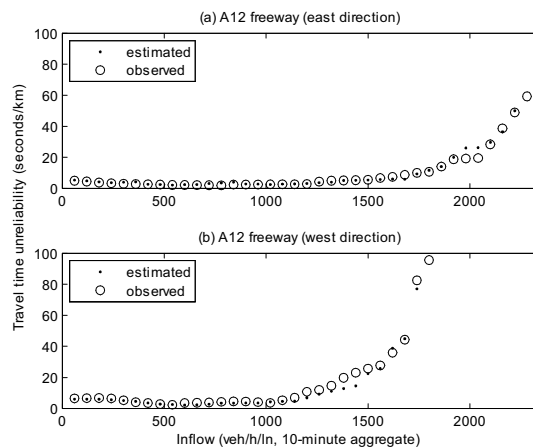


Figure 4.14: Estimated and observed travel time unreliability as a function of inflow levels on A12 freeway (upper figure: A12 freeway east direction; lower figure: A12 freeway west direction)

4.7 Discussion

As discussed in section 4.2.2, the measurement time interval affects the reliability analysis:

- Firstly, the size of measurement interval affects the precision of inflow since inflow is an indicator of traffic flow and hourly inflows will be used in both traffic assignment model and traffic monitoring systems. For instance, Smith & Ulmer (2003) reported that there is significant noise in the (in)flow resulting in unstable (in)flow measurements in case of small measurement intervals. On the other hand, 1-hour counts are not useful since they also average high and low volumes into average volumes which may ignore the phenomenon of the flow stochasticity. Smith & Ulmer (2003) concluded that measurement intervals between 10 and 15 minutes are appropriate to compute hourly flows. Moreover, a very large measurement interval like 1-hour also average high and low travel times into average travel times which may underestimate travel time variability.
- Secondly, the size of measurement interval affects the measurement of the probability of traffic breakdown (Persaud et al. (1998), Lorenz & Elefteriadou (2000)). At a small measurement interval (e.g. 1 minute), the traffic stream is capable of absorbing brief fluctuations in the flow rate without resulting in a high risk of traffic breakdown. This is because the flow rate is only sustained over a short period. On the other hand, if the aggregation interval is increased to a large value (e.g. 15 minutes), the breakdown probability is substantially higher because the flow rate is sustained over a much longer time period.

Figure 4.15 shows curves of the probability that a traffic break down will occur in 5, 10, 15, 20, 30 minute intervals on the A12 freeway corridor. The probability of breakdown for any given flow is larger if the time interval is larger. This is because, for example, a 20 minute flow of, say, 2400 veh/h/ln is an average of 5 minute flows, many of which could be much higher than 2400 veh/h/ln and have a higher probability of breakdown than 5 minute flows of 2400 veh/h/ln.

$p_5(q_{in})$ represents the probability of a breakdown occurring under an inflow of q_{in} in a given 5-minute interval. Thus, the probability of a breakdown occurs in a whole hour $p_{60}(q_{in})$ is (Kuehne & Anstett 1999):

$$p_{60}(q_{in}) = 1 - (1 - p_5(q_{in}))^{12} \quad (4.8)$$

More generally, Eq. 4.8 could be written as

$$p_T(q_{in}) = 1 - (1 - p_\Delta(q_{in}))^{\frac{T}{\Delta}} \quad (4.9)$$

where,

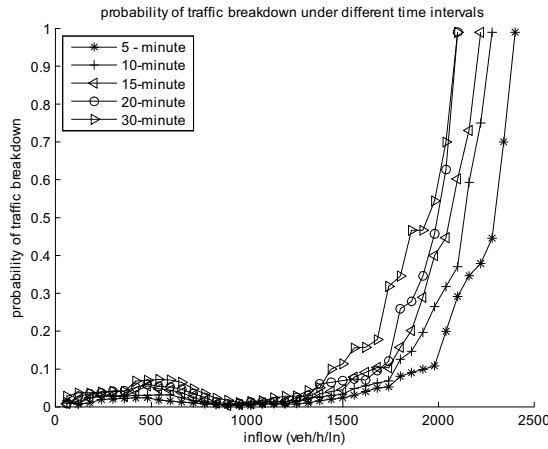


Figure 4.15: Probability of traffic breakdown as a function of inflow levels under different time intervals on the A12 freeway

- $p_T (q_{in})$ = probability of traffic breakdown estimated in intervals of duration T for a given inflow q_{in}
- q_{in} = inflow
- $p_{\Delta} (q_{in})$ = probability of traffic breakdown estimated in intervals of duration Δ for a given inflow q_{in}

Figure 4.16 demonstrates the fitted travel time unreliability as a function of inflow levels under different time intervals: 5-minute, 10-minute, 15-minute, 20-minute, and 30-minute. In case of traffic flows on small time interval like 5-minute, the travel times are relatively reliable since the flow rate is only sustained over a short period. On the other hand, if the time interval is increased to a large value (e.g. 30-minute), the travel times are relatively unreliable since the flow rate is sustained over a much longer time period. However, below the critical travel time unreliability inflow λ_{itr} , the time interval only has a slight or no impact on travel time unreliability analysis. Table 4.6 shows the fitted parameters for travel time unreliability as a function of inflow levels under different time intervals. The graph also provides the evidence that the λ_{itr} decreases as the time intervals increase.

Table 4.6: Fitted parameters for travel time unreliability function under different time interval

time interval	$TTUR_0$	β	γ	λ_{itr}	MAREs
5	3.44	0.70	13.25	1680	26%
10	3.40	0.35	12.75	1500	20%
15	3.67	0.60	11.5	1500	18%
20	3.49	0.60	8.5	1320	14%
30	3.64	0.85	8.25	1320	15%

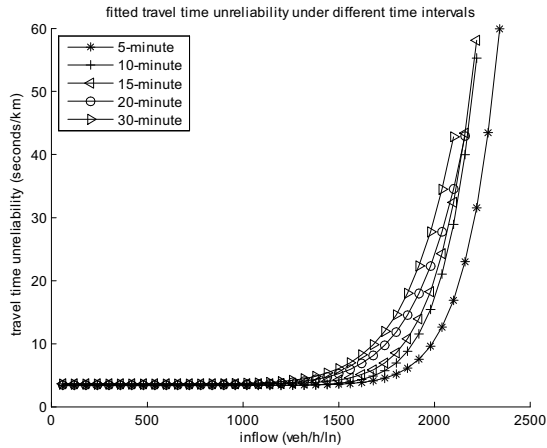


Figure 4.16: Fitted travel time unreliability-inflow function under different time interval on A12 freeway

4.8 Conclusions

The two elements of travel time unreliability, probability of traffic breakdown and travel time uncertainty (variability), are both regarded as a function of inflow levels. This chapter tested the CTTR (Conceptual Travel Time Reliability model) using the empirical data and developed a TLZ function to empirically fit the CTTR. An empirical investigation of travel time variability and travel time unreliability under the different inflow levels on six densely used freeway corridors in the Netherlands provides the preliminary findings:

1. The probability of traffic breakdown increases with the increasing inflows
2. In the inflow-travel time variability relation, below critical transition inflow λ_t ($q_{in} < \lambda_t$) travel time variability is low and travel times are certain
3. For inflows between λ_t and λ_c ($\lambda_t \leq q_{in} \leq \lambda_c$) travel times are uncertain and travel time variability increases with rising inflow.
4. Above λ_c ($q_{in} > \lambda_c$) travel times are relatively certain, mostly in free flow region, but unstable.
5. Travel time unreliability increases with the increasing inflows
6. Below a certain threshold value of inflow λ_{itr} , travel time unreliability is low, but
7. Above the certain threshold value λ_{itr} , travel time unreliability sharply increases with rising inflows.
8. Given a certain inflow level, a higher measurement time interval contributes to less reliable travel times, but

9. The measurement time interval may not play a significant role in travel time variability (uncertainty).

Chapter 5

Extended Travel Time Reliability Model

5.1 Introduction

In chapter 4, the travel time reliability model is regarded as a function of the principal input factor: inflow. Recall from section 4.2.3, on the one hand, the inflow level plays an important role in both the probability of traffic flow breakdown and the travel time uncertainty and hence the travel time unreliability. On the other hand, traffic breakdown and travel time uncertainty for a given inflow level vary with internal and external factors, such as road geometry, adverse weather conditions, traffic control, traffic accidents etc. For instance, in case of a short weaving section length, drivers already at the beginning of the weaving section experience considerable pressure to change lanes. This may lead to break down at lower flow rates than in the case of longer weaving length. It is probable, therefore, that short weaving length results in less reliable travel times. Another example, the volume of traffic which a road can handle on one day may cause severe congestion on the next due to for instance traffic accidents affecting capacity. Thus, there is a need to investigate how the other factors impact on travel time reliability. In this chapter, we analyze the following four factors influencing travel time reliability:

- 1 *Road Geometry Impacts*
- 2 *Adverse Weather Impacts*
- 3 *Speed Limits Impacts*
- 4 *Traffic Accidents Impacts*

In general, these four factors have strong impacts on traffic flow operations (see Figure 2.3). Road characteristics like the length of weaving section affect capacity (Zhang & Rakha 2005); adverse weather conditions may reduce the capacity in a traffic network (Maze et al. 2006); Speed Limits (SL) influences speed (Ulfarsson et al. 2005); traffic accidents may sharply reduce the capacity in a road (Knoop et al. 2008). Thus, it is expected that the changes of capacity or demand due to these four factors may affect the probability of traffic breakdown, travel time uncertainty and thereby travel time unreliability. In the next four sections, the impacts of these four factors on travel time reliability are investigated, respectively. Thereafter, section 5.6 summarizes the main results and discusses some implications.

5.2 Road Geometry Impacts

Travel times on freeways are variable due to variations both in demand and in capacity. One group of factors affecting capacity are geometrical characteristics of freeways, such as the number of ramps (on-ramp or off-ramp) and weaving sections per unit road length and their physical characteristics. It is expected that these characteristics also influence the probability of traffic breakdown, travel time variability and travel time reliability. In section 5.2.1 we briefly outline the effects of road geometry on traffic flow operations. Then we present the developed travel time reliability model in section 5.2.2 followed by a detailed data description in section 5.2.3. Thereafter, some preliminary results are illustrated and analysis are discussed in section 5.2.4.

5.2.1 Overview of the impacts of road geometry on traffic flow operations

Freeway corridors are constituted of freeway segments, categorized into three types: basic freeway segments, weaving sections, and off/on-ramp sections.

Weaving vehicles can heavily affect the quality of service on a freeway especially when the weaving has to be done in relatively short distance. One of the important characteristics of weaving sections is the weaving section length. Existing studies on weaving section length and its impact on operational performance typically focused either on the relationship between weaving length and weaving speed, or the relationship between weaving length and capacity. Typically, the former is found most in the U.S. practice, while the latter is usually applied in, for example, Germany and The Netherlands. Fitzpatrick & Nowlin (1996) considered weaving speed as a measure of effectiveness to investigate the relationship between weaving length and weaving speeds under different weaving volumes (on-ramp plus off-ramp volume) on one-way frontage roads. They concluded that traffic operations on weaving sections with a length of 100m began to breakdown sooner than weaving sections with length of 200m and above and suggested that the weaving sections on a frontage road should have a weaving length of at least 200m. The effect of weaving length on weaving section capacity was, for example, studied by (Vermijs & Schuurman 1994) and (Dijker & Schuurman 2003). They used the microscopic simulation model FOSIM (Freeway Operations SIMulation) to calculate capacities for symmetrical and asymmetrical weaving sections, respectively, which were included in the Dutch freeway capacity manual. They found a somewhat limited influence of weaving section length, mainly because the minimum lengths according to Dutch design guidelines are already considerable. Zhang & Rakha (2005) validated the INTEGRATION model using field data. They estimated the capacity of weaving sections using the validated model. It was found that the length of weaving section has larger impacts on the capacity when weaving sections are of short lengths.

5.2.2 Model development

Speed reduces in the process of lane changing as the volume, or the share of weaving vehicles, increases. When the relationship with weaving section length or the ramp length

(ramp length means the length of on-ramp acceleration lane or off-ramp deceleration lane) is considered, according to lane changing in the case of short weaving sections or short ramp segments drivers already at the beginning of the weaving section or ramp segment experience considerable pressure to change lanes. This leads to drivers more often accepting gaps which require significant speed reductions to regain acceptable following distances, which may lead to *breakdown* at lower flow rates than in the case of longer weaving sections or longer ramp segments. It is probable, therefore, that there is a relation between the length of weaving sections or ramp segments and travel time variability or travel time reliability (Tu, van Lint & van Zuylen 2006b) (Tu, Dijkster & van Zuylen 2006). In this context, travel time variability or travel time unreliability can be regarded as a function of ramp length or weaving length.

Not only the length, but also the other road characteristics, such as the distances between off/on ramps (or the number of ramps or weaving section), have influence on the network performance. The more ramps, the more time and space in which the driver must change lanes. Consequently, more vehicle conflicts might be caused and hence higher travel time variability and less reliable travel time. Travel time variability can be considered as a function of the distances between ramps (the number of ramps per ten kilometers) in a freeway corridor. A weaving section comprising of one on-ramp and one off-ramp is regarded as two ramps in this study. Both on-ramp and off-ramp are regarded as a ramp.

The hypotheses below will be tested in the rest of this section:

1. Travel time uncertainty increases with the decreasing on-/off-ramp length or weaving length
2. The probability of traffic breakdown increases with the decreasing on-/off-ramp length or weaving length
3. As a result of hypothesis 1 and hypothesis 2, travel time unreliability increases with the decreasing on-/off-ramp length or weaving length
4. The shorter the distances between ramps, the higher probability of traffic breakdown along a freeway corridor (Tu, van Lint & van Zuylen 2007c) and the higher travel time unreliability.

5.2.3 Empirical data

In order to address the above model and hypotheses, we base our analysis on data obtained from the Regiolab-Delft monitoring system. Figure 5.1 illustrates the general framework for collecting traffic data of travel times and freeway characteristics: first collect basic freeway data (i.e. weaving length, ramp length, route length, number of lanes etc.) and loop detector data; next implement data cleaning to replace the missing or corrupted data; then process these data (e.g. travel times).

Weaving sections

Weaving sections are formed when a merge and a diverge in close proximity require either merging or diverging vehicles to execute one or more lane changes. In traffic engineering,

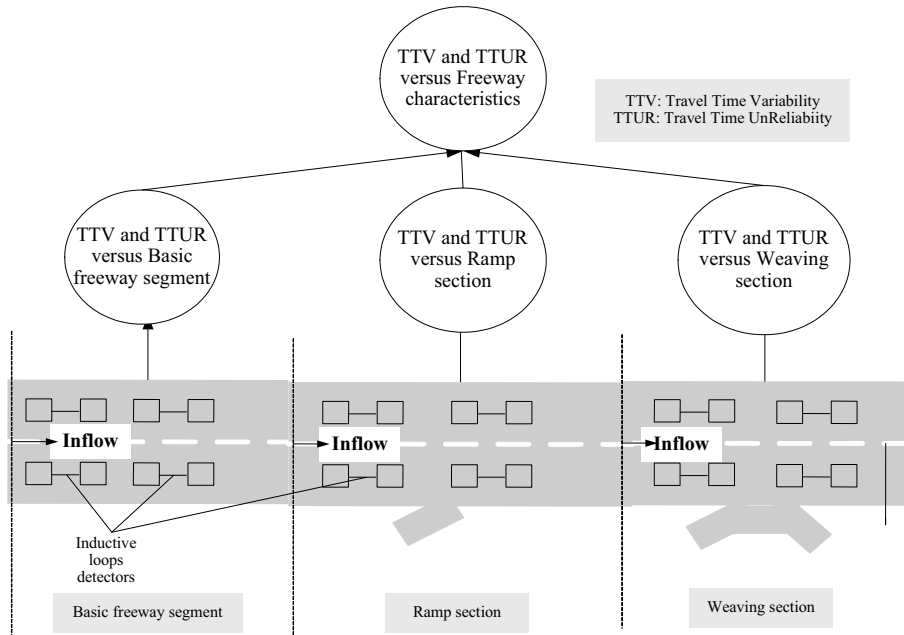


Figure 5.1: General framework for collecting traffic data of travel time and freeway characteristics in Regiolab-Delft traffic monitoring systems in The Netherlands

three types of weaving sections are traditionally distinguished based on the minimum number of lane changes required for completing the weaving manoeuvres (TRB 2000) (see examples in Figure 5.2).

Type A weaving section: All weaving vehicles must make one lane change to complete their manoeuvre successfully. The most common form of Type A weaving segment is formed by a one-lane on-ramp followed by a one-lane off-ramp, with the two connected by a continuous auxiliary lane.

Type B weaving section: One weaving movement can be made without making any lane change, while the other weaving movement requires at most one lane change.

Type C weaving section: One weaving movement can be made without making any lane change, while the other weaving movement requires at least two lane changes.

Due to the lack of data for the other two types of weaving sections, only the first type of weaving sections' data have been collected. Eight weaving sections with three lanes have been selected. Their basic information of weaving sections is shown in Table 5.1 The weaving length ranges from 570m to 2450m.

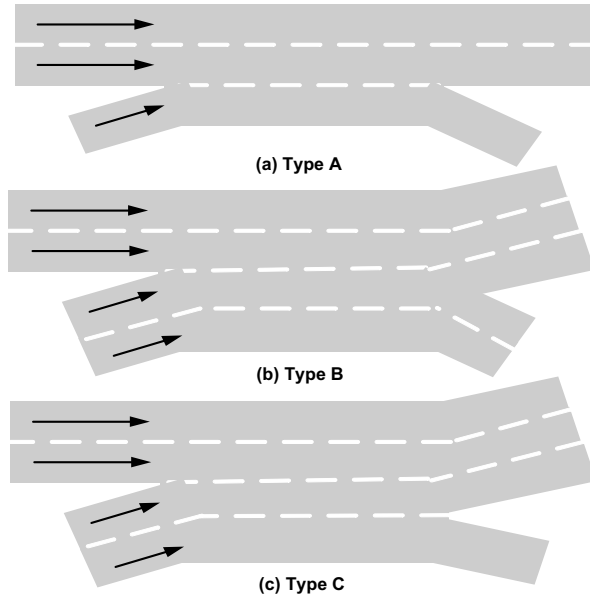


Figure 5.2: Examples of three types of weaving sections

Table 5.1: Physical data of eight weaving sections

code	freeway	weaving length (m)	lanes
w1303	A13	570	3
w1302	A13	670	3
w1301	A13	690	3
w1511	A15	750	3
w1601	A16	750	3
w1314	A13	800	3
w1513	A15	950	3
w1611	A16	2450	3

Ramp sections

Ramp sections are designed to facilitate smooth merging of on-ramp vehicles into the freeway traffic stream and smooth diverging of off-ramp vehicles from the freeway traf-

fic stream onto the ramp. The basic information of twenty ramp sections (nine on-ramp sections and eleven off-ramp sections) is illustrated in Table 5.2. The lengths of the acceleration lane at on-ramps range from 150m to 420m while the lengths of deceleration lane at off-ramps range from 100m to 550m.

Table 5.2: Physical data of twenty ramp sections

code	freeway	ramp length (m)	type
r1212	A12	150	on-ramp
r2006	A20	160	on-ramp
r2004	A20	250	on-ramp
r1203	A12	300	on-ramp
r1501	A15	320	on-ramp
r1205	A12	350	on-ramp
r2015	A20	350	on-ramp
r2002	A20	350	on-ramp
r1511	A15	420	on-ramp
r1215	A12	100	off-ramp
r2011	A20	150	off-ramp
r2016	A20	200	off-ramp
r2017	A20	200	off-ramp
r1513	A15	240	off-ramp
r1312	A13	250	off-ramp
r2003	A20	280	off-ramp
r2001	A20	350	off-ramp
r1204	A12	400	off-ramp
r1202	A12	500	off-ramp
r1201	A12	550	off-ramp

Basic freeway segments

Basic freeway segments are outside the influence area of ramps or weaving areas of freeway. The basic information of three basic freeway segments is provided in Table 5.3. The basic freeway segments are approximately in 2230 meters length on average.

Table 5.3: Physical data of three basic freeway segments

code	freeway	section length (m)	lanes
b1208	A12	1905	3
b2008	A20	2240	2
b2018	A20	2550	2

Freeway corridors

Six freeway corridors are illustrated in Table 5.4. The corridors are on average (approximately) 13.5 km long, ranging from 9.7 km to 17.3 km.

Table 5.4: Physical data of six freeway corridors

code	freeway	length (m)	β *
1201	A12	17280	4.1
1211	A12	15520	7.1
1301	A13	10590	8.5
1501	A15	9725	7.2
2001	A20	10500	6.7
2011	A20	17215	7.0

*) β = number of ramps per ten kilometers

5.2.4 Results and analysis

In this section, the effects of ramp length or weaving length on travel time reliability under all inflow levels are discussed.

Ramp sections

A ramp section is an area of traffic demand competing for space. Upstream freeway traffic competes for space with entering on-ramp vehicles in merge areas. Figure 5.3 demonstrates the relationship between (a) travel time variability under free flow conditions and congested conditions, (b) probability of traffic breakdown, (c) travel time unreliability and length of acceleration lane on eight on-ramp sections. It can be seen that TTV in free flow conditions does not vary with the increasing length of the acceleration lane, but both TTV after breakdown and the probability of traffic breakdown slightly decreases with the increasing length of the acceleration lane with a length below 300m. Therefore, TTUR decrease as the length of the acceleration lane increases with a length below 300m. The length of an on-ramp section increasing 100 meters can reduce travel time variability of 3.3 seconds/km (see Eq. 5.1).

$$TTUR = \begin{cases} 15.7 - 0.033 \times L_a, & L_a < 300m \\ 5.8, & L_a \geq 300m \end{cases} \quad (5.1)$$

in which L_a = length of acceleration lane on on-ramp.

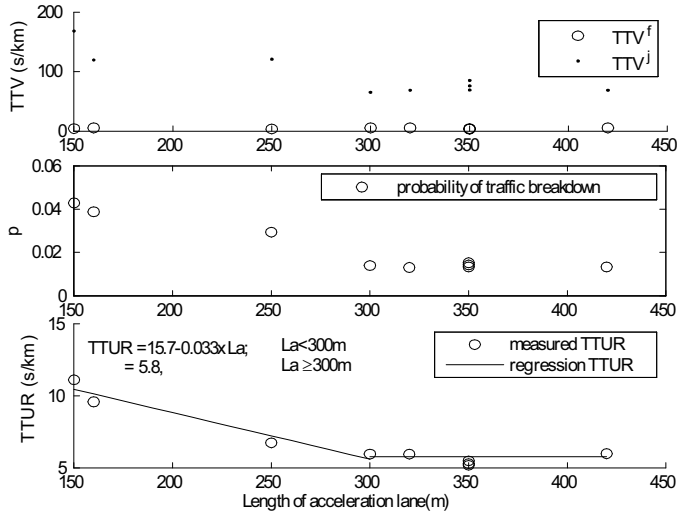


Figure 5.3: Travel time variability and travel time unreliability as a function of the length of acceleration lane

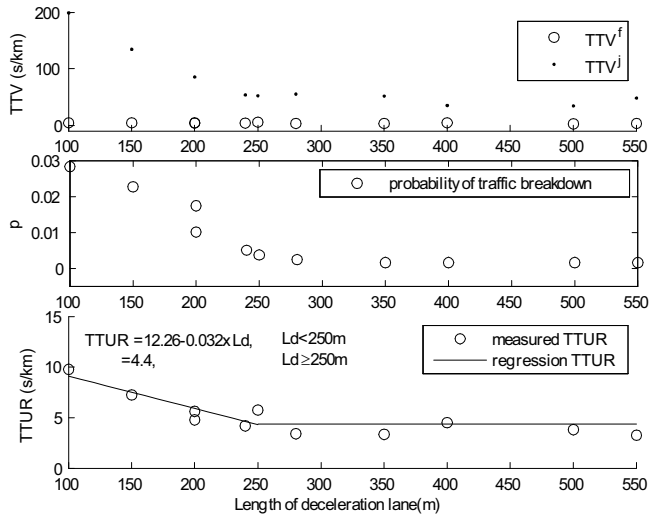


Figure 5.4: Travel time variability and travel time unreliability as a function of the length of deceleration lane

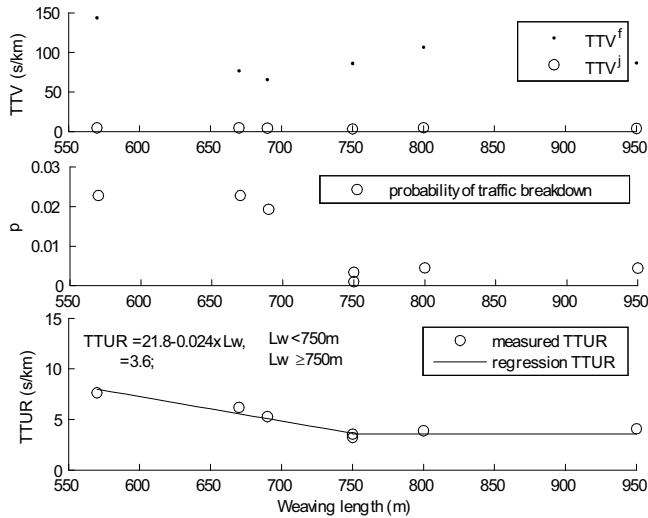


Figure 5.5: Travel time variability and travel time unreliability as a function of weaving length

At off-ramp, the basic manoeuvre is a diverge. That is, a single traffic stream separated into two streams. Figure 5.4 depicts the relationship between (a) TTV, (b) probability of traffic breakdown, (c) TTUR and length of the deceleration lane on eleven off-ramp sections. It can be seen that TTV in free flow conditions does not vary with the increasing length of the deceleration lane, but both TTV in congested conditions and the probability of traffic breakdown slightly decreases with the increasing length of the deceleration lane with a length below 250m. Therefore, TTUR decrease as the length of the deceleration lane increases with a length below 250m. The length of an off-ramp section increasing 100 meters can reduce travel time variability of 3.2 seconds/km (see Eq.5.2).

$$TTUR = \begin{cases} 12.7 - 0.032 \times L_d, & L_d < 250m \\ 4.4, & L_d \geq 250m \end{cases} \quad (5.2)$$

in which L_d = length of deceleration lane on off-ramp

Weaving sections

Weaving sections can heavily affect the quality of service on a facility since the weaving vehicles have conflicts on a relatively short weaving length. Figure 5.5 demonstrates the relationship between weaving length and (a) travel time variability, (b) probability of traffic breakdown, (c) travel time unreliability. It can be seen that TTV in free flow conditions does not vary with the increasing weaving length, but both TTV in congested conditions and the probability of traffic breakdown slightly decreases with the increasing weaving length with a length below 750m. Therefore, TTUR decrease as the weaving length increases with a length below 750m. An increase in the length of a weaving section by 100 meters can reduce travel time variability by 2.4 seconds/km (see Eq.). For long weaving sections there is more space for weaving vehicles to handle the unexpected events. There-

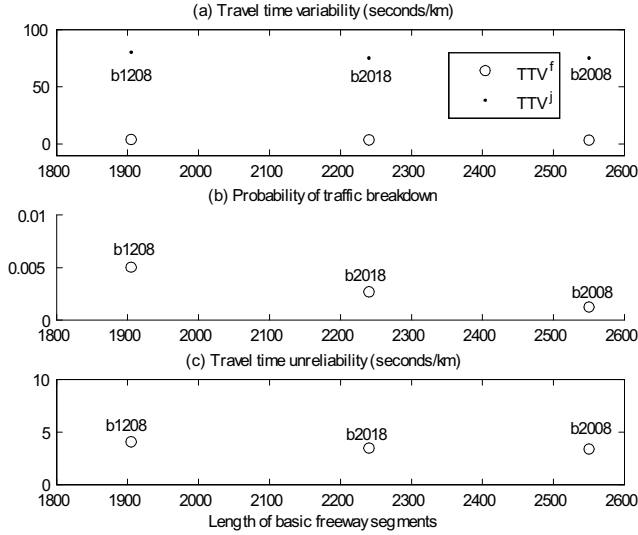


Figure 5.6: Travel time variability and travel time unreliability as a function of length of basic freeway segment

for the probability of traffic breakdown will be lower than short weaving sections (Tu, van Lint & van Zuylen 2006b). Thus the traffic flow in longer weaving sections is more stable than the traffic flow in shorter weaving sections.

$$TTUR = \begin{cases} 21.8 - 0.024 \times L_w, & L_w < 750m \\ 3.6, & L_w \geq 750m \end{cases} \quad (5.3)$$

in which L_w = length of weaving sections

Basic freeway segments

Figure 5.6 shows both (a) TTV, (b) probability of traffic breakdown, and (c) TTUR on the three basic freeway segments under the whole inflow scale. All three basic freeway segments are uniform segments. Traffic flow within basic freeway segments can be highly variable depending on the conditions constructing flows at upstream and downstream bottleneck locations (Steward et al. 1996). We choose three basic freeway segments. The travel time unreliability on basic freeway segments are about 4 seconds/km.

Comparing Figure 5.3, Figure 5.4, Figure 5.5, and Figure 5.6, TTUR at weaving sections and at ramp sections in short weaving distance are higher than that at basic freeway segments due to the conflicts of the weaving vehicles at the weaving sections or at the ramp sections. It is concluded that the more ramps or weaving sections along a route corridor, the more vehicles conflicts and the higher probability of traffic breakdown. Therefore, travel times are less reliable.

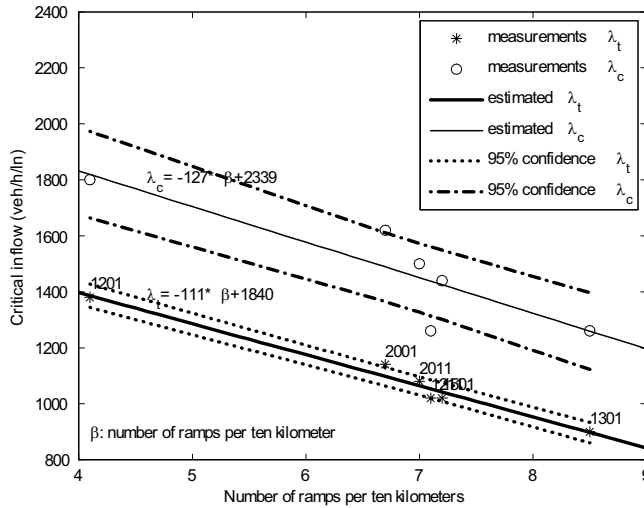


Figure 5.7: Critical transition inflow λ_t and critical capacity inflow λ_c for 90th percentile travel times under freeway characteristics for six freeway corridors on the freeway in Regiolab-Delft traffic monitoring systems

Freeway corridors

Travel time variability

In chapter 4, the static inflow-travel time variability is found to be determined by two critical inflows, λ_t and λ_c . These two critical inflows varied with different circumstances like road geometry. Figure 5.7 demonstrates the relationship between the two critical inflows on the inflow-percentile travel time relations and the number of ramps per ten kilometers (β) (the average distance between ramps) on freeway corridors. In general, both critical transition inflow λ_t and critical capacity inflow λ_c decrease with the rising number of ramps per unit road length β , as shown in Eq. 5.4:

$$\begin{aligned}\lambda_t &= -111 \times \beta + 1840 + \varepsilon_t, R^2 = 0.97 \\ \lambda_c &= -127 \times \beta + 2339 + \varepsilon_c, R^2 = 0.77\end{aligned}\tag{5.4}$$

where,

- λ_t = critical transition inflow
- λ_c = critical capacity inflow
- β = number of ramps per ten kilometers
- ε_t = error item
- ε_c = error item

Consequently, in case of no ramps (only basic freeway segments, $\beta = 0$ in Eq. 5.4) in the freeway corridors, the critical transition inflow λ_t is around 1840 veh/h/ln while the

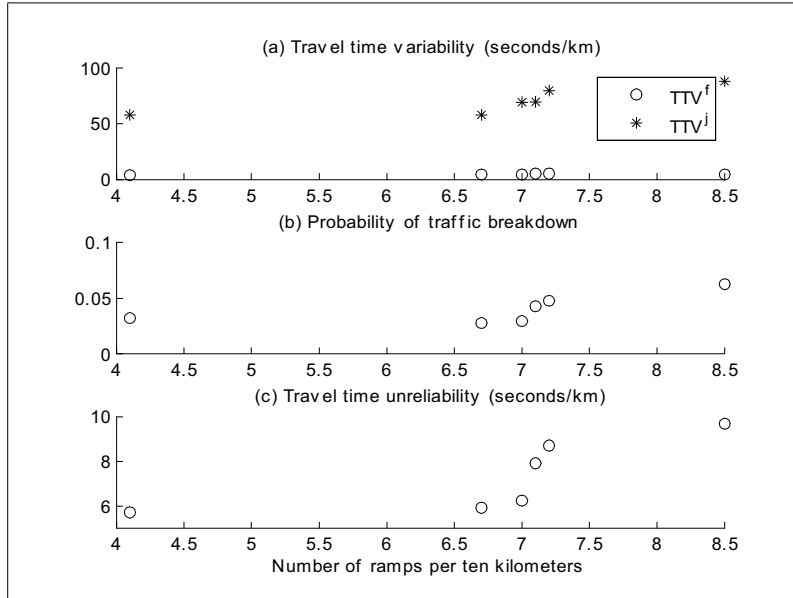


Figure 5.8: The relations between (a) travel time variability, (b) probability of traffic breakdown, (c) travel time unreliability and number of ramps per unit road length

critical capacity inflow λ_c is about 2340 veh/h/ln. That is, travel time variability and travel time unreliability sharply increases above λ_t , a much lower value than λ_c . Furthermore, the difference between λ_t and λ_c decreases with the rising β . It seems that the freeway characteristics β have more impacts on λ_c than on λ_t .

Travel time unreliability

Figure 5.8 shows the relationship between the number of ramps per ten kilometers (β) and (a) travel time variability, (b) probability of traffic breakdown, (c) travel time unreliability on six freeway corridors. Clearly, during the free flow conditions, β has no impacts on travel time variability. After traffic breakdown, travel time variability increases with rising β . Moreover, the probability of traffic breakdown also increases with increasing β . Therefore, the travel time unreliability increases with increasing β . That is, travel times are less reliable in case more ramps on an unit road length (or the short average distances between two ramps along the freeway corridors). As can be seen in Figure 5.8 (c), in case the average distances between off/on ramps (or weaving sections) is below 3 km, travel time unreliability sharply increases. In terms of the implications for practice, the average distances should be at least 3 km.

Table 4.2 describes the number of ramps per ten kilometers on freeway corridors and Table 4.4 shows the critical travel time reliability inflows λ_{ttr} of the freeway corridors. It provides the evidence that a too short distance between off/on ramps result in lower λ_{ttr} and hence less reliable travel times. If β is larger than 6.7 (the average distance smaller than 3 km), the λ_{ttr} sharply decreases with the rising β .

5.2.5 Conclusions

This section presented an empirical investigation into the effects of road geometry on the probability of traffic breakdown, travel time uncertainty and travel time unreliability. Where these studies find that there exists threshold values L for the length of ramp sections and weaving sections. Below L , travel time unreliability increases with the decreasing length of ramp sections or weaving sections. Above L , the length has far less impact on travel time unreliability. In a freeway corridor, travel time reliability is strongly affected by the number of ramps per unit road length. Above a certain threshold value, the more ramps, the more travel time unreliability. Such a finding can be of significant importance for geometric design standards:

- the length of deceleration lanes should be longer than 250m;
- the length of acceleration lanes should be longer than 300m;
- the weaving length should be longer than 750m;
- the number of ramps per ten kilometers in a freeway corridor should be smaller than 6.8, or the distance between off/on ramps should be larger than 3 km.

The empirical results also provide below evidences:

- Short ramp (on-ramp, off-ramp, and weaving section) length results in:
 - Higher possibility of traffic breakdown
 - Higher travel time uncertainty (variability)
 - Higher travel time unreliability
- Travel time uncertainty in congested conditions increases with the decreasing of the distances between off/on ramps
- The probability of traffic breakdown increases with the decreasing of the distances between off/on ramps
- Travel time unreliability decreases with the decreasing of the distances between off/on ramps

5.3 Adverse Weather Impacts

Adverse weather conditions may (locally) reduce the capacity in a traffic network, but may at the same time yield (global) changes in traffic demand, due to people changing routes, departure time, mode or even reconsidering taking a trip together. It is therefore expected that travel time reliability is strongly affected by adverse weather since travel time reliability results from the variations both in demand and capacity. In the next subsection, we briefly present the overview of the impacts of adverse weather on traffic flow operations. Then, the method to model the impacts of adverse weather on travel time reliability is built up in section 5.3.2, followed a description of the empirical data to investigate the impacts in section 5.3.3. Thereafter, preliminary results and analysis are presented in section 5.3.4.

5.3.1 Overview of the impacts of adverse weather on traffic flow operations

In the literature a considerable amount of information concerning the impact of weather conditions on traffic operations can be found, which can be categorized into three predominant aspects: impact on traffic demand, impact on traffic safety, and impact on capacity and traffic flow relationships, such as the flow-occupancy and speed-flow relationships.

Firstly, several studies have found that weather conditions have significant impact on traffic demand (Mcbride et al. (1977), Hanbali & Kuemmel (1993), Hanbali (1994), Knapp (2000), Keay & Simmonds (2005), Maze et al. (2006)). For example, Hanbali & Kuemmel (1993) investigated volume reductions due to winter storms across varied snowfall intensities, time of day, day of week, and roadway type in USA. They found that the reductions of volumes ranged from 7 percent to 56 percent, depending on the category of winter event and concluded that volume reductions increase with the total volume of snow. Maze et al. (2006) described the impact of weather identified through both the literature and the prior research conducted by the Center for Transportation Research and Education (CTRE) in USA. They noted that depending on the type of traffic (commuter, commercial, long-distance travel, etc.) and on the severity of the weather, roadway traffic volumes have shown to be less than 5 percent lower during rain storms, and 7 to 80 percent lower for snow storms.

Secondly, research on the impact of weather conditions on traffic safety began in the fifties. From then on, substantial studies have focused on this subject (Mcbride et al. (1977), Brodsky & Hakkert (1988), Savenhed (1994), Shankar et al. (1995), Brow & Baass (1997), Nofal & Saeed (1997), Edwards (1998), Khattak et al. (1998), Khattak et al. (2000), Eisenberg (2004)). For instance, Savenhed (1994) found that severe injury rates on roads with snow and ice can be several times greater than on roadways under normal conditions. Maze et al. (2006) reported that crash rates increases during inclement weather, with crash rates increasing dramatically during snowstorms.

Thirdly, adverse weather affects traffic supply factors. This does not only imply capacity (which generally decreases with adverse weather), but in fact the entire fundamental diagram. The effect of adverse weather conditions on the flow-occupancy and speed-flow relationships has been investigated by several researchers (Koshi et al. (1981), Ibrahim & Hall (1994), Brilon & Ponzlet (1996)). Regression analyses were performed to select a proper model representing the flow-occupancy and speed-flow relationships for uncongested operations (Ibrahim & Hall 1994). They reported that light precipitation, of either form, does not have a very large effect on free-flow speeds, maximum flows, or speed at maximum flows and both heavy rain and snow can have great effects, such as a 50km/h reduction in free-flow speeds, and nearly a 50 percent reduction in maximum observed flows. Several researchers have measured the extent to which capacity is influenced by weather (Jones et al. (1970), Kleitsch & Cleveland (1971), Ries (1981), Maze et al. (2006)). For example, Maze et al. (2006) noted that heavy rains (more than 6.4 mm/h) reduce freeway capacity by an average of 14 percent and heavy snows (12.7 mm/h) reduce freeway capacity by an average of 22 percent.

In short, it is found in prior researches that adverse weather conditions reduce not only traffic demand and capacity, but also traffic safety. This section tries to quantify the impact

of adverse weather conditions on travel time reliability rather than on capacity or demand per se.

5.3.2 Model development

Adverse weather may affect road performance and traffic flow operations. For instance, rainy weather condition may (a) lead to lower visibility due to the reflection on wet surface, (b) reduce the friction on the road surface, and (c) hence result in longer braking distance and longer time headway (Eisenberg 2004). Adverse weather conditions lead to higher accident risk (Zhang & Rakha 2005). Therefore, adverse weather conditions may have negative impacts on traffic flow operations. In this context, traffic breakdown may more easily occur under adverse weather conditions than normal weather conditions. Figure 5.9 shows an example of traffic breakdown as a function of inflow levels on both normal weather conditions and rain weather conditions on A12 freeway in The Netherlands. It is found in the graph that rain weather conditions results in higher probability of traffic breakdown compared with the probability under normal weather conditions. Consequently, we postulate below hypotheses:

- Adverse weather conditions result in more traffic breakdown
- Adverse weather conditions lead to more variability in travel times after traffic breakdown
- Therefore, travel times under adverse weather conditions become less reliable compared with that under normal weather conditions

5.3.3 Empirical data

The traffic data are obtained from Regiolab-Delft traffic monitoring system (see section 4.2). The weather data are collected from weather stations of the Dutch Meteorological Institute KNMI, in which 37 weather stations spread across the Netherlands. Five groups of weather variables are distinguished: rain, fog, ice, storm, and snow. Note that only the presence of the adverse weather is studied in this dissertation. For instance, rain weather conditions indicate that rain is occurring, but the severity of rain has not been reported. Of course, information concerning extra variables, like the quantity or duration of snowfall would have been desirable.

This research assumed a 10-minute time interval for weather and traffic data. Thus, weather data from Rotterdam weather station and traffic data were combined using constraints of date, hour, and 10-minute intervals since the studied freeways are close to the Rotterdam weather station. Since the measurement took places each hour during one year, each station disposes of 8760 observations per variable. The weather data used here consists of the whole year of 2004 measurements. The distance from these six freeways to the Rotterdam weather station are within 30 km. Therefore, the hourly-based weather measurements in Rotterdam weather station can be the representative data of the six freeway corridors.

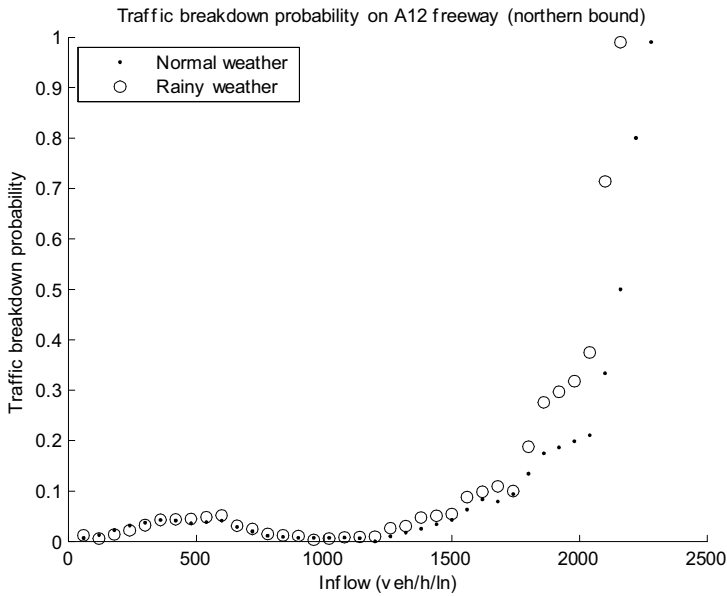


Figure 5.9: Example of traffic breakdown as a function of inflow levels on both normal weather conditions and rainy weather conditions (A12 freeway, northern bound, 2004)

Finally, all results will be tested for their statistical significance by means of Z-statistic.

$$Z_x = \frac{\bar{x} - \mu}{\sigma / \sqrt{n}} \quad (5.5)$$

in which

- \bar{x} = mean value of travel times under adverse weather conditions
- μ = mean value of travel times under normal weather conditions
- σ = standard deviation of travel times under normal weather conditions
- n = number of samples of travel times under adverse weather conditions

Z-score is 1.96 if the confidence level is 95%.

5.3.4 Results and analysis

After the data was collected and processed, a total of 22986 10-minute travel time/weather records were available for each of the six routes. The results of this research are presented in five categories - impacts of rain, snow, ice (black ice), fog, and storm conditions. All these five weather conditions were compared against normal weather conditions, which are defined as no rain, no snow, no ice, no fog, and no storm. The database contained 9126

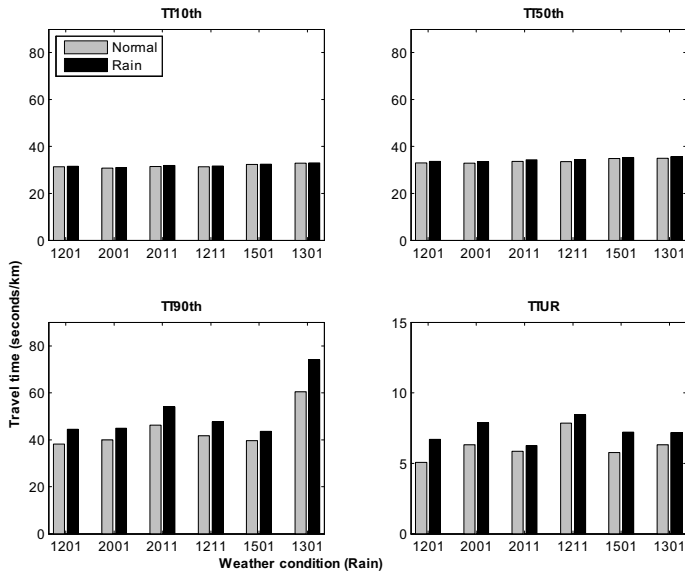


Figure 5.10: Percentile travel time and travel time unreliability under rain weather conditions on six freeway corridors. $TTUR$ =travel time unreliability

records under normal weather conditions. Since there are only limited data available under snow, ice, fog, and storm weather conditions, the impacts of these four adverse weather conditions under all inflow levels on travel time reliability will be investigated. The impacts of rain conditions under both all inflow levels and different inflow levels will be investigated, respectively.

Rain impact

The rain data were divided into two categories (0, under normal weather conditions; 1, under rain weather conditions) for the analysis of the impacts of rain on travel time reliability. The database contained 10668 records under rain conditions.

Figure 5.10 shows that the differences in $TT10th$ and $TT50th$ (median travel time) between under normal weather and rain weather conditions are not large, only increasing by 1%-2% and 2%-5%, respectively. However, the graph illustrates that under rain conditions $TT90th$ increases by 6%-40%, whereas $TTUR$ increases by 31%-106%. In this context, rain weather conditions slightly affect the free flow speed, but strongly result in less reliable travel time. Figure 5.11 also illustrates the travel time distribution under normal weather conditions and rain weather condition on A12 freeway (east direction). It can be seen that rain weather condition have more impact on $TT90th$ than on $TT10th$ and $TT50th$.

From the Z-statistic point of view, rain weather conditions significantly increase mean travel time (Z-statistic of 46, 71, 163, 171, 275, and 157 for six routes).

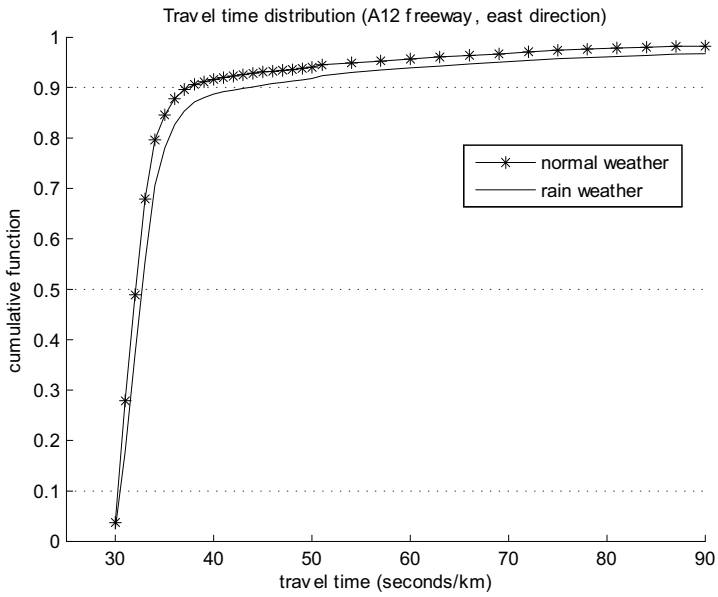


Figure 5.11: Travel time distribution on A12 freeway (east direction) under both normal weather conditions and rain weather conditions

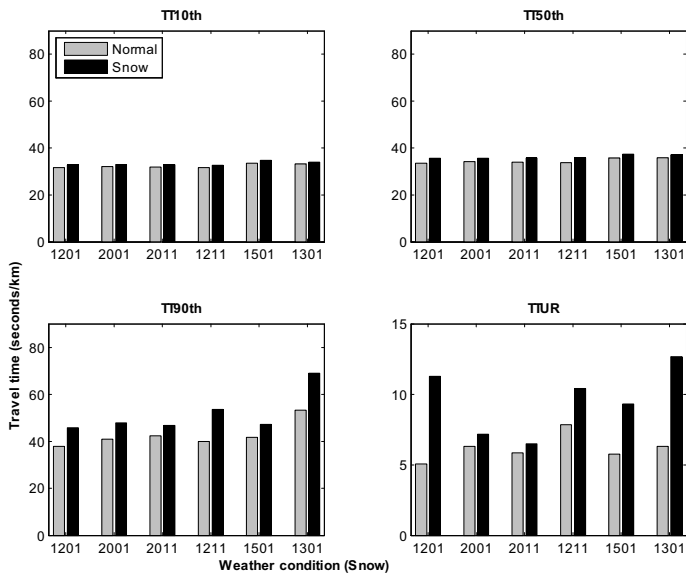


Figure 5.12: Percentile travel time and travel time unreliability under snow weather conditions on six freeway corridors

Snow impact

The snow data were divided into two categories (0, under normal weather conditions; 1, under snow weather conditions) for the analysis of the impacts of snow on travel time reliability. The database contained 510 records under snow conditions.

Figure 5.15 shows that the differences in TT_{10th} and TT_{50th} (median travel time) between under normal weather and snow weather conditions are not large, only increasing by 2%-4% and 4%-6%, respectively. However, the graph illustrates that under snow conditions TT_{90th} increases by 10%-34%, whereas $TTUR$ increases by 27%-83%.

From the Z-statistic point of view, snow weather conditions significantly increase mean travel time (Z-statistic of 23, 42, 59, 23, 28, and 28 for six routes).

Fog impact

The fog data were divided into two categories (0, under normal weather conditions; 1, under fog weather conditions) for the analysis of the impacts of fog on travel time reliability. The database contained 1626 records under fog conditions.

Figure 5.13 shows that the differences in TT_{10th} and TT_{50th} (median travel time) between under normal weather and fog weather conditions are not large, only increasing by 1%-2% and 4%-6%, respectively. However, the graph illustrates that under fog conditions TT_{90th} increases by 2%-63%, whereas $TTUR$ increases by 5%-50%.

From the Z-statistic point of view, fog weather conditions significantly increase mean travel time (Z-statistic of 28, 16, 28, and 27 for four routes: 1201, 2001, 1211 and 2011).

Ice impact

The ice data were divided into two categories (0, under normal weather conditions; 1, under ice weather conditions) for the analysis of the impacts of ice on travel time reliability. The database contained 576 records under ice conditions.

Figure 5.14 shows that the differences in TT_{10th} and TT_{50th} (median travel time) between under normal weather and ice weather conditions are not large, only increasing by 0%-4% and 5%-8%, respectively. However, the graph illustrates that under ice conditions TT_{90th} increases by 26%-102%, whereas $TTUR$ increases by 32%-83%.

From the Z-statistic point of view, ice weather conditions significantly increase mean travel time (Z-statistic of 35, 67, 143, 91, 58, and 60 for six routes).

Storm impact

The storm data were divided into two categories (0, under normal weather conditions; 1, under storm weather conditions) for the analysis of the impacts of storm on travel time reliability. The database contained 480 records under storm conditions.

Figure 5.15 shows that the differences in TT_{10th} and TT_{50th} (median travel time) between under normal weather and storm weather conditions are not large, only increasing by 0%-1% and 0%-3%, respectively. However, the graph illustrates that under storm conditions TT_{90th} increases by 2%-50%, whereas $TTUR$ increases by 7%-35%.

From the Z-statistic point of view, storm weather conditions significantly increase mean travel time (Z-statistic of 3, 10, 44, 44, 75, and 55 for six routes).

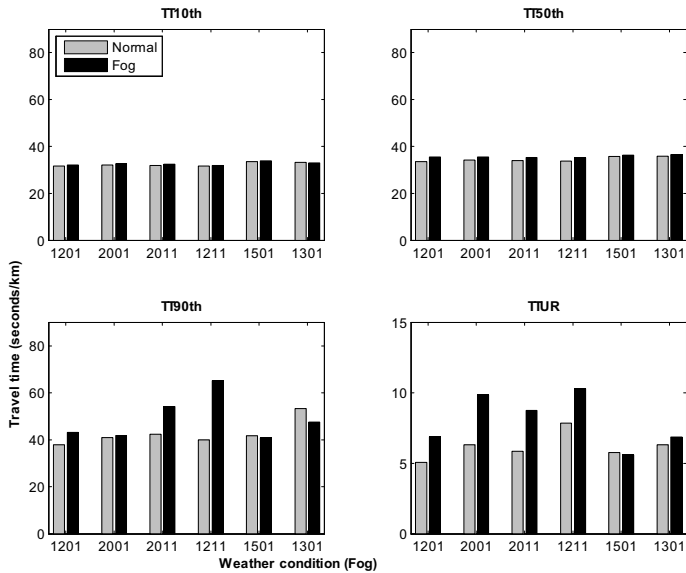


Figure 5.13: Percentile travel time and travel time unreliability under fog weather conditions on six freeway corridors.

Rain impact under different inflow levels

The empirical data on the route level shows there are two critical inflows in the inflow-TTV relationship, in line with the findings in a prior study (Tu, van Lint & van Zuylen 2007b). Figure 5.16 demonstrates the TTV as a function of inflow levels under both normal weather and rain conditions for six freeway corridors in The Netherlands. Note that we only investigate the effect of rain conditions on travel time reliability since the insufficient data of the other weather conditions. Following interesting findings can be found in the graph:

- Under both normal weather and rain weather conditions, the critical inflows λ_t and λ_c for travel time variability can be identified. Table 5.5 shows the estimated critical inflows parameters for TTV, and
- Both critical inflows λ_t and λ_c values under rain weather conditions are smaller than that under normal weather conditions;
- In case of the inflow is below λ_t under both rain weather and normal weather conditions, TTV does not vary with the varied inflow. The rain weather conditions do not increase TTV compared with that under normal weather conditions.
- In case of the inflow is above λ_t , TTV sharply increases with rising inflow under both rain weather and normal weather conditions while TTV under rain weather conditions is larger than that under normal weather conditions.

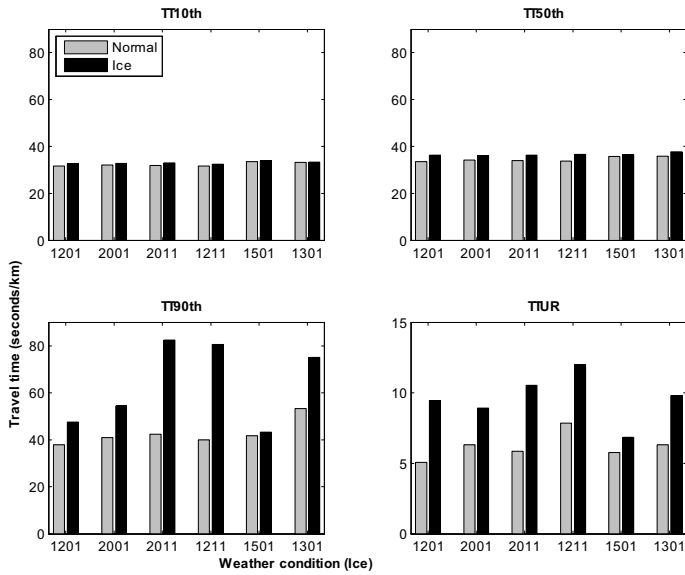


Figure 5.14: Percentile travel time and travel time unreliability under ice weather conditions on six freeway corridors.

Table 5.5: Estimated critical inflow parameters for TTV

λ value		r1201	r2001	r2011	r1211	r1501	r1301
λ_t	Rain	1320	1260	960	960	780	780
	Normal	1380	1260	1080	960	960	1020
λ_c	Rain	1680	1440	1380	1320	1320	1260
	Normal	1800	1500	1500	1320	1320	1320

As shown in chapter 4, travel time unreliability increases with rising inflows. Figure 5.17 illustrates travel time unreliability as a function of inflows under both normal and rain weather conditions on six freeway corridors in The Netherlands. As can be seen in the graph, travel time unreliability increases with rising inflows and travel times under rain weather conditions are less reliable than those under normal weather conditions.

Discussion

On average, adverse weather conditions have a slight influence on TT_{10th} and TT_{50th} increasing by 2% and 4%, respectively; while the adverse weather conditions have significant impacts on TT_{90th} and $TTUR$. For instance, Figure 5.17 (upper left figure) shows that in case inflow is 2160 veh/h/ln, the travel time variability is 30 seconds/km under normal weather conditions while the travel time variability is 72 seconds/km under rain weather conditions. That is travel time variability under rain weather conditions are 2.4 times larger than travel time variability under normal weather conditions.

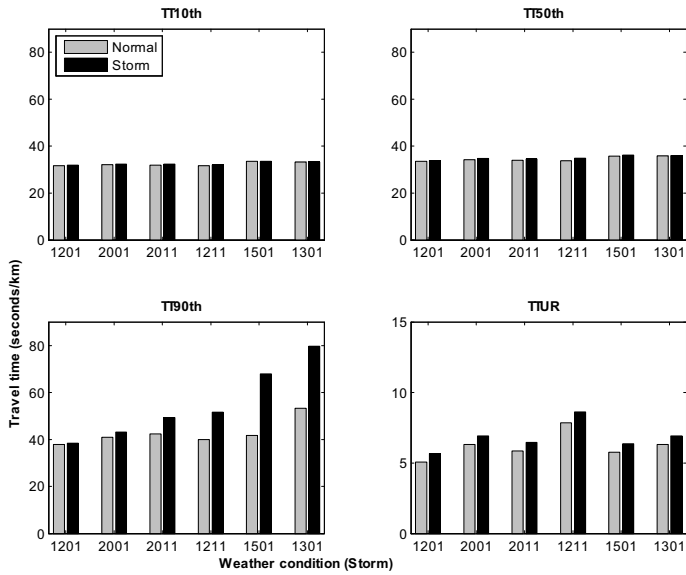


Figure 5.15: Percentile travel time and travel time unreliability under storm weather conditions on six freeway corridors

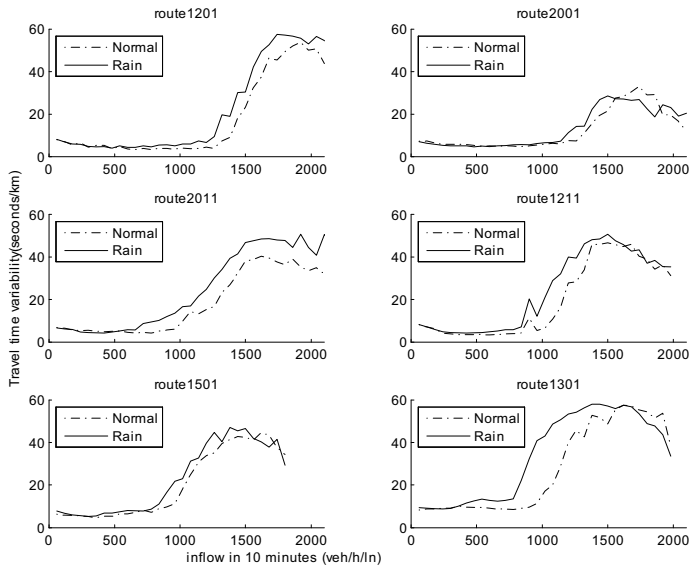


Figure 5.16: Travel time variability as a function of inflow level under both normal weather and rainy weather conditions.

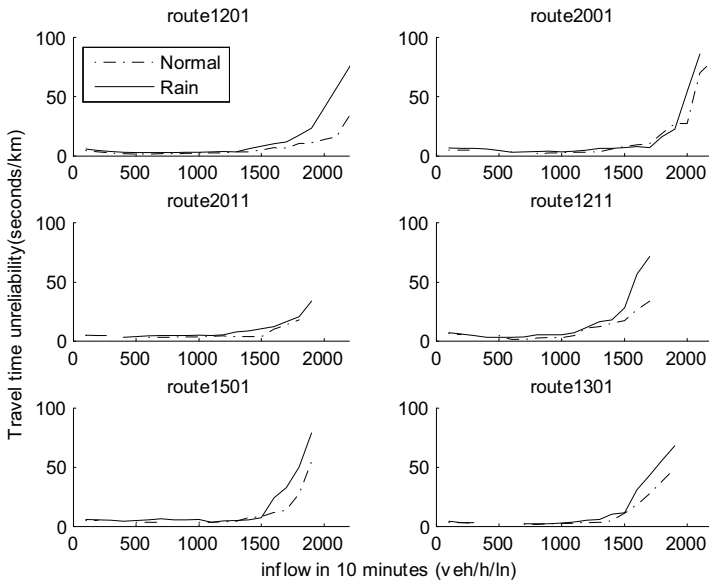


Figure 5.17: Travel time unreliability as a function of inflow levels on six freeway corridors in The Netherlands

The reason that the adverse weather conditions result in the negative impacts on travel times probably are twofold: firstly, adverse weather conditions lead to the reduction in road capacity and hence higher probability of traffic breakdown; secondly, adverse weather conditions may contribute to more traffic accidents than normal weather conditions.

5.3.5 Conclusions

On a basis of empirical data from six freeway corridors, the preliminary results of the effect of adverse weather conditions are shown as follows:

- Adverse weather conditions clearly have negative effects on both travel time uncertainty and travel time (un)reliability of freeway corridors. Travel times are less reliable under adverse weather conditions than under normal weather conditions, especially at higher inflow levels.
- Adverse weather conditions have a slight influence on $TT10th$ and $TT50th$, but
- Adverse weather conditions have significant impacts on $TT90th$, travel time uncertainty and travel time unreliability
- Rainy weather conditions have little or no effect on travel time variability below λ_t , but have positive impacts on travel time variability above λ_t .

5.4 Speed Limits Impacts

The steady increase of traffic demand on freeways during the past decades has led to high rate of congestion in The Netherlands. This has resulted in long delays for individual drivers but also to a high level of air pollution along the roads. Increasing the capacity of the freeway by increasing the number of lanes is a solution which is not always acceptable or even preferable to alternative approaches. In Europe the legislation prohibits the extension of roads if the air quality around the roads does not satisfy the legal requirements. An approach to reduce the air pollution consists of exerting some kind of control over the flow of vehicles by means of signals, with the objective to reduce the speeds.

Over the last two decades many studies have been done to investigate the relationship between average travel speed and emission rates. Laboratory tests, empirical measurements and model calculations have lead to a good understanding of emission dependence of average speed (van der Meer April, 2007). van der Meer (April, 2007) shows that the average speed has an effect on NO_x and PM_{10} emission rate. Most vehicles have the cleanest operations at a speed between 60km/h and 80km/h. In order to reduce the emissions, the (static) speed limits were implemented on freeways around Delft region like A13 freeway and A20 freeway. However, little is known about the effects of Speed Limits on travel time reliability. Therefore, in this section, we try to quantify the effect of SL on travel time reliability (route level) using a statistical analysis on the basis of empirical data. Before we address the SL impacts, we outline the effect of SL on traffic flow operations from the literature in section 5.4.1. Thereafter, the methodology to investigate the effects of SL on travel time reliability is setup in section 5.4.2. Then the related empirical data are presented in section 5.4.3. The preliminary results are shown in section 5.4.4.

5.4.1 Overview of the effects of speed limits on traffic flow operations

A great number of studies have researched the impact of SL on traffic flow operations, which can be categorized into four predominant aspect: impact on (local) speed, impact on traffic dynamics, impact on traffic safety, and impact on environment.

Firstly, several studies have shown that SL have significant impact on speed (Upchurch & Rahman 1989) (Aljanahi et al. 1998) (Ulfarsson et al. 2005). Those studies conclude that SL causes a statistically significant reduction in mean (local) speed. The impact of SL on speed dispersion does not seem to have a definitive trend. For example, in Arizona, after a rise in the SL on the rural interstate from 55 to 65 mph, an increase in (local) speed dispersion was observed (Upchurch & Rahman 1989). However, Ulfarsson et al. (2005) reported that SL reduced speed deviation for the uphill direction, which is a beneficial effect, but increased the speed deviation for the downhill direction.

Secondly, two views on the effects of SL on traffic dynamics can be found (Smulders 1990) and (Hegyí et al. 2005). The first emphasizes the homogenization effect, whereas the second is more focused on the prevention of traffic breakdown. The idea of homogenization is that SL reduce the speed differences between vehicles which are expected to result in a higher (and safer) traffic flow. The traffic breakdown prevention approach focused more on the prevention of too high densities. This is best realized if SL are dynamic and change according to the traffic conditions.

Thirdly, as far as traffic safety is concerned, a large number of studies has pointed to the increase in fatalities on these high-speed freeways when SL were relaxed (Garber & Graham 1990) (Godwin 1992) (Rock 1995) (Vernon et al. 2004) (Wong et al. 2005) (Lee et al. 2006). For example, Wong et al. (2005) used the SL relaxation on 19 road sections in Hong Kong and analyzed its impact on accident counts. The results show that after the relaxation of SL the accident counts went up by about 20-30%. After all other possible factors were neutralized by the comparison group method, they concluded that the relaxation of SL did have an adverse impact on road safety.

Fourthly, SL affect fuel consumption and emissions (den Tonkelaar 1991) (den Tonkelaar 1994) (Riemersma et al. 2004) (Int Panis et al. 2006). For instance, the fuel consumption and the CO , NO_x , CO_2 and hydrocarbon emissions from passenger cars and lorries on freeways were calculated by den Tonkelaar (1994) based on the actual driving speeds on both types of road before and after the introduction of the SL (from 120km/h to 100km/h) in The Netherlands. It is found that after the introduction of the SL and the decrease of the average driving speeds, fuel consumptions and emissions on these freeways decreases significantly.

5.4.2 Methodology

In the literature, tight SL reduces mean (local) speed, affects capacity, increases traffic safety, and reduces the fuel consumption and emissions. However, few of the studies carried out research on the effects of SL on travel time reliability. Travel time reliability not only depends on the travel time variability, but also relies on the probability of traffic breakdown. In terms of travel time variability, Tu, van Zuylen & van Lint (2007) investigated the effects of speed limits on travel time variability on two freeway corridors. They reported that SL clearly has impact on travel times. Tight SL result in larger $TT10th$ and $TT50th$ in comparison with relaxing SL. Tight SL do not lead to higher $TT90th$ than relaxing SL in case of high inflows. With respect to traffic breakdown, speed limits could prevent a breakdown by limiting the inflow into the tight speed limits area (for instance, the 80km/h area, see Figure 5.18). But inside the tight SL area, the probability of traffic breakdown may increase due to tight SL in case of high inflows since small disturbance may lead vehicles with the speed of 80km/h to drop below 70km/h (breakdown).

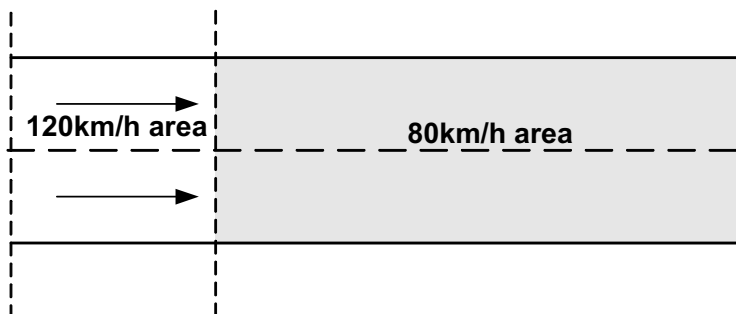


Figure 5.18: Illustration of a freeway corridor with one area in 120km/h and one are in 80km/h

Therefore, it may be expected that tight SL reduce the travel time variability, but increase the probability of traffic breakdown. Thus, whether the effects of SL on travel time reliability is positive is unknown. In the rest of this section, the effects of SL on travel time variability, traffic breakdown, and travel time unreliability will be tested on a basis of a large empirical dataset.

5.4.3 Empirical data

Dynamic or static Speed Limits (SL), are one of the Variable Message Systems control measures applied on Dutch freeways by the display of speed signals on gantries over the road. These gantries are at places where also detectors are located that provide speed measurements and count data. On the A13 freeway SL were imposed of 100 km/h for the first half of the road and 80 km/h on the part of the road close to Rotterdam. The SL of 80km/h on the last part of the A13 freeway was introduced to investigate whether environmental effects of the freeway could be reduced. The road passes through a suburb of Rotterdam and the air quality and noise around the road need to be improved considerably. The experiment had the objective to show the environmental benefit and to investigate what the impact would be on the travel times and throughput. The result were positive, the travel time extensions were limited, no more congestion occurred and the air quality improved slightly.

After that positive experience the SL were also applied on other roads, like the A20 freeway. The results were less good, because congestion increased on several roads. The A13 was not a bottleneck itself, so a SL did not have much influence on the throughput of the road. The A20 was a capacity bottleneck itself, so that the congestion increased thereafter the introduction of the SL.

Two related freeway corridors (routes) are selected from Regiolab-Delft, as shown in Table 5.6. The corridors are 7.79 km and 7.655km, respectively. The freeway corridor on A13 we tested in this dissertation is the part of A13 freeway in which SL were imposed of 100km/h.

Table 5.6: Physical data of two freeway corridors

Code	Freeway	Direction	Length(m)
1301	A13	From The Hague to Rotterdam north	7790
2001	A20	From Rotterdam west to Rotterdam east	7655

5.4.4 Results and analysis

Figure 5.19 illustrates the probability of traffic breakdown as a function of inflow levels under tight speed limits and relaxing speed limits on two freeway corridors. In the graph, tight SL slightly increases the probability of traffic breakdown on A13 freeway, but substantially increases the probability of traffic breakdown on A20 freeway, especially in case

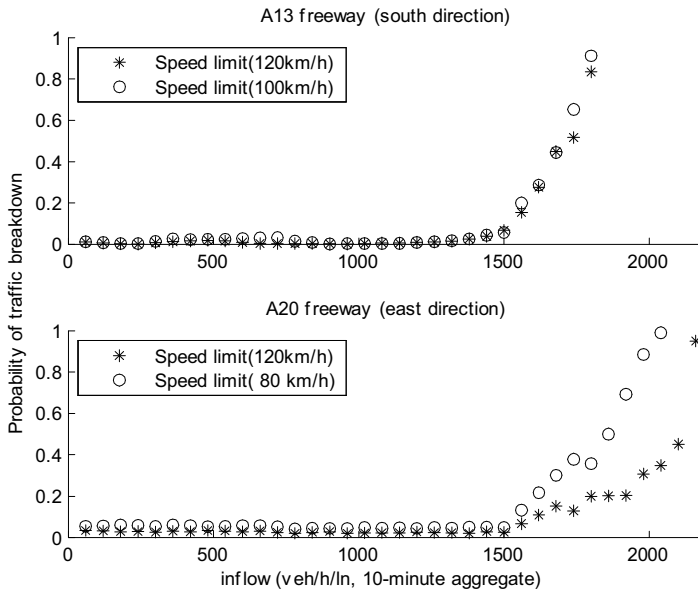


Figure 5.19: Probability of traffic breakdown as a function of inflow levels under tight speed limits and relaxing speed limits on two freeway corridors

of high inflows. This is probably due to A20 has more tight SL (A13: 100km/h and A20: 80km/h).

Figure 5.20 and Figure 5.21 illustrate the empirical relationship of TTV before breakdown and after breakdown as a function of inflow on A13 freeway and A20 freeway, respectively. Clearly the empirical data provide evidence that the tight SL results in lower both TTV^f and TTV^j on both freeway corridors. The higher SL difference (A13 from 120km/h to 100km/h, A20 from 120km/h to 80km/h), the larger TTV difference between relaxing SL and tight SL.

Figure 5.22 demonstrates travel time unreliability as a function of inflow levels on two freeway corridors. For A13 freeway, below a certain inflow level (1500 veh/h/ln), tight SL increases travel time unreliability by an average of 13.5%; above the certain inflow level, tight SL reduces travel time unreliability by an average of 21.6%. For A20 freeway, it holds an opposite trend. Below a certain inflow level (1500 veh/h/ln), tight SL reduces travel time unreliability by an average of 33.9%; above the certain inflow level, tight SL increases travel time unreliability by an average of 59.1%.

To conclude, tight SL reduces the travel time uncertainty, but increases the probability of traffic breakdown. Therefore, the effects of tight SL on travel time reliability highly depend on the SL value.

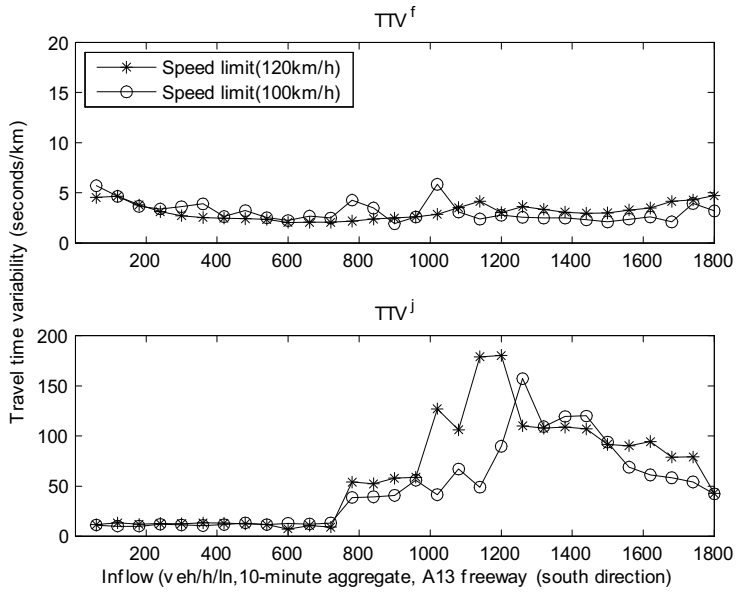


Figure 5.20: Travel time variability before and after traffic breakdown as a function of inflow levels on A13 freeway

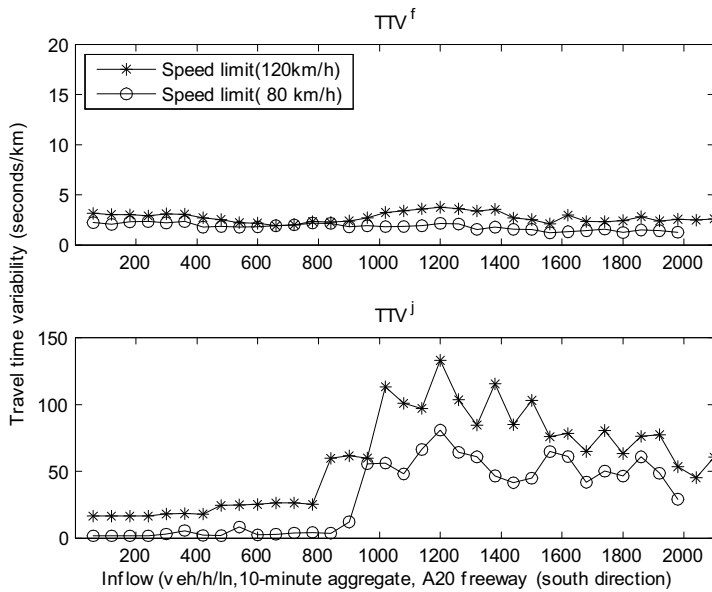


Figure 5.21: Travel time variability before and after traffic breakdown as a function of inflow levels on A20 freeway

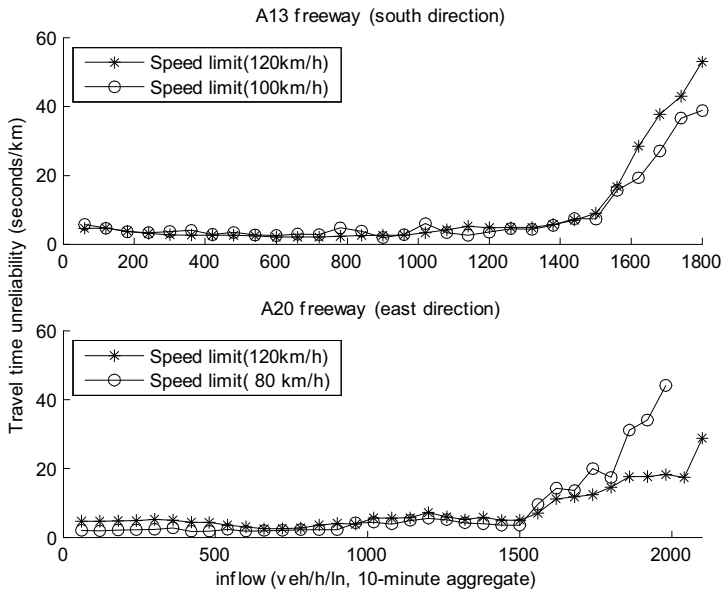


Figure 5.22: Travel time unreliability as a function of inflow levels on two freeway corridors

5.4.5 Conclusions

Tight SL reduce mean speed and fuel consumption and emission. The effect of tight SL on travel time reliability is still unknown. In this section, an empirical analysis is carried out to investigate the effects. The data presented here demonstrates following results:

- SL clearly has impact on travel times. Tight SL results in larger TT_{10th} and TT_{50th} in comparison with relaxing SL.
- Tight SL reduces the travel time uncertainty (variability)
- Tight SL increases the probability of traffic breakdown, and
- The effects of tight SL on travel time unreliability depends on the tight SL value.

5.5 Traffic Accidents Impacts

Studies have indicated that accidents contribute to one of the main parts of the freeway delay (Sheu et al. 2001). This may increase in the coming years. It is clear that accidents on freeways interrupt traffic flows unexpectedly, and thus, are a major cause of extra congestion. They can reduce the capacity of a road and act as bottleneck to the traffic flow. In turn, this perturbation of the flow can cause more accidents due to e.g. stop-and-go manoeuvres. The unpredictability of traffic accidents and its increasing effects

on freeway traffic congestion undermine significantly the mobility on freeways. In this section, we therefore aim to gain insight into the relationship between traffic accidents and travel time reliability by linking speed and flow measurement data from loop detectors to accident records collected on some freeways in the Netherlands.

5.5.1 Overview of the impacts of traffic accidents on traffic flow operations

Information on causation and contributory factors of traffic accidents have been studied for a long time (for example, (Carsten et al. 1989) (Conche & Tight 2006) (Knoop et al. 2007) (Knoop et al. 2008)). In the literature, however, few researches focus on the impact of traffic accidents on traffic flow operations.

Goolsby analyzed 27 accidents that occurred between 1968 and 1969 on a 10.5-km section of the Gulf Freeway in Houston using 1-min volume counts (Goolsby 1971). Based on these data, Goolsby estimated that an accident where a vehicle blocks one of three lanes will result in an average capacity reduction of 50%. Furthermore, he concluded that an accident blocking two of three lanes will reduce capacity by an average of 79%. By simply blocking the shoulder lane(s) on a three-lane segment it was found that capacity may reduce by an average of 33%.

Simth et al. (2003) reported that capacity reduction due to an accident was measured for over 200 accidents that occurred on urban freeways in the Hampton Roads region of Virginia. They found that accidents significantly reduce the capacity remaining on freeway segments. An accident blocking one of three freeway lanes resulted in a mean capacity reduction of 63%, while an accident blocking two of three freeway lanes resulted in a mean capacity reduction of 77%. They also suggested that accident capacity reduction should be modeled as a random variable, not a deterministic value.

Chen et al. quantified the effect of accidents on travel times by obtaining the accident records from the California Highway Patrol (CHP) in which each accident record has a start and an end time, a type classification, and a location (Chen et al. 2003). They concluded that both the standard deviation and the median of travel times are larger when there are accidents. The time delay of accidents is about 5 minutes per accident per vehicle.

Knoop et al. (2008) collected high quality videos of the traffic flow around two accidents on Dutch freeways recorded from a helicopter in 2007 and measured the two-directional traffic passing the accident locations. During the accidents, drivers would take the time to watch the accident ("rubbennecking" effect) for both directions of the freeway on which accident occurred. They concluded that the capacity of the road in the direction of the accident is reduced by more than half as not all lanes are in use and the capacity at the opposite direction is reduced by half by the "rubbennecking" effect.

Although these studies provided evidence that traffic accidents significantly reduce freeway capacity and increase travel time variability, none of the studies directly investigated the impact of traffic accidents on travel time reliability under different demand (inflow) levels. The next subsection will set up the method to investigate the accidents' impacts using empirical data from Regiolab-Delft monitoring system.

5.5.2 Methodology

Since traffic accidents result in lower freeway capacity and they can reduce the capacity of a road and act as bottleneck to the traffic flow, it is expected that traffic accidents lead to the higher possibility of traffic breakdown for a certain inflow level and result in higher travel time uncertainty as well. Once traffic accidents lead to traffic breakdown, the travel time uncertainty (variability) due to accidents is much worse than that of normal breakdown. In the Dutch freeway case, the average number accidents is about 20-30 accidents per km of freeway per year. Although traffic accidents contribute to the extreme long delays on freeways, the impacts of traffic accidents on reliability of travel times are still unknown. Therefore, below three hypotheses are made to test the impact of traffic accidents on travel time reliability:

- Traffic accidents lead to the higher probability of traffic breakdown for a certain inflow level
- Traffic accidents result in a higher travel time variability
- Traffic accidents contribute to less reliable travel times

5.5.3 Empirical data

In order to investigate the impact of traffic accidents on travel time reliability of freeway corridors and test the above hypotheses, detailed traffic accidents and traffic flow data were collected on a motorway that connects the city of Rotterdam with its harbour.

The Harbour of Rotterdam is connected to the hinterland by waterways, railways, pipelines and one motorway, the A15. The fact that incidents often occur on the A15 motorway is well known; the number of accidents per kilometer road in the considered area is about 50% higher than on another motorway like the A12 (Knoop et al. 2007).

For the study, we used a list of all police-registered accidents in the Netherlands. The list contained the accidents that happened in the years 2003 – 2005. The list was provided by the Transportation Research Centre of the Ministry of Transport, Public Works and Water Management (“Rijkswaterstaat Dienst Verkeer en Scheepvaart”). Each accident that is reported by the police is written down in this file. Therefore, the data source we used only gave the registered accidents (the police patrols are not called for every accident). However, most of the accidents on the motorway are included in the file (Knoop et al. 2007). Also locations, parties involved and other properties of the accident are reported. We filtered the data and considered the part concerning the sections on the A15 freeway. This section connects the ring-road of Rotterdam with the port area, and it consists of a large number of merging, diverging and weaving sections, ending with the junction “Ridderkerk-South” (connection to the A16-south).

The duration of traffic accidents was not reported. But the time traffic accidents occurred was recorded in the traffic accident data. In this study, every accident that occurred we take a temporal pattern, i.e. from the time the accident occurred and for the following 3 hours on that particular day. This pattern is considered as traffic data in case traffic accidents occurred. Then the remaining part of the data is considered as traffic data without

traffic accidents. The assumption of 3 hours here is rather arbitrary. The effects of traffic accidents (or the accidents duration) may be overestimated by using this assumption. However, in order to avoid the effects of traffic accidents on the traffic data without traffic accidents, we choose the assumption of 3 hours. Once traffic accidents occurred in a section along a freeway corridor, the whole freeway corridor is considered to be affected. Therefore, we have two datasets:

1. *Traffic data with accidents*: all data include the data that traffic accidents occurred
2. *Traffic data without accidents*: data exclusive the data that traffic accidents occurred

5.5.4 Results and analysis

After the data was collected and processed, a total of 52560 10-minute travel time records were available for the A15 freeway corridor. The dataset contained 1838 records with the influences of traffic accidents (only covers 3.5% of the total traffic data) and 50722 records without the impacts of traffic accidents (96.5% of the total traffic data).

Travel time variability

Figure 5.23 shows the travel time variability without accidents and with accidents as a function of inflow levels on A15 freeway in the Netherlands. Traffic accidents increase travel time variability by an average of 1.9% before traffic breakdown. After breakdown, traffic accidents increase travel time variability by an average of 27.0%.

Probability of traffic breakdown

Figure 5.24 shows the probability of traffic breakdown as a function of inflow levels under both conditions of with and without traffic accidents. The probability of traffic breakdown increases with rising inflows not only in case without accidents but in case with accidents. Since traffic accidents reduce the capacity of freeway, the graph shows that traffic accidents increase the probability of traffic breakdown by an average of 8.4%. Therefore, the instability of traffic flows under traffic accident conditions is much higher than that without traffic accidents.

Travel time unreliability

Traffic accidents result in higher travel time variability and higher probability of traffic breakdown. Moreover, travel time reliability is caused by both travel time variability and probability of traffic breakdown for a given inflow level. Therefore it is expected that traffic accidents lead to less reliable travel time. Figure 5.25 clearly provide evidence that travel time unreliability with traffic accidents is 7.8% higher than without traffic accidents. The influence of traffic accidents on travel time reliability is not as serious as one might assume at first sight (one may assume traffic accidents are the main sources of travel time unreliability).

5.5.5 Conclusions

The empirical analysis on a basis of a large dataset of registered accidents records and traffic flow data from loop detectors indicate that traffic accidents have significant impacts on travel time reliability of freeway corridors:

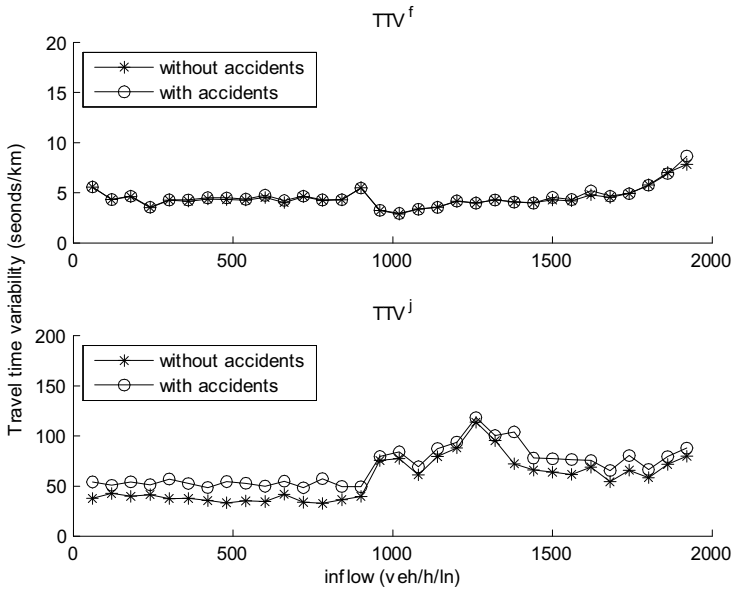


Figure 5.23: Travel time variability as a function of inflow levels under conditions both of with and without accidents on A15 freeway in the Netherlands

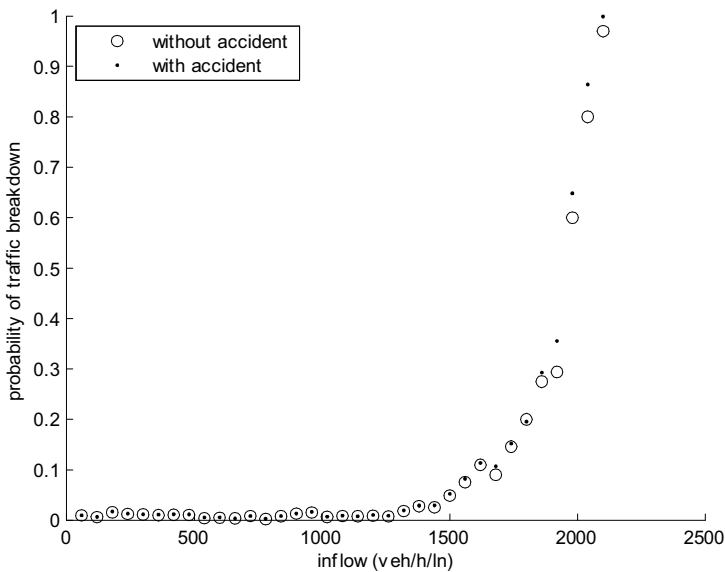


Figure 5.24: Probability of traffic breakdown as a function of inflow levels under both conditions of with and without accidents on A15 freeway in the Netherlands

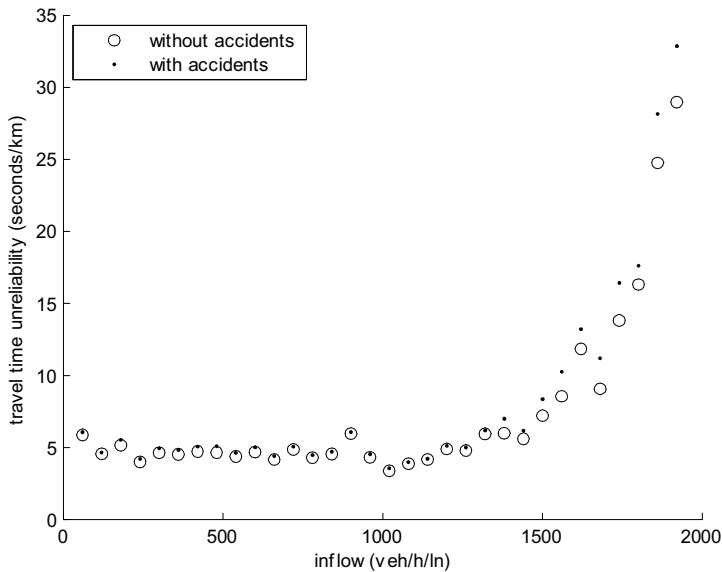


Figure 5.25: Travel time unreliability as a function of inflow levels under conditions both of with and without accidents on A15 freeway in the Netherlands

1. On both cases, before and after traffic breakdown, traffic accidents increase travel time uncertainty
2. Traffic accidents increase the probability of traffic breakdown on the freeway corridor
3. Traffic accidents increase the travel time unreliability, but
4. Traffic accidents are not the main source of travel time unreliability

5.6 Conclusions

Travel times are the result of the interactions among diverse factors like traffic flow, road geometry, adverse weather, etc. Travel time reliability comprises two elements: instability of travel times (probability of traffic breakdown) and travel time uncertainty (variability). It is expected that the factors which influence travel times also affect the probability of traffic breakdown and the travel time uncertainty and hence travel time unreliability. In this chapter the effects of four factors (road geometry, adverse weather, speed limits, and traffic accidents) on travel time reliability of freeways have been analyzed.

Firstly, we investigated the road geometry's impacts. The empirical results indicate that travel time unreliability increases with the decreasing weaving length or length of ramp section. In a freeway corridors, travel time reliability is strongly affected by the average

distance between off/on ramps (the number of ramps per unit road length β). Below a certain threshold value, 3km between ramps in Dutch freeway, travel time unreliability sharply increases with the decreasing distance. Therefore, in terms of practical design guidelines, the travel time reliability criterion on the freeway sections investigated here would require that (a) increasing the ramp length may result in more reliable travel times; (b) the distance between ramps in a freeway corridor should be larger than 3 km.

Secondly, the adverse weather conditions clearly have negative effects on travel time unreliability, increasing both the probability of traffic breakdown and the travel time uncertainty.

Thirdly, Speed Limits reduce the travel time uncertainty, but increase the probability of traffic breakdown. Therefore, the effect of tight SL on travel time reliability depends on the tight SL value.

Finally, traffic accidents lead to 1.9% higher travel time variability in free flow conditions only, but result in 27.0% higher travel time variability in congested conditions. Travel time unreliability with the influence of traffic accidents is 7.8% higher than without traffic accidents. Furthermore, traffic accidents are not the main cause of travel time unreliability.

To conclude, these four factors affect both the probability of traffic breakdown and the travel time uncertainty and thereby travel time reliability is influenced by these four factors.

With respect to the implications of the study for practice, this might imply:

- Longer ramp length or weaving length in a network design may lead to more reliable travel time.
- At peak hours or at adverse weather conditions, closing some on/off ramps along a freeway corridor (increase the distance between ramps) can improve the travel time reliability significantly.
- Tight Speed Limits reduce the emission on freeways. However, it may increase the unreliability in travel times and increase the mean travel time.
- In order to improve travel time reliability on freeways, more attention should be paid to the recurrent traffic congestion (or high demand) rather than to the effects of traffic accidents.

Chapter 6

Model Applications

6.1 Introduction

The travel time reliability model has been calibrated and validated in chapter 4 and chapter 5 using the empirical data from Regiolab-Delft traffic monitoring systems. As discussed in Chapter 1, the way travelers react on the reliability will affect their choice behaviors like mode, route and departure time choices. Since the choice behaviors play a crucial role in a Dynamic Traffic Assignment (DTA) model, the reliability-based traffic assignment model will produce more validated results. For instance, strategic departure time choice for the case of stochastic networks has been analyzed by Li et al. (2008) for homogeneous travellers with identical preferred arrival time. They assumed that travelers adapt their choice behavior based on not only the expected travel cost, but also the variability of travel time. They conclude that travelers depart earlier after a long term adaptation, when they consider travel time reliability as part of the travel cost, since they attach a safety margin to their travel times. The travel demand was spread over a longer time period.

The objective of the calibrated and validated model is, among others, to develop a methodology to use travel time reliability as an attribute in choice behaviors. Assessment of the role of travel time uncertainty both in route choices and in departure time choices and its impact on a network level is done by assigning the traffic to a network, taking into account that travelers attach a certain value to travel time reliability, next to travel time itself. In this chapter, the resulting, calibrated and validated travel time reliability model is applied in the traffic assignment model.

In section 6.2, the feasibility of the applications in traffic assignments will be discussed. Then a simple test network will be described in section 6.3. Thereafter, section 6.4 presents some experimental results and section 6.5 puts forward some discussions. Finally, section 6.6 draws the main conclusions.

6.2 Applicability in traffic assignments

Due to the stochastic supply and demand, the travel conditions vary within-day and over days. The unreliability of travel time and its role in traveller's choice behavior are becoming increasingly important issues involved in network modeling. It is well known

that a variety of travel time functions nowadays are available and can be directly used in traffic assignment models to determine route choice based on travel times. Travel time is the main component in the utility function to model travellers choice behaviors (Bliemer 2001). However, travel time reliability is another crucial component influencing travellers' choice behaviors (mainly the departure time choice, route choice, and mode choice behavior). As discussed in chapter 3, it is necessary to develop a set of DTA models which can be integrated the travel time reliability due to the fact that the traffic network itself is probabilistic and uncertain. Over the past decade, DTA concerned travel time reliability has been studied by (e.g. Mirchandani & Soroush (1987), Liu et al. (2002), Li et al. (2008)). Liu et al. (2002) proposed a stochastic dynamic user optimal model based on stochastic dynamic network with the assumption that route travel times are variable. The link travel time has two components: one is deterministic flow-dependent travel time and the other one is the stochastic delay which is modeled as a non-negative normal distribution. The proposed model captured the traveller's route choice characteristics such that a trade-off between a route with longer but reliable travel time versus another route with shorter but unreliable travel time. Hollander & Liu (2008) investigated the outputs of each single run of a Traffic Microsimulation Model (TMM) as estimates of traffic conditions on a single day. Running a TMM multiple times gives a range of travel time measurements since some parameters, such as in car following, gap acceptance and lane changing, are specified as a distribution, and the TMM draws different values from this distribution in every run. Li et al. (2008) claimed that under stochastic networks, modeling departure time choice is more important than modeling route choice. They assumed that the cost function for modeling travelers' departure time and route choice behavior is composed of three parts: expectation of travel time at any departure time t , expectation of schedule delay at time t and travel time reliability component which is represented by the standard deviation of travel time distribution at time instant t . Stochastic travel times are due to stochastic capacities in their research. The above studies with respect to the travel time reliability either assume travel times as a certain distribution like normal distribution or derive the travel time reliability like standard deviation of travel times from the simulations. Thus it is not directly possible to model travel behavior incorporating travel time reliability, especially in traffic assignments. It would be quite promising when some travel time reliability function would be available to facilitate traffic assignment with more accurate estimation or prediction of real traffic conditions. This thesis provides these possibilities to model travel time reliability directly as well as facilitate the reliability-based traffic assignment.

The cost function is assumed to be composed of travel time, travel time reliability, and schedule delay. We will focus on the cost function instead of the utility function throughout the remainder of this chapter. Generally, we have a time dependent travel time function on a link level $t_a(q_a(k), C_a)$ as a function of time dependent link volume and link capacity. Where $q_a(k)$ is the volume on link a at time instant k and C_a is the capacity of link a .

In order to compute the route travel time $t_r(q_{in}(k))$ we have to know the link travel times $t_a(\tau)$, denoting the travel time on link a for vehicles that enter the link at time instant τ ($\tau \geq k$). Travel time is additive by nature. Hence, the route travel time $t_r(q_{in}(k))$ have an additive structure such that they can be computed by adding the link travel times $t_a(\tau)$ on the consecutive links:

$$t_r(q_{in}(k)) = \sum_{a \in r} t_a(\tau) = \sum_{a \in r} t_a(\theta_{ar}^{od}(k)) \quad (6.1)$$

in which $t_r(q_{in}(k))$ is the actual travel time on route r when departing at time instant k and $\theta_{ar}^{od}(k)$ is the time instant at which vehicles enter link a when traveling along route r from o to d and departing at the origin at time instant k .

For the time instant of departure we take the middle of the time interval, $(k - 0.5)\eta$, such that for the first link on the route r :

$$\theta_{ar}^{od}(k) = (k - 0.5)\eta \quad (6.2)$$

where η is the size of each of the departure time periods.

We define the operator $\langle \cdot \rangle$ as the mapping from time instant to time interval by just looking in which time interval the time instant lies. For all links (except for the first link) on route r we compute $\theta_{ar}^{od}(k)$ as

$$\theta_{ar}^{od}(k) = \theta_{a-1,r}^{od}(k) + t_{a-1}(\langle \theta_{a-1,r}^{od}(k) \rangle) \quad (6.3)$$

As discussed in Chapter 2, since the schedule delay is more like the way how travellers react on the travel time unreliability, it is necessary to take both unreliability and schedule delay into account in the travel choice behaviors. A route cost function has many components like travel time, travel distance, toll charge, fuel consumption, travel time reliability, schedule delay etc. In this application the route cost function incorporated travel time, travel time reliability, and schedule delay which can be formulated as:

$$c_r(q_{in}^r(k)) = \alpha \times t_r(q_{in}^r(k)) + \beta \times TTUR(q_{in}^r(k)) + \gamma_1 \times \max\{0, ESD\} + \gamma_2 \times \max\{0, LSD\} + \epsilon_r \quad (6.4)$$

in which

$c_r(q_{in}^r(k))$	= travel cost on route r when departing at time instant k for a given inflow q_{in}^r
ESD	= Early Schedule Delay
LSD	= Late Schedule Delay
$\alpha, \beta, \gamma_1, \gamma_2$	= weights for travel time, travel time unreliability, ESD , and LSD , the so-called taste preference
$TTUR(q_{in}^r(k))$	= travel time unreliability on route r when departing at time instant k for a given inflow $q_{in}^r(k)$
ϵ_r	= the error term, accounting for model noise

The TLZ reliability function (see Eq. 4.7 or Eq. 6.5) can be applied in traffic assignments. The inflow can be obtained from the assigned flow in each time step of the traffic assignment. The other parameters can be estimated from the empirical data. Therefore,

the reliability function proposed in this dissertation can be easily embedded in the traffic assignment model.

$$TTUR(q_{in}^r(k)) = TTUR_0 \left(1 + \beta \left(\frac{q_{in}^r(k)}{\lambda_{tr}} \right)^\gamma \right) \quad (6.5)$$

in which

$TTUR(q_{in}^r(k))$	= travel time unreliability on route r for a given inflow rate $q_{in}^r(k)$ at time instant k
$q_{in}^r(k)$	= the assigned route inflow rates at time instant k
$TTUR_0$	= free flow travel time unreliability
λ_{tr}	= critical travel time unreliability inflow
β, γ	= parameters

6.3 Test network

To illustrate the model described in the previous chapters, an example is included here. The proposed travel time reliability model (TLZ function) is applied to a small hypothetical two-link network (see Figure 6.1) with a single OD pair (o, d) . Table 6.1 shows the free flow travel times and other characteristics of the two links. The dynamic (reliability-based and non-reliability based) traffic assignment model (Li et al. 2008) is used for computing a dynamic user-equilibrium. The assignments follow the strategic departure time choice using Vickrey's bottleneck model, assuming that travelers are fully aware of the stochastic properties of the travel time and schedule delay distributions at all departure times with or without the influences of travel time reliability. The base OD travel demand is 2000 veh. The fitted travel time reliability model is used in the utility (cost) function of the traffic assignment model.

The reliability-based dynamic traffic assignment underscores the importance of knowing how travel time and its reliability are valued by travellers. Many qualitative and attitudinal studies of travel choice behavior have found that the reliability are rated by users as a very important feature, affecting both their perceptions and levels of use of the different modes (see e.g. Abdel-Aty et al. (1996), Lam (2000), Bates et al. (2001), Chen & Recker (2001), Bogers & van Zuylen (2004), de Jong et al. (2006)). De Jong et al. (2006) recommended the value of (un)reliability (the standard deviation of travel times) for car travel is 0.8, for public transport is 1.4 and for freight transport is 1.24. Yet, travelers seem to value more highly a reduction in variability than in the mean travel time for a journey (for example, Abdel-Aty et al. (1996), Bates et al. (2001)). Although these studies took standard deviation as the indicator of travel time unreliability and they reported different values on travel time reliability, the value of time $\alpha = 1$ and the same value of (un)reliability $\beta = 1.2$ in the cost function of the DTA (Eq. 6.4) are used for the simplicity sake. Whether the value β of standard deviation is valid for the percentile travel time should be further studied.

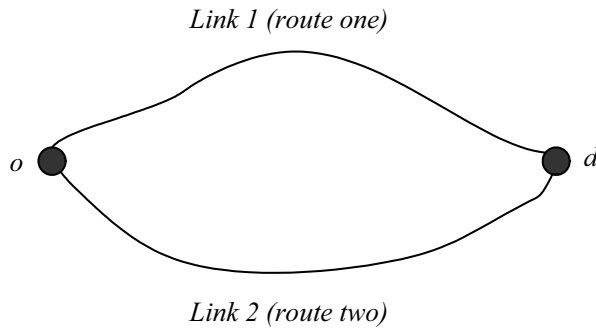


Figure 6.1: Two-link network with a single OD pair

Table 6.1: Link characteristics

	free flow travel time (minutes)	link length (km)	capacity (veh/h)	maximum speed (km/h)
link 1	5	10	1800	120
link 2	6	10	2100	100

6.4 Experimental results

Figure 6.2 presents the results on the long term equilibrium departure flow patterns. It can be seen that the departure patterns are significantly different with or without including travel time reliability in the utility (cost) function. Travelers depart earlier or later when they consider travel time reliability as part of the travel cost, since they attach a safety margin for their travel times. This finding also confirms the conclusions of (Li et al. 2008), who conducted an analytical investigation of strategic departure time choice under stochastic capacities using Vickrey’s bottleneck model and found that travelers compensate for the unreliability by departing earlier.

Travel time unreliability affects route choice as well. Given the same inflow level, travel times on route two are less reliable than on route one. 50.5% travelers choose route two when they don’t include reliability in the utility function while 51.9% travelers choose route two when they do include reliability in the utility function. Thus, travelers prefer the more reliable route (Figure 6.3).

6.5 Discussions

In this dissertation, we present a route-level reliability function in terms of these two attributes, which is considered as a function of several factors like inflow. The test network illustrated here is a simple two-link network (Figure 6.1), which shows that the TLZ

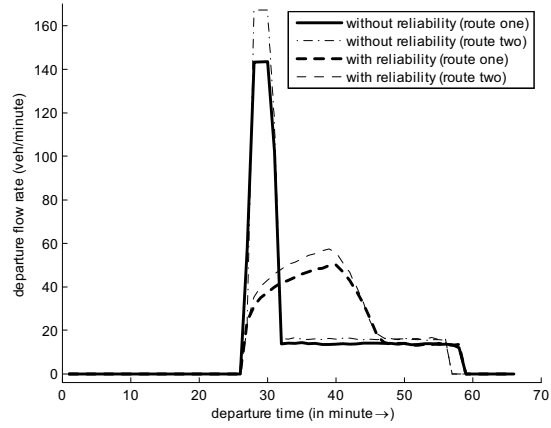


Figure 6.2: Departure flow patterns with and without reliability in the dynamic traffic assignment model

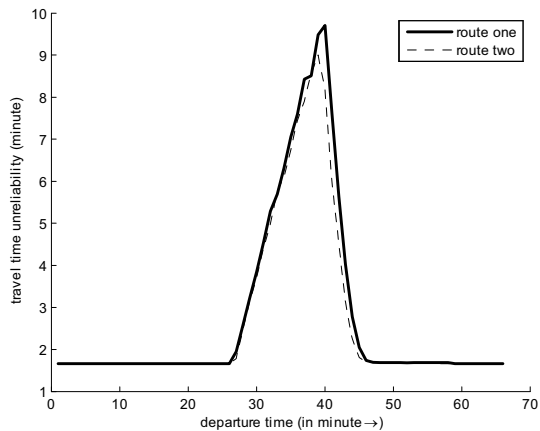


Figure 6.3: Unreliability as a function of departure time in the dynamic traffic assignment model

function provides a better face-validity in DTA models. The *TLZ* function can be applied to larger networks as well since the *TLZ* function is a route-based reliability function which uses one variable, inflow, and three parameters (including the critical travel time unreliability inflow λ_{itr}). The inflow can be obtained from the flow assigned to the route, while the parameters depend on the network characteristics, like link capacity, average distance between ramps, etc.

The relationship between the breakdown probability of a route and the breakdown probability of sections has been formulated in this dissertation. However, the relationship for travel time unreliability between a route and sections can not be derived due to the fact that: the travel time unreliability is determined by both the breakdown probability and the travel time uncertainty; the travel time uncertainty has a strong correlation among the adjacent sections of a route. Yet, the proposed *TLZ* model is a route-based function, in which the critical travel time unreliability inflow is determined by multiple factors like road geometry, adverse weather, traffic accidents, traffic control measures etc.

6.6 Conclusions

Travel time reliability, a crucial cost component which influences traveller's choice behavior (mainly regarding departure time and route choice behavior), can not be derived directly in the way travel time can be. Thus, it causes difficulty to model travel behavior incorporating travel time reliability. In the traditional approach, the travel time reliability can be obtained from the results of numerous simulations. The travel time reliability model proposed in this dissertation is regarded as a function of the inflow and of multiple factors like road geometry, traffic control etc. Meanwhile, the multiple factors can be directly obtained from inputs or outputs of the traffic assignment models. Therefore, it is quite promising that the travel time reliability model can be used within the traffic assignment model to estimate or predict traffic conditions. In this chapter we have presented the application of the travel time reliability model in a reliability-based (dynamic) traffic assignment models. A simple dual-route network was considered in the application putting emphasis on the route choice and the departure time choice behaviors with and without reliability in the dynamic traffic assignment model. The calculation for a small network shows that the reliability model can be easily integrated in the simple assignment and shows that travel time reliability affects the traffic assignment through not only departure time choice but also route choice.

Chapter 7

Conclusions and Further Research

This closing chapter summarizes the main conclusions of the work presented in this thesis. Thereafter, we describe how the presented model is applied in the traffic assignment model. We conclude this chapter with directions for future research.

7.1 Conclusions

In this dissertation we clarified the components of travel time reliability and established a new travel time reliability model. Correspondingly, the conclusions of this thesis are summarized into three categories: conceptual model, inflow-travel time reliability model, and extended travel time reliability model. Thereafter, we summarize the validation of the model and the model application.

7.1.1 Conceptual travel time reliability model

1. A generic travel time reliability model has been developed as a function of a variety of factors. In essence, these factors are conditionals, that is, the function expresses travel time reliability for a certain inflow level, given certain road characteristics, and given all other relevant factors such as traffic control measures, the prevailing traffic state, and external factors like weather, luminance, etc.
2. The empirical characteristics of travel time-flow and speed-flow relations provide the evidence that there is no one-on-one relationship of speed and flow or travel time and flow, neither in the free flow, nor in the congested branch of the fundamental diagram. The measurements of both relations are widely scattered. It appears to be that travel times have three regions: (a) below critical transition inflow, certain and stable travel times, (b) between critical transition inflow and critical capacity inflow, uncertain and unstable travel times, and (c) above critical capacity inflow, certain but unstable travel times.
3. Based on the above empirical analysis, travel time reliability includes two parts: uncertainty (variability) in travel times and instability of travel times, of which the

latter is closely related to the probability of traffic breakdown. Travel times are unreliable if (a) travel time is uncertain, (b) travel time is currently certain yet unstable in the next moment, or (c) the consequences of instability (breakdown) are severe.

4. The *Conceptual Travel Time Reliability* (CTTR) model expresses travel time unreliability as a function of a variety of factors, which includes both the variability in travel times and the probability of traffic breakdown.
5. The probability of traffic breakdown on a certain section is categorized into the spontaneous breakdown and the induced breakdown. In this context, the probability of traffic breakdown of a route is formulated as the product of the probability of traffic breakdown of adjacent sections along the route.

7.1.2 Inflow-travel time reliability model

1. The travel time reliability model is considered as a function of the principal factor: inflow. Inflow denotes exclusively vehicles entering the studied freeway section or corridor at the upstream entry of the main carriageway, which does not include the flow of on- and off-ramps along the freeway corridor. Moreover, the measured inflows are aggregated on 10-minute time periods.
2. A general framework for empirically setting up the CTTR model is presented, in which there are several key elements such as data sources, data cleaning, historical database etc. Since there are no actual travel times measured in Regiolab-Delft monitoring systems (van Zuylen & Muller 2002), the so-called *Piece-wise Linear Speed Based (PLSB)* algorithm is used to estimate travel times on routes of adjacent freeway sections (van Lint & van der Zijpp 2003). The PLSB method reconstructs vehicle trajectories and hence mean travel times based on time series of speed and volume measurements of consecutive detector locations along a route.
3. The large empirical dataset applied in this research indicates that there exists two critical inflow levels, namely critical transition inflow λ_t and critical capacity inflow λ_c in the (static) travel time variability-inflow relations. It is also found that when volumes are (a) below λ_t , the variability in travel times are low; (b) between λ_t and λ_c , travel time variability sharply increases while the rising inflows, and (c) above λ_c , travel time variability decreases while the inflow increases. However, above λ_c travel time reliability can not be relatively high since in this region it is very likely that the flows will break down soon.
4. In this context, the CTTR model which covers both the variability in travel times and the instability of travel times is tested based on the empirical data. It is found that both the probability of traffic breakdown and travel time (un)reliability increase when the inflow increases, and
5. Travel time unreliabilities estimated by the CTTR model from the empirical traffic data are fitted to the TLZ (Tu, van Lint, van Zuylen) reliability function so that the TLZ function is more flexible than the CTTR model.

7.1.3 Extended travel time reliability model

1. The travel time reliability model is generic and extended to become a function of multi factors like road geometry, adverse weather, Speed Limits, traffic accidents, etc.
2. Travel times on freeways are variable due to variations both in demand and in capacity. One group of factors affecting capacity are geometrical characteristics of freeways, such as the number of ramps (on-ramp or off-ramp) and weaving sections per unit road length and their physical characteristics. The effects of road geometry on the probability of breakdown, travel time uncertainty and travel time unreliability are investigated. These studies find that there exists threshold values L for the length of ramps sections (both on/off-ramps and weaving sections). Below L , travel time unreliability increases with the decreasing length or ramps sections. Above L , the length has far less impact on travel time unreliability. In a freeway corridor, travel time reliability is strongly affected by the average distance between ramps. Below a certain threshold value, the shorter the distance between ramps, the less reliable the travel times will be. Such a finding can be of significant importance for geometric design standards in terms of travel time reliability, at least in the Regiolab-Delft case:
 - The length of deceleration lanes should be longer than 250m;
 - The length of acceleration lanes should be longer than 300m;
 - The weaving length should be longer than 750m;
 - The number of ramps per ten kilometers in a freeway corridor should be less than 6.8.
3. Adverse weather conditions may (locally) reduce the capacity in a traffic network, but may at the same time yield (global) changes in traffic demand, due to people changing routes, departure time, mode or even reconsidering the decision to make a trip all together. It is therefore expected that travel time reliability is strongly affected by adverse weather since travel time reliability results from the variations both in demand and capacity. On a basis of empirical data from six freeway corridors, the preliminary results of the effect of adverse weather conditions are shown as follows:
 - Adverse weather conditions clearly have negative effects on both travel time uncertainty and travel time (un)reliability of freeway corridors. Travel times are less reliable under adverse weather conditions than normal weather conditions, especially at higher inflow levels.
 - Adverse weather conditions have slight influence on $TT10th$ (10th percentile travel time) and $TT50th$ (median travel time), but
 - Adverse weather conditions have significant impacts on $TT90th$ (90th percentile travel time), travel time uncertainty and travel time unreliability
4. Tight Speed Limits (SL) reduce mean (local) speed, affect capacity, increase traffic safety, and reduce the fuel consumption and emissions. An empirical analysis is

carried out to investigate the effects of tight SLs on travel time reliability. It provides evidence that:

- SL clearly has impact on travel times. Tight SL results in larger *TT10th* and *TT50th* in comparison with relaxing SL.
 - Tight SL reduces the travel time uncertainty (variability)
 - Tight SL increases the probability of traffic breakdown, and
 - The effects of tight SL on travel time unreliability depends on the tight SL value.
5. It is clear that accidents on freeways interrupt traffic flows unexpectedly, and thus are a major cause of extra congestion. They can reduce the capacity of a road and act as bottleneck to the traffic flow. The unpredictability of traffic accidents and its increasing effects on freeway traffic congestion undermine significantly the mobility in urban areas. The empirical analysis in this dissertation indicates that traffic accidents have negative impacts on travel time reliability of freeway corridors:
- Traffic accidents increase travel time uncertainty
 - Traffic accidents increase the probability of traffic breakdown on the freeway corridor
 - Traffic accidents increase the travel time unreliability
 - Traffic accidents contribute to the less reliable travel times, but they are not the main source of travel time unreliability.

7.1.4 Model applications

In this subsection the newly developed travel time reliability model is applied in Dynamic Traffic Assignment (DTA) models. Travel time reliability, a crucial cost component which influences travelers' choice behaviors (mainly the departure time choice and route choice behaviors), can not be derived directly in the way travel time can be. Thus, it causes difficulty to model travel behavior incorporating travel time reliability, especially in traffic assignments. In the traditional approach, the travel time reliability can be obtained from the results of numerous simulations or the assumed travel time distribution. The *TLZ* function proposed in this dissertation is a route-based reliability function which is regarded as a function of inflow with a parameter λ_{tr} . The inflow can be directly obtained from outputs of the DTA models and the λ_{tr} is regarded as a function of factors like road geometry, traffic control measures etc, all of which are also available in a DTA context. Therefore, the implementation of the travel time reliability model into the traffic assignment model to estimate or predict traffic conditions, promises well. In a preliminary study, we tested the *TLZ* reliability model in a DTA model with a simple dual route network. The DTA experiments yield plausible and face-valid results, which confirm that the *TLZ* model can be easily integrated in the assignments.

7.2 Future Research

In this final section we contemplate upon the future research directions that naturally follow from the research described in this dissertation. The scientific and technological challenges are:

- The model could be made more generic, suited for all motorways under all conditions,
- Modeling the reliability of individual travel times, using monitoring data from single vehicles,
- Modeling reliability of travel times on urban road networks,
- The analysis of the additivity of unreliability of segments of routes.

Firstly, we will consider several model improvements which are expected to improve either the validity or application potential of the model. Secondly, we briefly consider application of the modeling approach to alternative traffic systems. Finally, we will consider other research directions.

7.2.1 Research directions for reliability model improvements

1. In this dissertation, the influences of the freeway characteristics on travel time reliability have been investigated. For this, we have looked at freeway characteristics like the length of ramp sections and weaving sections, and the number of ramps per unit road length. However, many other factors such as for example horizontal and vertical alignment, lining and lane-width affect travel time reliability as well. These could be the subject of further research.
2. Traffic control measures comprise a second strand of factors affecting travel time reliability. In this thesis only the impacts of static speed limits on travel time reliability have been discussed. The impact of control other measures such as dynamic speed limits, ramp metering, route guidance, and tidal lanes on travel time reliability could be investigated. on travel time reliability.
3. The same applies to the external effects we investigated. First of all, a more detailed characterization of weather circumstances than just the presence of precipitation (as used in this thesis) could yield more detailed insight into the effect of weather on travel time reliability. Also traffic composition (e.g. the percentage trucks) and illumination (day versus night) may affect travel time reliability. Finally, the unpredictability of weather conditions may also result in the unreliability of travel times.
4. Finally, more research is required to understand and quantify the effects of accidents and incidents on travel time reliability. For the accidents analysis, 3-hour is chosen as the accident duration, while distinguishing accident-related traffic data and accident-free traffic data. This is somewhat arbitrary. The accident duration

should be estimated more accurately. Moreover, accidents have different severities. The consequence of traffic accidents varies with the severity of it. If the sufficient data is available, the impacts of traffic accidents on travel time reliability, given a certain severity of accidents, can be investigated.

7.2.2 Research directions for alternative traffic systems

1. This dissertation validated the travel time reliability model for an inductive loop based traffic data collection system. An interesting and relevant research question is whether results of similar quality can be obtained using a different type of data collection system, specifically non-infrastructure bound system (e.g. based on GSM/GPS).
2. Traffic signals interrupt urban traffic periodically and cause delays which play an important role in travel time and hence travel time reliability. Accordingly, urban flows are affected by both the signal control delays and queuing delays while free-way flows are mostly affected by queuing delays only. Therefore the travel time reliability model, in its current form, should include at least one more parameter, traffic signals.

7.2.3 Other research directions

1. The question how to define and measure travel time variability (uncertainty) is still wide open, albeit that there is a strong case for using more robust measures based on percentiles such as travel time variability is considered as the difference between 90th percentile travel time and 10th percentile travel time.
2. The probability of traffic breakdown of a route is formulated as the product of the probability of traffic breakdown of adjacent sections along the route. However, the unreliability of a route can not be simply conceived as the sum of the unreliability of each of its subsections, due to the strong cross-correlations of traffic over space and time. Further research is needed to unravel the (most likely location and time specific) relationship between unreliability on different spatiotemporal scales.
3. The travel time reliability measures and models developed in this thesis are intuitive from a research and a DTA model-application perspective (they are e.g. expressed in travel time units per unit space). However, they may not be that intuitive from a policy perspective. Further research is needed to translate the findings in this thesis into policy implications (e.g. in terms of infrastructure and transport planning, and network design and management).
4. In this dissertation, both a section-based and a route-based travel time reliability model have been developed. A logical next step is to also look at travel time reliability on a network-level:
 - The integration of travel time reliability from different routes into a road network performance

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- The interpretation of travel time reliability as a performance indicator of a road network
 - The differences between reliability models on freeway and urban road networks

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Appendix A

Regiolab-Delft Traffic Monitoring Systems

Regiolab-Delft is a traffic laboratory with many participating organizations. Road authorities (the Netherlands Ministry of Transport, Public Works and Water Management, the province of Zuid-Holland, municipality of Delft), representatives of the traffic industry (Siemens and Vialis) and research and educational institutes (the Test center for traffic systems of the Ministry of Transport, CONNEKT, Research school TRAIL and the Delft University of Technology) work together. The Regiolab-Delft aims to collect traffic data from the region of Delft, to analyze the data and to integrate the information. Existing detection Equipment, such as the motorway loop detection system Monica of the Ministry and loop detection at controlled intersections in Delft, are extended with new and sometimes experimental means to detect traffic.

Based on these means of detection it will be possible to recognize traffic patterns, to find origin destination relations, to measure road user's reaction on dynamic traffic measurements, and to measure and predict travel times. Regiolab-Delft provide the participating organizations and if requested also the national and regional information centers (TIC) with combined information about the traffic condition in the total area (managed by different road authorities). Based on this information, existing dynamic models can be validated and new models can be developed. As an example, all results reported in chapter 4 and chapter 5 are based on data from the Regiolab-Delft laboratory.

Appendix B

Offline Travel Time Estimations

This appendix outlines the algorithm to (offline) estimate travel time from loop detectors data applied in this thesis. This algorithm is largely based on the work of van Lint and van der Zijpp (2003).

B.1 General Framework

This travel time estimator is referred to as *trajectory methods*. The objective of the trajectory method is to estimate travel times along a path of adjacent sections by means of reconstructing *imaginary vehicle* trajectories. Let us assume that each section has detectors measuring vehicle speeds at the up-and downstream edge respectively. Let us also assume that these detectors produce harmonic time averaged speeds for each measurement period p . Finally, let us define our path consisting of K adjacent sections, for which we have detector measurements. In an off-line situation the data provided by these detectors comprise a space time grid of regions $\{k, p\}$, $k \in [1, \dots, K]$, and $p \in [1, \dots, P]$, see Figure B.1. In this space-time grid, we only know prevailing local speeds of the detectors at the up-and downstream edges of each region for each measurement period p . Now let us suppose *imaginary* vehicles traverse this grid, starting at section 1 ($x = 0$) each r time-steps.

The headway r between consecutive vehicles at the starting points is usually referred to as the *resolution* of the trajectory method. For ease of notation let us define each region $\{k, p\}$ as a rectangular area in space-time with bottom left corner $\{x_0, t_0\}$ and top-right corner $\{x_1, t_1\}$. The trajectory algorithm for a single vehicle trajectory can now be schematically presented as follows:

Clearly, all we require to add points to the individual trajectories is the location in space time where they exit their current region $\{k, p\}$, which is emphasized by the grey box in Figure B.2. This exit-point determines in turn the entry-point of the vehicle in the next region, and allows us to deduce path-level vehicle trajectories, and hence path travel times.

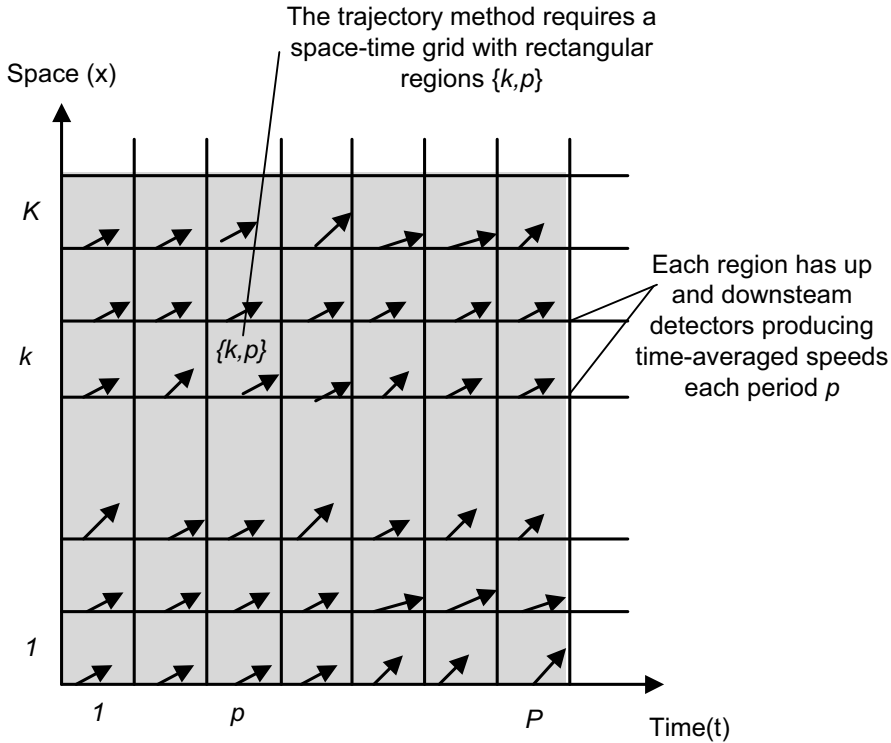


Figure B.1: Trajectory method requires a space-time grid with rectangular region $\{k, p\}$, which are enclosed between up-and downstream detectors and have (duration) length p .

B.2 Section Level Travel Time Estimators Based on a Linear Function of Speed

Vehicles are likely to anticipate to slower or faster speed regimes downstream and gradually adapt their speeds to it. We consider the speed $v_i(t)$ of a vehicle i traversing a section between detector location d and $d + 1$ as a function of the distance of that vehicle to these up-hand downstream detectors at x_d and x_{d+1} . We obtain:

$$v_i(t) = V(d, p) + \frac{x_i(t) - x_d}{x_{d+1} - x_d} (V(d + 1, p) - V(d, p)) \tag{B.1}$$

Let x_{ikp}^0 denote the entry location of a vehicle i in section k $[x_d, x_{d+1}]$ at entry time t_{ikp}^0 such that

$$x_i(t_{ikp}^0) = \begin{cases} x_{ikp}^0, & t_{ikp}^0 = t_0 \\ x_0, & t_{ikp}^0 > t_0 \end{cases} \tag{B.2}$$

Eq. B.1 is an ordinary differential equation, for which the solution reads:

$$x_i(t) = x_{ikp}^0 + \left(\frac{V(d,p)}{A} + x_{ikp}^0 - x_d \right) \cdot \left(e^{A(t-t_{ikp}^0)} - 1 \right) \quad (\text{B.3})$$

$$A = \frac{V(d+1,p) - V(d,p)}{x_{d+1} - x_d}$$

B.3 Trajectory Method Based on Piece-Wise Linear Speeds

We consider the speed on section k at a convex combination of the time average speeds at up-and downstream detectors. We first evaluate the conditions:

$$x_i^0 + \left(\frac{V(d,p)}{A} + x_i^0 - x_0 \right) \cdot \left(e^{A(t-t_{ikp}^0)} - 1 \right) > x_1 \quad (\text{B.4})$$

Consequently calculate the exit location and time with

$$\{x_i^*, t_i^*\} = \begin{cases} \left\{ x_1, t_i^0 + \frac{1}{A} \ln \left(\frac{\frac{V(d,p)}{A} + x_1 - x_0}{\frac{V(d,p)}{A} + x_i^0 - x_0} \right) \right\}, & \text{condition holds} \\ \left\{ \left(\frac{V(k,p)}{A} + x_i^0 - x_0 \right) \cdot \left(e^{A(t_i - t_i^0)} - 1 \right) + x_i^0, t_1 \right\}, & \text{otherwise} \end{cases} \quad (\text{B.5})$$

with

$$|A| > 0$$

Care must be taken with A values close to zero. This could lead to numerical problems. In practice this applies when the upstream and downstream observed speeds are nearly equal. Note that in these cases the assumption of piecewise constant speeds is justified, and hence below equation may be used instead.

$$\{x_{ip}^*, t_{ikp}^*\} = \begin{cases} \left\{ x_1, t_{ikp}^0 + \frac{(x_1 - x_{ikp}^0)}{V(k,p)} \right\}, & V(k,p) \cdot (t_1 - t_{ikp}^0) + x_{ikp}^0 > x_1 \\ \left\{ V(k,p) \cdot (t_1 - t_{ikp}^0) + x_{ikp}^0, t_1 \right\}, & \text{otherwise} \end{cases}$$

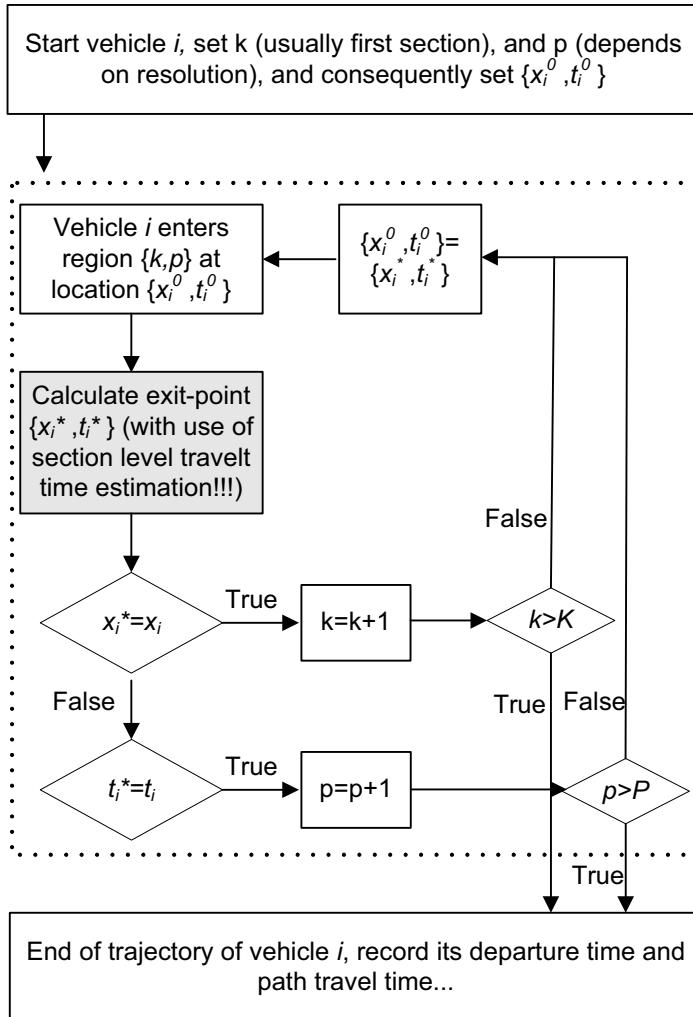


Figure B.2: Schematic representation of a trajectory method: different section level travel time estimators can be plugged in the framework easily (grey) box left center in the schema).

Appendix C

Data Cleaning

In this appendix, we briefly identify various approaches to tackle the missing data problem in traffic engineering.

1. *Null replacement*, that is, leave the data as is (i.e. incomplete), or (if the receiving model requires so) replace missing data with some default value(e.g. zero, one, -99 etc.), and let the model receiving the data handle the missing data problem. These default replacements are called *null* values.
2. *Simple imputation*, that is, replacing missing values by ad-hoc (statistical) procedures. These could include: the sample mean, median or other descriptive statistic, the last known value, a forecasted value by means of a time series or a regression model (even a neural network) or a spatial interpolate.
3. *Model based imputation*, which in essence is a special case of simple imputation. In this case missing values are replaced by procedures related to knowledge of the (physical) process generating the data, rather than statistical methods. Examples include for instance traffic flow simulation models in combination with Kalman Filters.
4. Multiple imputation, in which case the corrupted data set is replicated a number of times, say $N > 1$ times, each in which the missing data are replaced through some simple or model based imputation method. Then, with the N "complete" data sets. N predictions or inferences can be made, which can be statistically summarized. The key notion is that with Multiple imputation, the statistical properties of the sources data and the inherent uncertainty related to missing data are preserved.

In the Piece-wise Linear Speed Based trajectory algorithm (see Appendix B) imaginary vehicles traverse through a grid in space and time (Figure B.1) This grid is constituted of section k enclosed by up- and downstream detectors and periods p , during which each of these detectors produces harmonic time average speeds of all passing vehicles. The space time grid yields a rectangular data set of size $D \times P$ (No of Detectors \times No of Periods). If data are missing, obviously, in some cells $\{k, p\}$ no exit location can be calculated. Here we will employ the examples that the first three strategies described above to deal with these missing data.

Null replacement

For the strategy "Null Replacement" we do not fill in the gap with a default value, rather, we omit the particular detector output at that period and calculate exit locations and times at the first available measurement. There are two exceptions: (a) at the (spatial!) boundaries, if some measurement is missing, it is substituted with the last known value, and (b) if some imaginary enters a region where the upstream detector value is missing it is assumed to enter with its last known speed. In case of structural failure at the most up- or downstream detector on the route, obviously, no replacement is made: the resulting travel times will be biased in the negative direction. Note that in the methodological sense, the PLSB trajectory method is robust since it does not require the input data set to be complete, as long as an exit location downstream can be calculated.

Simple imputation through interpolation

The second strategy to be applied is simple imputation. Since the PLSB method is an offline method, we can employ interpolation in both the spatial and temporal direction, given the route is equipped with detectors $d \in \{1, \dots, D\}$ and a database of measurement U from these detectors periods $p \in \{1, \dots, P\}$ is available. The location of each detector is denoted by x_d . Suppose at some detector d during time period p no data are available, the spatial interpolation procedure we fill in this gap according to

$$U^{space}(d, p) = \begin{cases} U(d + d_a, p) & d + d_a \leq D \\ U(d - 1, p) + \frac{x_d}{x_{d+n} - x_{d-1}} U(d + d_a, p) & 1 < d < D \\ U(d, p - 1) & otherwise \end{cases}$$

in which $U(d + d_a, p)$ is the first available measurement in the spatial direction. Similarly, in the time direction we can repair the gap with

$$U^{time}(d, p) = \begin{cases} U(d, p + p_a) & p + p_a \leq P \\ U(d, p - 1) + \frac{1}{k+1} U(d, p + p_a) & 1 < p < P \\ U(d, p - 1) & otherwise \end{cases}$$

in which $U(d, p + p_a)$ is the first available measurement in the time direction. We will fill in the gap with the minimum of both interpolates (implying the maximum constraint of traffic throughput (flows) and travel time (speeds)), that is

$$U^*(d, p) = \min(U^{space}(d, p), U^{time}(d, p))$$

Model based imputation

In this case a first order Lighthill, Witham and Richards (LWR) model in combination with an extended Kalman Filter is introduced to deal with the missing data. Suppose that during period $p - 1$ all data are available from all detectors and (if present) on and off ramps connected to the sections. These available measurements then constitute the initial conditions. The LWR model is run for one measurement period p which yields predictions of density, flow (and speed) on each section. After each predictive step, each prediction of flows (and speeds) is corrected by means of an extended Kalman Filter. The Kalman Filter combines the model prediction with measurements and weights these two components by their (assumed) uncertainty.

Summary

Travel time reliability has a significant effect on route choice and departure time choice, and is generally conceived as an important factor, particularly for trips, such as journey-to-work, where time constraints (e.g. arrival time) may impose significant penalties on an individual. Recent empirical studies support this and suggest that travelers are interested in not just travel time saving but also in a reduction of travel time unreliability. Travel time reliability as a performance indicator of mobility has also entered the political arena. In the Dutch national transport policy (*Nota Mobiliteit*), for example, travel time reliability plays a central role and improving travel time reliability on the entire road network ("from door to door") is considered as one of the key objectives for the Ministry of Transport, Public Works and Water Management in the coming decade. To predict future traffic conditions on the transport network, policy makers and transport planners rely on tools such as Dynamic Traffic Assignment (DTA) models. However, until now no valid travel time reliability models are available which can be used in DTA models. In this thesis, such a model (and the underlying theory) is developed on the basis of empirical data. This new model quantifies the effects of traffic control and infrastructure design on travel time reliability in a DTA context and as such can help making decisions on measures to improve travel time reliability.

The characteristics of empirical (i.e. based on real data) travel time-flow and speed-flow relations provide the evidence that there is no one-on-one relationship of speed and flow or travel time and flow, neither in free flow, nor in congested conditions. The measurements of both relations are widely scattered. The data reveals that travel time as a function of flow has three regions: (1) below critical transition inflow, where travel times are certain and stable; (2) between critical transition inflow and critical capacity inflow, where travel times are both uncertain and unstable; and (3) above critical capacity inflow, where travel times are certain but unstable. Based on these observations, in this dissertation we define travel time unreliable (a) if travel time is uncertain; (b) if travel time is currently certain but unstable (meaning it might steeply increase in the near future due to traffic breakdown), and / or (c) if the consequences of instability (breakdown) are severe. Therefore, travel time reliability in our theory incorporates two elements: uncertainty (variability) in travel times and instability of travel times, the latter which is closely related to the probability of traffic breakdown, in the sense of the transition of free flow to synchronized flow.

The investigation of travel time unreliability is the process of quantifying these two elements. Whether a driver will experience traffic breakdown is seen as a risk. Risk is then characterized by two quantities: the probability (likelihood) of occurrence of breakdown and the associated consequence (magnitude). In this dissertation, a (Conceptual) Travel Time Reliability (CTTR) model is proposed, which is computed as the sum over

the products of the consequences (variability or uncertainty) and the corresponding probabilities of traffic breakdown. In this model, the probability of breakdown on a section is categorized into spontaneous breakdown and induced breakdown, which are supposed to be independent; the probability of breakdown of a route is formulated as one minus the product of the probability of non-breakdown of adjacent sections along the route.

In low flow conditions travel times are certain while the unreliability occurs in high flow conditions. Traffic flow is one of the most important factors (probably the most important one) influencing travel time reliability. Thus, the (in)flow of a road section can be considered as the principal parameter in the travel time reliability model. The CTTR model is considered as a function of a variety of factors. In essence, these factors are conditionals, that is, the function expresses travel time reliability for a certain inflow level, given certain circumstances. These circumstances include road characteristics, and all other relevant factors like traffic control measures, the prevailing traffic state (congested or not), and possibly external factors like weather, luminance, etc. The inflow-CTTR model is validated and calibrated on the basis of the empirical data collected from the Regiolab-Delft traffic monitoring system. It is found that both the probability of traffic breakdown and travel time unreliability increase with the increasing inflows.

Travel times on freeways are variable due to variations both in demand and in capacity. The CTTR reliability model is generic and has been extended to be a function of several of these “supply factors” like road geometry, adverse weather, Speed Limits, traffic accidents, etc. After developing the theoretical framework for the CTTR function, we analyzed and quantified how these factors affect travel time reliability on the basis of empirical data.

One group of factors affecting travel speeds and hence travel times are geometrical characteristics of freeways, such as the number of ramps (on-ramp or off-ramp) and weaving sections per unit road length and their physical characteristics. The effects of road geometry on the probability of breakdown, travel time uncertainty and travel time unreliability have been investigated in this dissertation. It is found that there exist threshold values L (i.e. 300 meters for on-ramp sections and 250 meters for off-ramp sections) for the length of ramps sections. Below the length L , travel time unreliability increases with the decreasing length or ramps sections. Above the length L , the length has far less impact on travel time unreliability. In a freeway corridor, travel time reliability is strongly affected by the average distance between ramps. Below a certain threshold value (i.e. 3 kilometers), the shorter distance between ramps, the less reliable travel times; above this value, the distance between ramps has far less impact on travel time unreliability.

Adverse weather conditions may (locally) reduce the capacity in a traffic network, but may at the same time yield (global) changes in traffic demand, due to people changing routes, departure time, mode or even reconsidering taking a trip together. It is therefore obvious that travel time reliability is strongly affected by adverse weather. The empirical analysis of the impacts of adverse weather conditions on travel time unreliability shows that adverse weather conditions have negative effects on travel time reliability of freeway corridors.

On some motorway sections Speed Limits (SL) are imposed for safety and environmental reasons. Tight SL (from a high value of the maximum speed to a low value) reduces mean (local) speed, reduces capacity, increases traffic safety, and reduces the fuel consumptions

and emissions. Obviously, SL has an impact on travel times, for instance, tight SL results in larger TT10th and TT50th in comparison with relaxing SL (from a low value of the maximum speed to a high value). Tight SL reduces the travel time uncertainty (variability), but increases the probability of traffic breakdown. Therefore, the effect of tight SL on travel time unreliability depends on the value of the maximum speed.

It is clear that accidents on freeways interrupt traffic flows in an unexpected way, and are a major cause of extra congestion. They can reduce the capacity of a road and act as bottleneck to the traffic flow. The unpredictability of traffic accidents and its increasing effects on freeway traffic congestion significantly undermine the mobility in road networks. The empirical analysis in this dissertation indicates that traffic accidents have negative impacts on travel time reliability of freeway corridors. Traffic accidents increase both the travel time uncertainty and the probability of traffic breakdown and hence the increasing travel time unreliability.

Application of the CTTR model on these large sets of empirical data under different circumstances consistently reveals that the CTTR model as a function of the inflow under all circumstances follows a monotonically increasing convex curve, much like the well-known BPR (Bureau of Public Roads) travel time functions which are widely used in DTA models. Therefore, we developed a BPR-like analytical formula for travel time unreliability, the so-called *TLZ* (Tu, van Lint, van Zuylen) reliability function, which approximates the CTTR function accurately. This *TLZ* function contains just three parameters, which are a function of road geometry, weather, and the other above mentioned factors affecting travel time unreliability. These parameters can be easily calibrated on the basis of empirical data. One of the parameters of the *TLZ* function is the critical travel time reliability inflow λ_{tir} . Below λ_{tir} travel time unreliability is low; but above λ_{tir} , travel time unreliability sharply increases with rising inflows.

Assessment of the role of travel time unreliability both in route choices and in departure time choices and its impact on a network level is done by assigning the traffic to a network, taking into account that travelers attach a certain value to travel time (un)reliability, next to travel time itself. This reliability cost component in the route and departure time choice models can be easily based on the *TLZ* reliability function developed in this thesis. This *TLZ* reliability function is regarded as a function of inflow with a parameter λ_{tir} . The inflow can be directly obtained from outputs of the traffic assignment models and the λ_{tir} is regarded as a function of factors like road geometry, traffic control measures etc, all of which are also available in a DTA context. Since travel time reliability plays an important role in travel choice behaviors, the reliability-based DTA model potentially has a higher face-validity than DTA without incorporating travel time reliability. In a preliminary study, we tested the *TLZ* reliability model in a DTA model with a simple dual route network, where we set the λ_{tir} parameter to values based on the empirical results obtained in this thesis. The DTA experiments yield plausible and face-valid results, which confirm that the *TLZ* model can be easily integrated in the assignments.

To conclude, the travel time reliability model developed in this dissertation yields a tractable approach that captures the main features of travel times and travel time (un)reliability. It provides a new, useful framework for describing and understanding travel time unreliability and to find ways to improve reliability and analyze the impact of unreliability. Such a reliability model can be easily embedded in a DTA model to estimate or predict traffic conditions. In that sense, the findings and results from this dissertation thesis pave the

way for quantitative and both theoretically and empirically underpinned research into the role of travel time reliability in route and departure time choice.

Samenvatting

De betrouwbaarheid van reistijd heeft een significant effect op de routekeuze en de vertrektijdkeuze van een individu en wordt vooral als een belangrijke invloedsfactor beschouwd bij bepaalde ritten, zoals die met woon-werk motief, waar het individu beperkingen heeft in de tijd, bijvoorbeeld in zijn of haar aankomsttijd. Recente empirische studies ondersteunen deze bevindingen en suggereren dat reizigers niet alleen geïnteresseerd zijn in een reductie van hun reistijd maar ook in een reductie van de reistijdontbetrouwbaarheid. Het begrip reistijdbetrouwbaarheid is ook in de politiek doorgedrongen en wordt gebruikt als prestatie-indicator van mobiliteit. In de Nota Mobiliteit bijvoorbeeld speelt reistijdbetrouwbaarheid een centrale rol. Door het Ministerie van Verkeer en Waterstaat wordt het verbeteren van de reistijdbetrouwbaarheid op het volledige wegennet (“van deur tot deur”) gezien als een van de hoofddoelstellingen voor het komende decennium. Voor het voorspellen van de toekomstige verkeerssituatie op het netwerk vertrouwen beleidsmakers en planners op instrumenten zoals Dynamische Verkeers-toedelingenmodellen (in het Engels Dynamic Traffic Assignment, kortweg DTA). Echter, tot op heden zijn er geen geschikte reistijdbetrouwbaarheidsmodellen beschikbaar die gebruikt kunnen worden in DTA-modellen. In dit proefschrift is een dergelijk model (en de onderliggende theorie) ontwikkeld op basis van empirische gegevens. Dit nieuwe model kwantificeert de effecten van verkeersbeheersing en het ontwerp van infrastructuur op reistijdbetrouwbaarheid in de context van DTA-modellen en kan op deze manier helpen in het maken van beslissingen over maatregelen voor het verbeteren van de reistijdbetrouwbaarheid.

De karakteristieken van empirische (dat wil zeggen, gebaseerd op echte data) relaties tussen reistijd en intensiteit en tussen snelheid en intensiteit tonen aan dat er geen een-op-eenrelatie bestaat tussen reistijd en intensiteit of snelheid en intensiteit, zowel niet in vrije omstandigheden als niet in congestie. De metingen van beide relaties vertonen een grote spreiding. De gegevens laten zien dat reistijd als functie van intensiteit drie gebieden bevat: (1) onder de kritische transitie-instroom, waar reistijden zeker en stabiel zijn; (2) tussen kritische transitie-instroom en kritische capaciteits-instroom, waar reistijden zowel onzeker als instabiel zijn; en (3) boven de kritische capaciteits-instroom, waar reistijden zeker maar instabiel zijn. Gebaseerd op deze observaties noemen we in dit proefschrift een reistijd onbetrouwbaar als (a) de reistijd onzeker is; (b) de reistijd zeker maar instabiel is (instabiliteit betekent dat de reistijd in de nabije toekomst sterk toe kan nemen als gevolg van het ontstaan van congestie, oftewel een zogenoemde ‘breakdown’ van het verkeer); en/of (c) als de consequenties van instabiliteit (de breakdown) groot zijn. Daarom bevat reistijdbetrouwbaarheid in onze theorie twee elementen: onzekerheid (variabiliteit) van reistijd en instabiliteit van reistijd. Het laatste is nauw verwant aan de kans op het ontstaan van congestie (de kans op breakdown), in de zin van een transitie van vrije doorstroming naar gesynchroniseerde stroming (‘synchronized flow’).

Het onderzoeken van reistijdonderbreikbaarheid is het proces van het kwantificeren van deze twee elementen. Eerst wordt de kans dat een reiziger een breakdown mee zal maken beschouwd als een risico. Vervolgens wordt dat risico beschreven door middel van twee grootheden: de kans op het optreden van een breakdown en de bijbehorende consequentie (omvang). In deze dissertatie wordt een Conceptueel Reistijdbetrouwbaarheidsmodel, of Conceptual Travel Time Reliability (CTTR) model geïntroduceerd, welke wordt berekend als de som over de producten van de gevolgen (variabiliteit of onzekerheid) maal de bijbehorende kansen op een breakdown van het verkeer. In dit model wordt de kans op een breakdown op een wegvak gecategoriseerd in een spontane breakdown en een geïnduceerde breakdown, welke onafhankelijk verondersteld worden. De kans op een breakdown op een route wordt geformuleerd als het één minus het product van de kansen op geen breakdown op alle wegvakken op de route.

In condities van lage intensiteiten is de reistijd zeker terwijl onzekerheid optreedt in condities van hoge intensiteiten. Intensiteit is een van de belangrijkste invloedsfactoren (waarschijnlijk de belangrijkste) van reistijdbetrouwbaarheid. Daarom kan de instroom naar een wegvak worden gezien als de principiële parameter in het reistijdbetrouwbaarheidsmodel. Het CTTR-model wordt beschouwd als een functie van meerdere factoren. Deze factoren worden eerst als conditioneel beschouwd, wat betekent dat de functie de reistijdbetrouwbaarheid berekent voor een bepaald niveau van instroom, gegeven bepaalde omstandigheden. Deze omstandigheden bestaan uit karakteristieken van de weg en alle andere relevante factoren zoals verkeersbeheersingsmaatregelen, de huidige toestand op de weg (vrije doorstroom of congestie) en mogelijke externe factoren zoals weer, lichtinval, enzovoorts. Het instroom-CTTR-model wordt gevalideerd en gekalibreerd op basis van empirische gegevens die verzameld zijn door het Regiolab-Delft verkeersmonitoringsysteem. Het blijkt dat zowel de kans op een breakdown van het verkeer als de reistijdonderbreikbaarheid toenemen met een toename van de instroom (de intensiteit) naar een wegvak.

De variabiliteit van reistijd op autosnelwegen is een gevolg van variaties in zowel de vraag als het aanbod. Omdat het CTTR-model generiek is, kan het worden uitgebreid om ook een functie te zijn van de aanbodfactoren, zoals wegontwerp, slecht weer, snelheidslimieten, verkeersongevallen, enzovoorts. Nu het theoretische raamwerk voor de CTTR-functie is ontwikkeld, analyseren en kwantificeren we hoe deze factoren de reistijdbetrouwbaarheid beïnvloeden op basis van empirische gegevens.

Eén factor die de rijnsnelheid en daarom de reistijd beïnvloedt is het geometrische ontwerp van de autosnelweg, zoals het aantal op- en afritten en weefvakken per eenheid van lengte en hun fysieke karakteristieken. In deze dissertatie zijn de effecten van het wegontwerp op de kans op breakdown, de reistijdonzekerheid en de reistijdonderbreikbaarheid onderzocht. Het blijkt dat er een grenswaarde L bestaat voor de lengte van de op- of afritten (300 meter voor opritten en 250 meter voor afritten). Als de op- of afrit korter is dan deze lengte L neemt de reistijdbetrouwbaarheid af met een afname van de lengte van de op- of afrit. Is de op- of afrit langer dan deze lengte L , dan heeft de lengte van de op- of afrit veel minder invloed op de reistijdbetrouwbaarheid. De reistijdbetrouwbaarheid in een autosnelwegcorridor wordt ook sterk beïnvloed door de gemiddelde afstand tussen op- en afritten. Onder een bepaalde grenswaarde (3 kilometer) blijkt dat een kortere afstand tussen de op- en afritten leidt tot minder betrouwbare reistijden. Boven deze grenswaarde heeft de afstand tussen op- en afritten veel minder invloed op de reistijdonderbreikbaarheid.

betrouwbaarheid.

Slechte weersomstandigheden kunnen (lokaal) de capaciteit op een verkeersnetwerk doen afnemen, maar kunnen tegelijkertijd ook leiden tot een verandering van de (globale) verkeersvraag, doordat mensen veranderen van route, vertrektijd of vervoerswijze of zelfs heroverwegen of ze de rit wel gaan maken. Het is daarom duidelijk dat de reistijdbetrouwbaarheid sterk beïnvloed wordt door slechte weersomstandigheden. De empirische analyse van de invloed van slecht weer op de reistijdbetrouwbaarheid laat zien dat slechte weersomstandigheden een negatief effect op de reistijdbetrouwbaarheid op autosnelwegcorridors hebben.

Op sommige autosnelwegvakken zijn lagere snelheidslimieten ingevoerd uit veiligheids- of milieuoverwegingen. Lagere snelheidslimieten reduceren, vergeleken met hogere snelheidslimieten, de gemiddelde (lokale) snelheid, reduceren capaciteit, verbeteren de verkeersveiligheid en reduceren het brandstofverbruik en emissies. Het is duidelijk dat snelheidslimieten een invloed hebben op de reistijd. Zo resulteren lagere snelheidslimieten bijvoorbeeld in een grotere 10-procents-percentiel en 50-procents-percentiel van de reistijd in vergelijking met een hogere snelheidslimiet. Lagere snelheidslimieten verlagen de reistijdonzekerheid (variabiliteit), maar verhogen de kans op breakdown. Daarom hangt het effect van een lagere snelheidslimiet af van de waarde van de maximum toegestane snelheid.

Het is duidelijk dat incidenten op de autosnelweg de verkeersstroom onderbreken op een onverwachte manier. Incidenten zijn een belangrijke oorzaak van extra congestie. Ze kunnen de capaciteit van een weg reduceren en vormen een knelpunt voor de verkeersstroom. De onvoorspelbaarheid van incidenten en de toenemende effecten op de congestie op de autosnelwegen ondermijnen de doorstroming van het wegennet. De empirische analyse in dit proefschrift laat zien dat incidenten negatieve gevolgen hebben voor de reistijdbetrouwbaarheid op autosnelwegcorridors. Incidenten zorgen voor zowel een toename van de reistijdonzekerheid als een toename van de kans op breakdown van het verkeer en zorgen dus voor een toename in reistijdonbetrouwbaarheid.

Toepassingen van het CTTR-model op deze grote sets van empirische gegevens onder verschillende omstandigheden laten zien dat het CTTR-model onder alle omstandigheden een monotoon toenemend convex gedrag vertoont, vergelijkbaar met de welbekende BPR-reistijd functie (Bureau of Public Roads), die veel gebruikt worden in DTA-modellen. Daarom is een BPR-achtige analytische formule ontwikkeld voor reistijdonbetrouwbaarheid, de zogenoemde *TLZ* (Tu, van Lint, van Zuylen) betrouwbaarheidsfunctie, welke de CTTR-functie accuraat benadert. Deze *TLZ*-functie bevat slechts drie parameters die een functie zijn van het wegontwerp, de weersomstandigheden en de andere bovengenoemde factoren die de reistijdonbetrouwbaarheid beïnvloeden. Deze parameters kunnen gemakkelijk gekalibreerd worden op basis van empirische gegevens. Een van de parameters van de *TLZ*-functie is de kritische reistijdbetrouwbaarheidsinstroom λ_{itr} . Bij een instroomwaarde onder λ_{itr} is de reistijdonbetrouwbaarheid laag; maar boven λ_{itr} neem de reistijdonbetrouwbaarheid sterk toe met een toenemende instroom.

Een studie naar de rol van de reistijdonbetrouwbaarheid in zowel routekeuze als vertrektijdkeuze en de invloed op een netwerkbrede schaal is uitgevoerd door het verkeer aan een netwerk toe te delen, rekening houdend met het feit dat reizigers, naast de reistijd zelf, een bepaalde waarde hechten aan reistijd(on)betrouwbaarheid. De betekenis van betrouwbaarheids in de route- en vertrektijdkeuze kan gemakkelijk worden gemodelleerd

met de *TLZ*-betrouwbaarheidsfunctie die ontwikkeld is in dit proefschrift. Deze *TLZ*-functie wordt beschouwd als een functie van de instroom met een parameter λ_{itr} . De instroom kan direct worden verkregen uit de uitvoer van toedelingsmodellen en de λ_{itr} wordt beschouwd als een functie van factoren zoals het wegontwerp, verkeersbeheersingsmaatregelen, enzovoorts, welke allemaal beschikbaar zijn in een DTA-context. Omdat de reistijdbetrouwbaarheid een belangrijke rol speelt in de keuzes van de reiziger, kan een DTA-model dat rekening houdt met betrouwbaarheid in potentie een hogere logische validiteit hebben dan een DTA-model dat geen rekening houdt met de betrouwbaarheid. In een eerste studie hebben we het *TLZ*-betrouwbaarheidsmodel getest in een DTA-model met een simpel netwerk met twee routes, waar we de parameter λ_{itr} hebben geschat op basis van de empirische resultaten van dit proefschrift. De DTA-experimenten tonen plausibele resultaten, welke bevestigen dat het *TLZ*-model gemakkelijk geïntegreerd kan worden in de toedelingen.

Ten slotte, het reistijdbetrouwbaarheidsmodel dat ontwikkeld is in deze dissertatie is een nieuw, bruikbaar raamwerk voor het beschrijven en begrijpen van reistijd(on)betrouwbaarheid, voor het analyseren van de invloed van onbetrouwbaarheid en voor het vinden van manieren om de betrouwbaarheid te verbeteren. Het levert een praktische aanpak dat de belangrijkste eigenschappen van de reistijd en de reistijd(on)betrouwbaarheid omvat. Een dergelijk betrouwbaarheidsmodel kan gemakkelijk in een DTA-model worden toegepast om de verkeerstoeestand te schatten of te voorspellen. Op die manier plaveien de bevindingen en resultaten van dit proefschrift de weg voor kwantitatief en zowel theoretisch als empirisch onderbouwd onderzoek naar de invloed van reistijdbetrouwbaarheid op routekeuze en vertrektijdkeuze.

(Dutch translation by Chris van Hinsbergen)

Summary (Chinese)

在今天全球经济激烈竞争的时代，人们出行不仅仅单纯地考虑平均的行程时间，对于行程时间可靠度也越来越关心。向用户提供与行程时间可靠度相关的出行信息，能大大减少用户出行的盲目性。这不仅能让用户受益，而且还可能使交通状况更稳定快速。同时，可靠度作为路网性能的重要指标之一也进入了决策者的视野。比如，北京市政府在2005年提出了针对2008年北京夏季奥林匹克运动会的一项特殊交通目标：从运动员的宿地到运动场馆的平均行程时间不超过30分钟。也就是说北京市政府希望50%的此类出行能在半小时内到达。另外一个例子是，荷兰政府在2004年向荷兰议会提交了一份议案。此份议案在交通运输方面的政策目标是“面向可靠的、可预测的交通出行”。这个政策的重点在行程时间可靠度上：荷兰政府希望95%的长距离出行（大于50公里）能在1.2倍的中值行程时间内到达；对于短距离出行（小于50公里），95%的出行能在中值行程时间加上10分钟内到达。北京和荷兰提出的这两项针对可靠度的议案或政策都表明行程时间可靠度的研究已经引起了决策层、研究者、工程实践者、规划人员的重视。决策者和管理规划人员需要利用动态交通分配模型来预测诸如行程时间、行程时间可靠度等的交通状况。此外，也需要一个稳定可靠的模型来评估交通控制和路网设计对行程时间可靠度的影响。

目前已有的行程时间可靠度的定量化方法和措施主要侧重于研究行程时间的变异性或只关注行程时间的分布。首先，本文通过经验数据鉴别了行程时间所包含的属性。流量—行程时间以及流量—速度的经验特征表明流量和行程时间在自由流和拥挤流都没有固定的一对一的关系，反而是一种非常离散的关系。经验分析证明行程时间—流量方程有三个区域：(a)低于临界过渡流量，确定而稳定的行程时间，(b)在临界过渡流量和临界通行能力流量之间，不确定也不稳定的行程时间，(c)高于临界通行能力流量，确定但不稳定的行程时间。本文将以下情况定义为不可靠的行程时间：如果(a)行程时间不确定(要么在自由流要么在拥挤流)；(b)行程时间当前处于确定状态，但在下一阶段不稳定；或者(c)不稳定导致的交通流突变的后果严重。因此，行程时间可靠度至少需要包含两个重要元素：行程时间的不确定性（或变异性）和行程时间的不稳定性。后者主要由交通流突变（breakdown，也就是交通流从自由流状态到拥挤状态）来表征。行程时间可靠度的研究就是如何量化这两个主要元素。假设一个司机是否遭遇交通

突变具有某种风险，那么该风险可以表征为两个因子：发生突变的概率（或似然性）以及突变所造成的后果。本文提出的概念性行程时间可靠度模型（CTTR）是基于风险分析的，而可靠度就是交通流突变概率与突变所产生结果的乘积之和。在CTTR模型中，路段的突变概率分为自发突变（spontaneous breakdown）和诱导突变（induced breakdown）。这两种突变是相互独立的。由此，本文建立了数学关系来表征路径突变概率与路段突变概率的关系。

高速公路上没有流量就没有所谓的交通拥堵产生，也就没有行程时间的不可靠问题了。由此看出，流量是影响行程时间可靠度最重要的因素之一。因此，CTTR模型至少有流量这个参数，同时CTTR模型也应是一个多变量的方程。也就是说，在特定的环境下，行程时间可靠度是一个以流量为变量的方程。这个特定的环境是指一定的路网特性、交通控制手段、天候情况以及是否有交通事故等等。本文用从Regiolab-Delft交通数据监控采集系统中采集的数据验证和校核了CTTR模型。结果表明交通流突变概率和行程时间可靠度都是随着流量的增加而增加。经验结果同时显示了行程时间可靠度与流量之间服从一定的关系。由此，本文用TLZ（Tu, van Lint, van Zuylen）方程来拟合这些结果。TLZ方程拟合结果表明：低于特定的阈值（指临界行程时间可靠度流量），行程时间比较可靠；高于这个阈值，行程时间可靠度随着流量的增加而降低。

本文将CTTR模型拓展为一个诸如道路几何特征、恶劣气候、交通控制措施、交通事故等多变量的方程。由于交通需求和交通供给的变异性造成了高速公路行程时间的变异。其中一组影响车速及行程时间的因素是高速公路的几何设计，如进出口匝道间距、匝道长度等。本文调查分析了道路几何设计对突变概率、行程时间不确定性以及行程时间可靠度的影响。结果表明，高速公路的匝道长度不能小于一定的阈值（如进口匝道不能小于300米；出口匝道不能小于250米）：小于这个阈值，行程时间的可靠度由于匝道长度的减小而大大降低；大于这个阈值，匝道长度对可靠度的影响则大大减小。对于一个高速公路长廊来说，进出口匝道的间距越短，行程时间越不可靠，尤其是在间距小于3公里的情况下。

恶劣的天气降低了路网的通行能力，同时因为恶劣天气改变了人们的出行路线、出发时间以及出行方式而影响了交通需求。因此可以预期恶劣天气对行程时间可靠度的影响很大。经验数据分析也证实了恶劣天气对于可靠度产生了负面的影响。例如，当流量接近2000辆每小时每车道时，雨天情况下的行程时间变异性是晴天的2倍左右。

设置通常限速标志主要是从安全和环境的角度来考虑的。已有的研究表明紧缩类限速标志（速度从高速设定为相对低速）降低了平均车速和通

行能力，但增加了交通安全和减少了废气排污。本文的经验数据分析显示紧缩类限速标志增加了出行的十分位和中位行程时间；紧缩类限速标志减少了行程时间的不确定性、但增加了交通流突变的概率。因此，紧缩类限速标志对行程时间可靠度的影响取决于限速值。

通常认为，交通事故是造成交通拥挤的重要因素之一。交通事故干扰正常交通流具有突然性，也具有不可预见性。交通事故降低了路网通行能力并造成了路网交通瓶颈。因此，交通事故是造成行程时间不可靠的主要原因之一。经验数据分析表明交通事故增加了行程时间的不确定性、交通流突变的概率、行程时间的不可靠性。因此交通事故对行程时间可靠度产生了负面影响：交通事故导致了行程时间不可靠度增加了8%左右。

为了分析行程时间可靠度对于路线选择和出发时间的影响，在动态交通分配时，可以在效用方程或费用方程中加入可靠度元素。本文提出的 *TLZ* 模型是一个以流量为变量、临界行程时间可靠度流量等为参数的方程。流量可以在动态交通分配的流量输出得到，而其他参数可以经过经验数据拟合而得。因此，*TLZ* 方程可以直接用于动态交通分配模型中。本文将 *TLZ* 方程应用于一个基于可靠度模型的动态交通分配模型。这个简单的应用实例表明 *TLZ* 方程可以较为容易地应用到动态交通分配模型中，以此同时行程时间可靠度对路线选择和出发时间都产生了影响。

综上所述，本文提出的行程时间可靠度模型提供了一种新的、有效的框架来描述和理解行程时间可靠度，并用一种简便的方法来量化行程时间及其可靠度的特征，因此也提供了一种新的途径来分析可靠度的影响。同时，这个可靠度模型可以内嵌到动态交通分配模型中来更准确、更稳定地估算或预测交通状况。

(Summary (Chinese version) has been approved by Prof. dr. Lijun Sun: 孙立军教授)

About the Author

Huizhao Tu (涂辉招) was born in Changting, Fujian Province, China, on April 3 1977. From 1996 to 2003, he studied at Tongji University where he received his BSc degree (with honor) in Civil Engineering in 2000 and his MSc degree in Transportation Engineering in 2003 under the supervision of Professor Lijun Sun. During his studies in Tongji he was involved in several



projects, among which, *Feasibility Study on Implementing Electronic Road Pricing in Shanghai* (Shanghai Sci-Tech Progress Award II, 2004), *Shanghai Infrastructure Management Systems* (Shanghai Sci-Tech Progress Award III, 2003), *Ten-Year Transport Strategy Development on Urban Road in Shanghai* (Shanghai Sci-Tech Progress Award III, 2005), and *Traffic Evaluation of Shanghai Urban Freeways, Traffic Data Collection and Processing for Shanghai*. His master thesis concerned the *Urban Roads Evaluation Model and its Implementation on Electronic Road Pricing in Shanghai*, defended successfully in 2003. In 2002 he attended an ITS training course in Shanghai, where he met his current supervisor, Professor Dr. Henk van Zuylen.

In November 2003, Huizhao Tu moved to The Netherlands to start his PhD at the Department of Transport and Planning, Delft University of Technology and TRAIL Research School. His PhD research is part of the ATMO project, short for Advanced Traffic MONitoring. ATMO is one of the projects in the cluster Traffic Management of the TRANSUMO program (TRANSition Sustainable MOBility). The central objective of ATMO is the development of knowledge and skills in the structural setup, realization, maintenance and operation of a robust and reliable monitoring system for heterogeneous traffic networks that generates (predicted) traffic information for both traffic managers and traffic participants. This research, conducted under the supervision of Professor Dr. Henk van Zuylen and Dr. Ir. Hans van Lint, resulted in a number of publications in proceedings of several international conferences, for instance TRR, TRB, IEEE ITSC, IFAC, and INSTR. His PhD dissertation addresses *Monitoring Travel Time Reliability on Freeways*. After he finished his PhD in April 2008, he started working for Grontmij, an international consultancy company in The Netherlands.

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