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Speed limits and their effect on freeway capacity

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Colophon

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Preface

"I learned that courage was not the absence of fear, but the triumph over it. The brave man is not he who does not feel afraid, but he who conquers that fear."

—Nelson Mandela

This thesis is the graduation work that concludes my Master of Science programme in the field of Traffic Engineering.

W.H. van Lindonk.

Delft, September 2020

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So far, writing this master thesis is the largest project I have undertaken in my life and I could not have done it without the help of a great group of people around me.

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Executive Summary

Introduction

In this thesis an investigation is performed into the effects of different speed limits on capacity. Much is known about factors that change the capacity of a roadway, but only very limited knowledge is available about the actual effects of a speed limit change on the capacity distribution. With its rich history of freeway speed limit changes (see chapter 1), The Netherlands has become a great test case for the evaluation of the effect of different freeway speed limits on capacity. Moreover, because capacity is such an important factor in the determination of travel times in the network during times of high demand, it is worth investigating if some limits make it more likely than others that traffic congestion will form at a particular location.

Locations

Several locations have been selected to be studied in this paper and two samples of locations have been generated. The first sample consists of a total of eight locations which have experienced a speed limit change from 120 to 130 km/h. The second sample consists of a total of seventeen locations, for which eleven locations have experienced a speed limit change from 120 to 130 km/h and subsequently to 100 km/h and for which the remaining six locations have experienced a speed limit change from 120 to 100 km/h. In these samples only two-lane bottlenecks are included, because these types of bottlenecks are most sensitive to congestion and because there is a clear distinction between the passing lane (left) and the shoulder lane (right) which provides a simple framework for the analysis of lane choice behavior.

Methodology

First, the "eight-location" sample has been analyzed in chapter 4, to investigate whether the capacity distributions of the complete roadway, the passing lane and the shoulder lane have significantly changed as a result of a change in the speed limit. In addition to this, levels of truck traffic have been gathered for these locations and a t-test for the comparison of means was performed, to test whether significant changes in truck traffic levels have occurred from one period to the next. Additionally, the effect of the speed limit on the lane flow distribution was estimated by means of a Z test for the comparison of proportions, to check whether the speed limit had led to significant changes in the lane flow distribution. After this, in chapter 5, the breakdown flow samples from both the "eight-location" sample and the "seventeen location" sample have been analyzed by means of Fixed Effects regression, to assess whether the height of the speed limit had a significant effect on the mean breakdown flow and to assess whether a significant relation was present between the speed limit and the fraction of flow in the passing lane.

Capacity Distribution Results

For the comparison of capacity distributions of the "eight-location" sample it was found that, for the complete roadway, 4 out of 8 locations showed a significant increase in capacity, while 3 out of the remaining 4 locations showed a significant decrease in capacity and 1 location was indeterminate. For the passing lane, it was found that breakdown flows had significantly increased for 5 out of 8 locations and had significantly decreased for 2 out of 8 locations, indicating increased passing lane use. For the shoulder lane, it was found that breakdown flows had significantly increased at 4 out of 8 locations and significantly decreased at the remaining 4. On the whole, despite recording significant changes for all locations but one, no uniform trend was visible in the data for the effect of the speed limit on capacity. It was, however, clear that the fraction of flow in the passing lane had increased under the 130 km/h limit.

Subsequently, truck traffic data was obtained for 4 out of 8 locations and a test was performed to check whether the level of truck traffic had stayed the same between measurement periods. It was found for two out of four locations that truck traffic had significantly increased, while for the remaining two locations no significant change in truck traffic was detectable. Strangely, for the two locations where truck traffic was found to have increased, capacity had stayed the same or even increased, while for the other two locations where truck traffic levels had stayed the same, capacity had significantly decreased.

To investigate whether the change in the speed limit had led to a significant change in passing lane use, a Z test for the comparison of proportions was performed on before and after samples of lane flow fraction data. It was found for all locations but one that the fraction of flow under the 130 km/h limit had increased.

Results from Fixed Effects Regression

Because of the fact that no clear direction could be found for the effect of the speed limit on capacity. Several fixed effects regressions have been performed on breakdown flow measurements from both the "eight-location" sample and the "seventeen-location" sample to assess whether a significant effect from the speed limit could be discovered when location specific effects were accounted for.

From the regressions on the "eight-location" sample it was found that the mean breakdown flow under the 120 km/h limit was higher (in the range of 60 to 110 vehicles per hour) than under the 130 km/h and that this effect was significant at at least the 5% level in 11 out of 12 regressions. In the "seventeen-location" sample, a similar regression was performed and also here the mean breakdown flow was found to be significantly higher under the 120 km/h limit (in the range of 80 to 190 vehicles per hour) than under the 130 km/h limit. For the 100 km/h limit, the results were less clear, but it is suggested that, when all other relevant effects are accounted for, that the mean breakdown flow is slightly lower than the mean breakdown flow under 120 km/h limit, but higher than the mean breakdown flow under 130 km/h limit. It should be noted, however, that only a relatively small proportion of measurements in the sample was derived under the 100 km/h limit and that this data may be less representative of "regular" conditions, as the data was obtained during the COVID-19 lockdown period.

High levels of significance were also recorded for the Lane Flow Fraction variable, which represented the fraction of flow in the passing lane and which was found to be strongly related to both the breakdown flow as well as the speed limit. For the relation between the flow fraction in the passing lane and the breakdown flow a quadratic relation was found to best represent the trend in the data. Additionally, a separate regression was run in which the speed limit was used as an explanatory variable of the fraction of flow in the passing lane and a significant positive relation was discovered between the speed limit and the passing lane utilization rate for both samples. As such, there is evidence to suggest that the speed limit has an effect on the lane flow distribution which, in its turn, affects the capacity of the roadway.

Conclusion

Despite several efforts at isolating the effects of a speed limit change on the capacity, it has proven to be difficult to account for all the other affect that affect capacity. As such, it is uncertain whether significant changes to the capacity distribution have occurred as a consequence of the change in the speed limit. On the basis of lane flow distribution data, it is relatively certain that a relation exists between the height of the speed limit and the utilization rate of the passing lane, which does indirectly affect the capacity of the roadway. Moreover, at least for the 120 km/h limit, clear and consistent evidence has been found that breakdown flows under this limit are higher than under the 130 km/h. As such, it can be expected that it may very well be the case that the speed limit does significantly affect capacity and that speed limits above 120 km/h lead to sub-optimal values of capacity.

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1. Introduction

1.1. Freeway Speed Limits in The Netherlands

The history of freeways in the Netherlands starts with the construction of the first freeway, the A12 between Voorburg and Zoetermeer, which was the first freeway in the world to include an emergency lane. For most of the following decades, no general freeway speed limit was present, which was not necessary either, because the principle constraint on the driving speed was the vehicle itself. As vehicles improved and were able to attain greater speeds, speeding became a problem and, for this reason, it was decided in February 1974 to regulate freeway speeds by imposing a legal limit of 100 km/h (Blekendaal, 2004). The decision to impose a speed limit was officially taken for the improvement of traffic safety, but the oil embargo during the oil crisis is also said to have played a major role (Blekendaal, 2004). Fourteen years later, on May 1988, the general limit was increased to 120 km/h, though for many freeway sections, especially near urbanised regions, the limit remained 100 km/h.

Figure 1.1. – The A12 between Voorburg and Zoetermeer in 1937 (Klassiekerweb, 2014)

In November 2005, the speed limit on a number of freeway arterials through densely populated regions was reduced from 100 km/h to 80 km/h (Hoogendoorn *et al.*, 2013) to reduce pollution in the area surrounding those freeway facilities. It was found that this reduction was not always effective due to more congestion formation, which was caused by a larger percentage of traffic driving in the right-most lanes, leading to fewer merging opportunities in merging areas (Tool *et al.*, 2006). Because of this, the 80 km/h limit has been changed back to 100 km/h for some of these sections in recent years.

More recently, in September 2012, with the aim of attaining travel time gains, the general limit on Dutch Freeways was officially increased from 120 km/h to 130 km/h and this increase was gradually rolled out over the network between 2012 and 2019. However, seven years later, following the nitrogen-dioxide crisis in 2019, this 130 km/h general limit increase has been largely revoked, as a general freeway speed reduction to 100 km/h has been imposed during the day (from 06:00 until 19:00), since 15 March 2020.

Since this date, the general speed limit on freeways is 100 km/h for all locations during the day, except for 80 km/h locations, while, during the night, speed limits are either 80, 100, 120 or 130 km/h, depending on what they were before the imposition of the daytime 100 km/h limit (Rijkswaterstaat, n.d.).

Because The Netherlands has such an extensive history of speed limit changes on freeways and because a large amount of different limits are currently present, the effects of speed limits on relevant policy outcomes (as defined in the previous section) are frequently under discussion. A wide variety of opinions exists among experts, as well as the general public, regarding the effects of a change in freeway speed limits on travel time gains and capacity. As of yet, no clear answer has been proposed for how capacity is affected by changes to static speed limits in the Netherlands. It is the aim of this thesis to investigate what effect on capacity can be experienced as a result of a change in the legal limit.

1.2. Findings on Speed Limits and Capacity

Generally, it is well known that higher speed limits are likely to cause lower levels of safety (Van der Pas, 2011) (SWOV, 2020) and more environmental damage (Lange *et al.*, 2011) (EEA, 2019), with the potential benefit of travel time gains under uncongested conditions.

In traffic flow theory, (mean) driving speed is one of the three fundamental aspects of the fundamental equation of traffic ($Q = K * U$) which states that the flow Q in vehicles per hour must be equal to the density of traffic K, expressed in the number of vehicles per kilometer, times the mean speed of those vehicles U. Because speed limits provide a restriction on such an important variable as speed choice, it may be the case that a change in the speed limit causes changes in the mean speed which will most certainly affect the flow and, potentially, the capacity flow.

Research has been performed on the application of variable speed limits for increasing flow stability through homogenization of traffic (Smulders, 1990) as well as for resolving jam waves (Hegyi and Hoogendoorn, 2010), but no clear empirical findings have been found with respect to static speed limits. In a paper by (Geistefeldt, 2011) it was found, for a number of locations in Germany, that on sections where a speed limit of 100 km/h or 120 km/h applied, median capacity values were slightly higher than for sections where no speed limit was present. Furthermore, it was found that the capacity distribution exhibited a lower level of variance under a speed limit, than in absence of any speed limit (Geistefeldt, 2011) (see Figure 1.2).

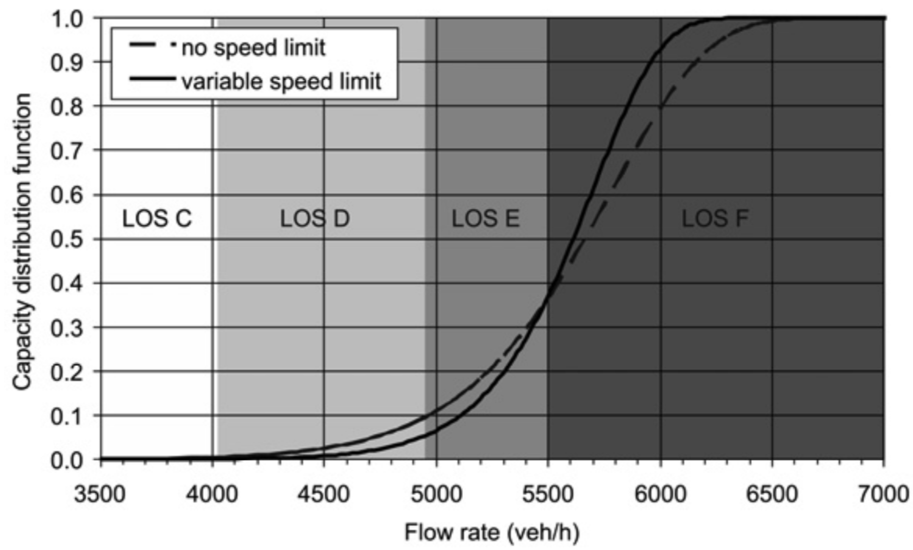

Figure 1.2. – Capacity distribution for no speed limit and a variable limit of 120 km/h (Geistefeldt, 2011, p.55)

Stability of Traffic

It is well known that the influence of slow vehicles as "moving bottlenecks" in the stream will increase when density K increases, because the number of passing opportunities will decrease (Knoop *et al.*, 2018). The increased incidence and influence of moving bottlenecks will lead to increased platoon formation and, on a two lane freeway, these platoons will generally be characterized by a slow vehicle (such as a truck) on the shoulder lane and a dense group of vehicles behind this slower vehicle, eagerly "waiting" in a fast-moving queue to overtake the slow vehicle via the passing lane.

It is known that platoons are generally unstable (Knoop *et al.*, 2018) and, for this reason, it can be expected that when a disturbance happens somewhere in the "fast-moving queue", it will generally be amplified throughout the platoon. As density K increases further, both the number of platoons as well as their lengths will increase, until the whole traffic stream becomes a platoon, making it more likely that a critical disturbance will be passed on among an increasingly large chain of vehicles. When the chain is sufficiently long and the disturbance sufficiently strong, the speed will drop rapidly and traffic breakdown will have occurred.

Ambiguous Travel Time Gains

Though speed limits determine the amount of potential travel time gains when traffic volumes are low, traffic stability will be the primary factor affecting travel times when volumes are high. Traffic stability is of paramount importance as the degree to which traffic is stable will determine whether the stream of vehicles will remain flowing at high volumes with relatively high speed or will break down, causing a reduction in both speed and flow and leading to travel time losses.

The Netherlands is a small and densely populated country where the freeway network is frequently saturated. As such, it could be argued that it is more important to set speed limits that will enhance traffic stability and maximum throughput during times of high demand than setting limits which only minimize travel time during times of low demand. It is not fully clear to what extent the height of a static speed limit can contribute to increased throughput and flow stability and it is therefore that the following research question is posed:

1.3. Research Question

To what extent does the height of the speed limit affect freeway capacity?

1.4. Scope of The Thesis

The focus in this thesis will be principally on two lane freeway bottleneck locations and there are three main reasons for this: Firstly, the largest share of multi-lane freeways in The Netherlands, as well as in the world, are two lane freeways. Secondly, bottlenecks on two lane freeways are more susceptible to congestion than bottlenecks on freeways with three or more lanes. Lastly, the strict distinction between a passing lane (left lane) and shoulder lane (right lane), which have very distinct speed and flow characteristics, is sharp and unambiguous and provides a clear framework within which lane choice behavior can be analyzed.

In this study, loop detector data from periods surrounding a speed limit change will be used to determine the capacity of static freeway bottlenecks, which will be used to assess whether significant changes in capacity have occurred. Sufficient traffic breakdown data must be available for a location to enable capacity estimation, which entails that only a limited selection of freeway bottlenecks is suitable for the assessment of capacity. Furthermore, earliest data for most locations is available from 2010, meaning that primarily the change from the 120 to 130 km/h limit can be assessed as well as the change from the 120 km/h and 130 km/h limits to the 100 km/h limit respectively.

1.5. Goal of this Thesis

When capacity is reached on a freeway, traffic breakdown will occur. Traffic breakdown is generally characterized by frequent speed oscillations in the stream, which will cause unnecessary braking and acceleration manoeuvres. Because of these manoeuvres there will be a larger risk of traffic accidents, more energy use and noise generation and the travel time will increase, which are all undesired consequences. As such, if the height of a speed limit has a significant effect on capacity, it is important to know for policy makers what this effect is, so that they can take it into account when imposing a certain speed limit at a freeway location. Consequently, it is the goal of this thesis to provide an answer to what extent the speed limit should be viewed as a relevant policy variable concerning freeway capacity issues.

1.6. Relevance

Rijkswaterstaat, being an executive agency for the ministry of Infrastructure and Water Management, is responsible for the management of roadway and waterway infrastructures in The Netherlands. As such, Rijkswaterstaat is also responsible for the management of the Dutch freeway network and implementation of speed limit policy - though the responsibility for enforcing these limits falls to the national police. Freeway capacity research is valuable to Rijkswaterstaat because it enables the organisation to optimize conditions on the network to carry maximum capacity. In light of recent changes in general speed limits on freeways, it is the aim of this thesis to investigate the effect of speed limits on capacity and to determine whether it should be viewed as a relevant policy variable for maximizing capacity. With the information acquired in this thesis it is hoped that decision makers at Rijkswaterstaat will gain a better understanding of the implications of different speed limits, so that location specific limits can be more effectively tuned to the situation at hand.

1.7. Structure of the Thesis

In the remainder of this thesis, an investigation will be performed on the capacity effects of different speed limits. In order to provide a theoretical framework, a definition will be given of capacity, as well as relevant theories concerning capacity in chapter 2. Next, in chapter 3, the research question will be revisited and five sub-questions with corresponding hypotheses will be proposed, which will be tested in chapter 4 and chapter 5. Additionally, an overview will be given of relevant methods as well as some characteristics of the data that has been used.

Subsequently, in chapter 4, the first three sub-questions will be evaluated. In this chapter, an evaluation will be made of the capacity effects of a limit change from 120 to 130 km/h on the capacity distributions of eight different locations, for the complete roadway, the passing lane and the shoulder lane. After this, effects from truck traffic will be investigated (section 4.4) as well as changes to the lane flow distribution (section 4.5).

For the evaluation of hypotheses four and five, regressions have been performed on breakdown flow data in chapter 5 to investigate whether significant changes in the breakdown flow distributions have occurred as a result of the change in the speed limit, when location specific factors are controlled for. Additionally, a relation between speed limits and the lane flow distribution is investigated to find out if a general trend can be discovered.

In chapter 6 the major findings of this study are presented and are compared to findings in literature. Additionally, limitations of this study with respect to methods and data are discussed. In chapter 7 the sub-questions are revisited and results of this thesis are used to support the answers to each of these sub-questions. Subsequently, at the end of chapter 7, the answers to the sub-questions are used to answer the research question of this thesis. Lastly, recommendations for future research, as well as for practitioners, are made in chapter 8.

2. Theory

In this chapter, relevant theories and literature are discussed to lay the foundation upon which the rest of the analyses in this thesis are built. In the beginning of this chapter elementary traffic flow theory is discussed and a definition is proposed for the concept of capacity. It is posed that capacity is a value of flow encountered at a given percentile of a distribution of breakdown flows and is, as such, a stochastic phenomenon. Next, factors affecting this breakdown flow distribution in the context of a single lane will be discussed to illustrate how dynamics in an isolated lane affect capacity. Subsequently, findings in literature regarding lane choice behavior are discussed and it is proposed that the behavioral "slugs and rabbits" framework, as proposed by Daganzo (2002a), provides a good frame of reference for interpreting the results in this paper from a behavioral point of view. In summary, it is discussed that, on the one hand, higher speed limits may lead to increased capacity because of a reduced influence of vehicle lengths on the gross time headway, while on the other hand, they may lead to reduced capacity, as a result of (inefficient) changes in lane choice behavior, which are affected by changes in preference speeds and mean driving speeds in the passing lane and shoulder lane respectively.

2.1. Definitions of Capacity

Roughly speaking, traffic can either be in a Free-flow state (F), where the average speed is above the critical speed threshold U^*, or in a Congested state (C), where the average speed is below the critical speed threshold U^*.

The identification of these states can be performed by means of plotting the data (see Figure 2.1) with respect to the three fundamental variables of traffic, which determine the shape of the fundamental diagram:

- Q which is the flow in vehicles per hour
- K which is the density in vehicles per kilometer
- U which is the average speed in vehicles per hour

By looking for the value of speed at which the fundamental diagram reaches maximum flow, the critical speed (U^*) can be found (Knoop *et al.*, 2018) which separates the free flow state F from congested state C.

Two transitions can occur between these states of traffic, which are both representative of a type of capacity. First, there is the $F \rightarrow C$ transition, for which the flow that occurs at this transition ($Q^{F \rightarrow C}$) is also called the "breakdown flow". Second, there is the $C \rightarrow F$ transition for which the flow that occurs at this transition ($Q^{C \rightarrow F}$) is called the "restoration

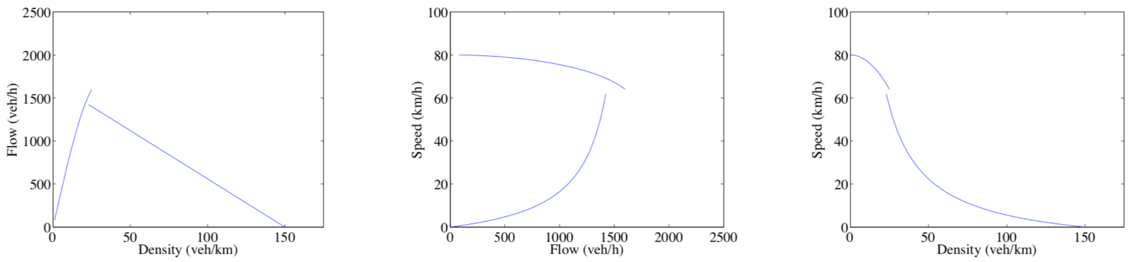

Figure 2.1. – A theoretical depiction of the fundamental diagram in three representations (Knoop et al., 2018, p.27).

flow". Both of these transitions can occur at different levels of flow and are, therefore, stochastic variables that follow a distribution (see Equation 2.1 and Equation 2.2):

$$Q^{F \to C} \sim (\mu_{Q^{F \to C}}; \sigma_{Q^{F \to C}}) \tag{2.1}$$

$$Q^{C \to F} \sim (\mu_{Q^{C \to F}}; \sigma_{Q^{C \to F}}) \tag{2.2}$$

Generally, the median (50th percentile) of the breakdown flow distribution ($Q_{50}^{F \to C}$) is considered to be the "free-flow" capacity, while the median of the restoration flow distribution ($Q_{50}^{C \to F}$) is considered to be the "queue-discharge" discharge capacity.

In most cases, due to differences in driving behaviors under the two transitions (Laval, 2011), the following condition holds:

$$Q_{50}^{F \to C} > Q_{50}^{C \to F} \tag{2.3}$$

This condition is also known as the capacity drop (Knoop *et al.*, 2018). In this thesis, the flow at the transition from state F to C ($Q^{F \to C} = Q^*$) is considered to be most relevant for investigating whether traffic stability is affected by a change in the speed limit. As such, estimations of Equation 2.1 will be performed for several locations by means of the Product Limit Method, as explained in chapter 3.

Consequently, although there are a number of different ways for defining the term "capacity" (Minderhoud *et al.*, 1996), capacity will be defined in this thesis as an **X-th percentile capacity** which represents:

The flow at the X-th percentile of the breakdown flow distribution ($Q_X^{F \to C}$) for the complete roadway.

All X-th percentile capacities will be based on **5-minute moving-averages**, updated at 1-minute intervals thus representing (realizations of) the **5-minute free-flow capacity**. This X-th percentile capacity is a result of the X-th percentile flows that occur at the moment of breakdown in the passing lane ($Q_{X,p}^{F \to C}$) and the shoulder lane ($Q_{X,s}^{F \to C}$), which follow their own respective distributions.

Because the breakdown flows of these lanes are such important constituents of the free-flow capacity, their distributions will also be estimated and analyzed in chapter 4. In the remainder of this chapter, several macroscopic and microscopic aspects that affect the breakdown flow distribution ($Q^{F \to C}$) will be discussed, to lay a foundation for the hypotheses posed in chapter 3 and the results of chapter 4 and chapter 5.

2.2. Determinants of the shape of the breakdown flow distribution of a single lane

Two parameters of the breakdown flow distribution are principally relevant here: The mean $\mu_{Q^{C \to F}}$, which is a measure of central tendency of the distribution, and the variance $\sigma_{Q^{C \to F}}$, which is a measure of the dispersion of the distribution. The value of $\mu_{Q^{C \to F}}$ is a direct result of the average flow that can be maintained for some time before breakdown occurs and is therefore indicative of the performance that can on average be expected from a freeway facility. The value of $\sigma_{Q^{C \to F}}$, on the other hand, is indicative of the wideness of the range of values for which breakdown does occur, and is indicative of reliability of the freeway facility. Ideally, a freeway facility has a high mean capacity with low variance so that it can reliably process large volumes of traffic.

For achieving a high flow (Q), short time headways (h_g) are needed, as is shown by Equation 2.4:

$$Q = \frac{3600}{\frac{1}{N}\sum_{i=1}^{N} h_i^g} \tag{2.4}$$

Where the gross headway h^g is equal to:

$$h_i^g = h_i^n + \frac{L_i}{v_i} = \frac{s_i^n + L_i}{v_i} \tag{2.5}$$

where h^n is the net (bumper-to-bumper) time headway in seconds, where L is the vehicle length in meters, where v is the speed of the vehicle in meters per second and where s_n is the net (bumper-to-bumper) distance headway in meters (see Figure 2.2).

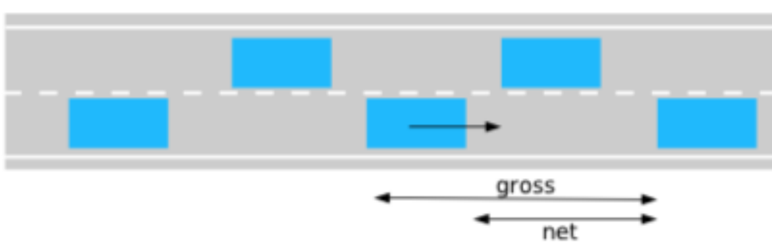

Figure 2.2. – Difference between gross- and net headways (Knoop et al., 2018, p.4)

As can be seen from Equation 2.5, three variables determine the height of the flow. The reader can verify that the driving speed has a negative influence on the time headway, and therefore a positive influence on the flow, while the net spacing and vehicle length have a positive effect on the time headway, and therefore a negative effect on the flow.

As such, it is the expectation that the mean of the breakdown flow distribution ($\mu_{Q^{F \to C}}$) will be positively influenced by the driving speed (which is affected by the speed limit) and will be negatively influenced by the number of trucks on the road, as well as other (environment) factors that cause gross vehicle spacing (s^g) to increase.

For achieving high stability, it is important that enough buffer space exists between subsequent vehicles, so that speed disturbances in the stream can be absorbed from one vehicle to the next and that traffic breakdown can be prevented. The condition under which this is the case is also called "weak string stability" for which the following criterion applies (Wang, 2018):

$$| H_i(z) |=| \frac{E_{s,i}(z)}{E_{s,i-1}(z)} |=| \frac{\text{Disturbance Magnitude Experienced by Leader}}{\text{Disturbance Magintude Experienced by Last Vehicle}} |< 1 \quad (2.6)$$

For this reason, short time headways can only be maintained in a stable manner, if the magnitude of the largest disturbance in the stream does not exceed the critical disturbance, such that the condition in Equation 2.6 is respected.

The breakdown flow height and sensitivity of the traffic stream is therefore determined at the intercept of the degree to which short time headways are maintained and the degree to which they are sufficient to absorb disturbances. Both time headways and disturbances are of a stochastic nature in reality and, as such, they both have their influence on the shape of the breakdown flow distribution.

2.2.1. Stochasticity in Time Headway Choice Behavior

Though the flow is determined by the average gross headway h_g (see Equation 2.4 and Equation 2.5), a lot of variation exists in headway choice behavior between drivers, as well as within the behavior of a particular driver. As can be seen from the graph in Figure 2.3 and, as has been shown by Marsden *et al.* (2003), Brackstone and McDonald (2007) and Risto and Martens (2013), headways are considerably lower than the 2.0 seconds that are recommended in most European countries (SWOV, 2013). Additionally, it can be seen that mean time headways are significantly shorter for higher driving speeds, which seem to "flatten out" around a value of about 1.2 seconds from 90 km/h and up (see Figure 2.3).

Brackstone and McDonald (2007) state that a large proportion of drivers generally overestimate their reaction time and braking skills and that this causes a large share of drivers to follow at close distances when in car-following mode. Moreover, Brackstone and McDonald (2007) show that drivers who engage in close following are not per definition more attentive to the driving task. This creates an additional risk to the stability of the flow (as well as a safety risk), because a longer reaction time entails that a disturbance of smaller magnitude can threaten the stability of the flow.

Additionally, Ossen (2008) has found that a major share of drivers consider more than one leader when following another vehicle and look at least two vehicles ahead in determining their speed and headway. Though Brackstone and McDonald (2007) argue that drivers should use this ability as an aid to driving in a safe and stable manner, many drivers use it to engage in even closer following and use it as a compensatory mechanism. This behavior

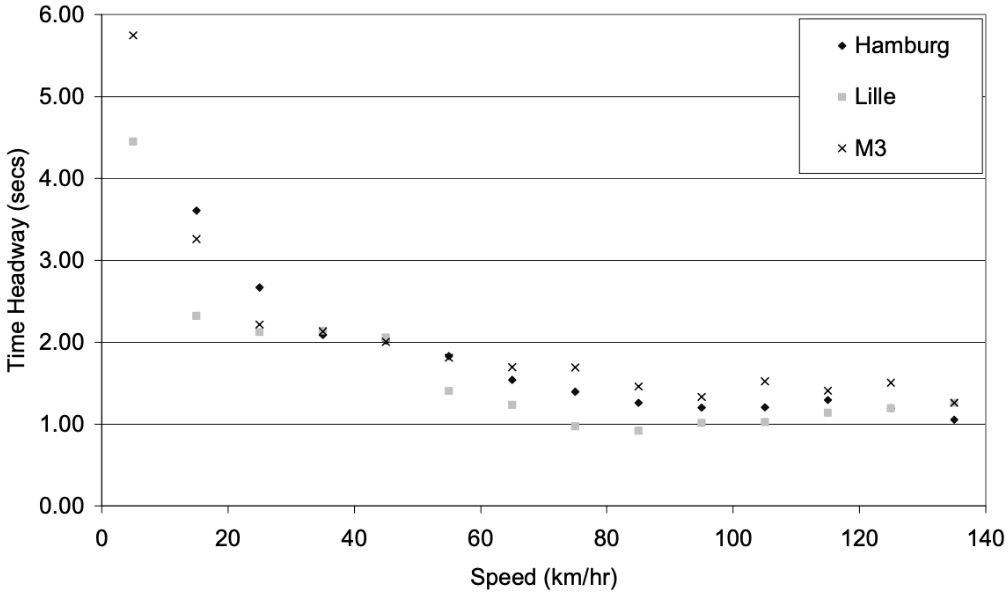

Figure 2.3. – Average time headways for different speeds, measured at three different sites throughout Europe (Marsden et al., 2003).

is consistent with the risk homeostasis hypothesis proposed by Wilde (1998), which states that in the presence of additional safety mechanisms, people adjust their behavior to match a preferred level of risk, rather than behave in a way in which absolute risk is minimized.

Perception Bounds

Marsden *et al.* (2003) and Ossen (2008) have found large variation in time headway choice among drivers. This is partly caused by heterogeneity in reaction times between drivers (Green, 2000), but it can also be attributed to driver specific thresholds for recognizing relative speeds, such as proposed in the Wiedemann model (Wiedemann, 1974). Higgs *et al.* (2011) have found, using the Wiedemann model, that these perception thresholds are dependent on the driver and are also dependent upon the speed. As such, Higgs *et al.* (2011) have found that drivers exhibit different behaviours depending on their speed and that this can imply an increase in aggression (short-distance following, stronger braking) at higher speeds. This is consistent with findings from Risto and Martens (2013), who have found that people exhibit larger absolute estimation errors regarding their headway when they are driving at higher speeds. When higher speed limits cause higher driving speeds, this could lead to more severe headway estimation errors and thus more headway variation over time, leading to less stability.

2.2.2. Stochasticity in the Occurence and Magnitude of Disturbances

There are two reasons for the existence of speed disturbances in the stream. The first reason has to do with estimation-errors and inconsistency in car-following behavior and the second reason has to do with dynamics related to lane changing.

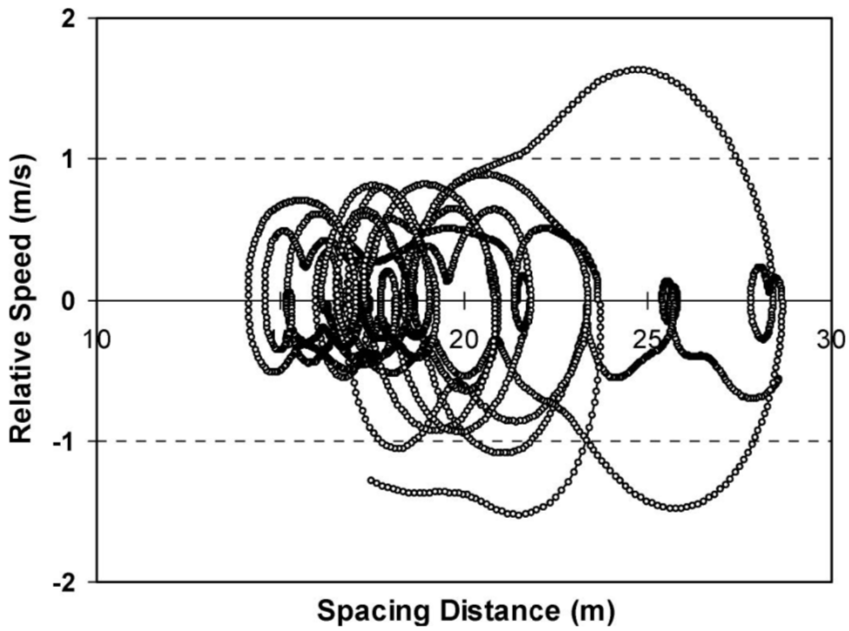

Figure 2.4. – Variation in following distance with respect to relative speed in a speed range of 30-35 m/s (Brackstone et al., 2002).

Firstly, errors in headway estimation, caused by perception thresholds and delayed reaction time, are at the root of headway inconsistency for any driver. An example of such inconsistency has been found by Brackstone *et al.* (2002), who have plotted the relative spacing of a vehicle over time against the relative speed in relation to its predecessor (see Figure 2.4). The presence of this "within driving style" heterogeneity, is one of the reasons why some degree of turbulence is always present in any traffic stream (Beinum, 2018) and why there is so much scatter around the equilibrium state of the fundamental diagram (Ossen, 2008). The degree to which drivers make errors in their headway estimation will affect the magnitude of disturbances in the stream and the frequency with which drivers will have to change their speed in response to a predecessor.

Secondly, on any multi-lane road, lane changing will occur. When making a lane change, a vehicle will leave a gap in the origin lane and will fill a gap in the destination lane. Depending on the intensity of traffic, the lane change may cause a speed disturbance in either the destination lane, the origin lane or both. If both lanes have high degrees of saturation, it can be expected that the origin lane will (potentially) experience a positive speed disturbance, as the follower of the lane-changing vehicle can accelerate to fill the gap. Similarly, in the destination lane, it can be expected that the maneuver will (potentially) lead to a negative speed disturbance, as the follower of the lane-changing vehicle in the destination lane may have to decrease its speed. These lane change disturbances can be passed on further upstream and may even trigger additional lane change maneuvers by other vehicles (Ahn and Cassidy, 2007). The degree to which drivers perform lane changes will directly affect the occurrence of disturbances, whilst the relative speed and headway, with which drivers enter other lanes, will determine the magnitude of disturbances.

2.3. How lane interaction dynamics shape the capacity distribution of a dual-lane freeway

The capacity ($Q_{50}^{F \to C}$) of a freeway facility is determined by the breakdown flow distributions of each particular lane of that facility and the way in which the lanes of the facility interact with each other. The degree to which the flow in a lane contributes to the total flow of the roadway will depend on the density (K) that is present at a particular moment. Lane flow fractions will change with changes in density and total flow (see Figure 2.5), because the lane in which drivers prefer to drive will depend on the conditions on the roadway.

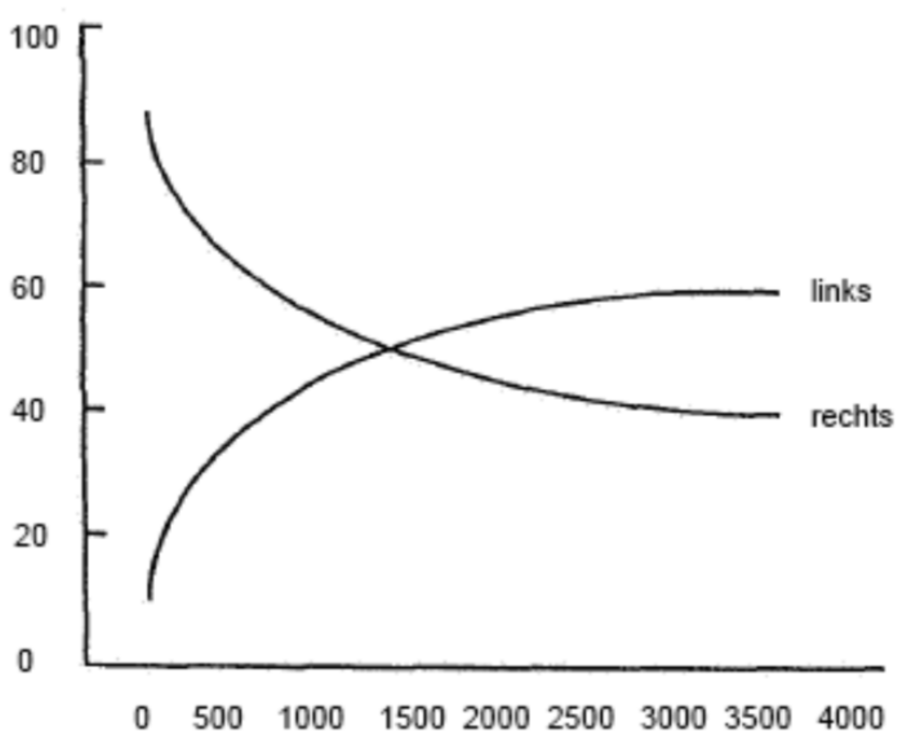

Figure 2.5. – Lane flow distribution on a two-lane freeway according to the Dutch book of guidelines on freeway design (Rijkswaterstaat, 2017, p.34) - "links" means left and indicates the passing lane, "rechts" means right and indicates the shoulder lane. Note how the flow fraction on the passing lane takes on a non-linear shape with respect to the amount of total flow.

Like in other European Countries, drivers in The Netherlands should keep right as much as possible and overtake on the left[1]. This means that different dynamics will be observed in the passing lane and the shoulder lane, which are a result of lane change behavior. Three types of lane changes can be identified: 1) mandatory lane changes, 2) discretionary lane changes and 3) voluntary (cooperative) lane changes (Knoop *et al.*, 2018).

Mandatory lane changes are lane changes that are needed because a certain lane ends or because they are needed to reach a certain destination. Mandatory lane changes will be

[1]article 3.1; RVV1990 (Rijksoverheid, 2020).

performed regardless of the traffic situation. For mandatory lane changes it was found for on-ramps and merging lanes that there was large variation in the moment at which drivers would speed up to synchronize with the main lane (Keyvan-Ekbatani *et al.*, 2016). Gap selection was deemed to be an easy task by most participants, as they would not mind driving faster than traffic in the main stream for a short period of time in order to merge into a gap.

Discretionary lane changes are performed at the discretion of the driver and often serve the purpose of improving the driving experience of the driver performing the lane change. Examples of which are: changing lanes to a faster lane (travel time concern), changing lanes towards a lane with less trucks (safety concern) or changing lanes towards a lane with more "relaxed" traffic dynamics (comfort concern). It is expected that discretionary lane changes will occur only if there are sufficient opportunities to do so (density is low) or if the (potential) gain is sufficiently high (Schakel *et al.*, 2012).

Lastly, voluntary (cooperative) lane changes may be performed by a driver to make room for another driver. It is expected that this type of lane change will primarily occur during periods of low demand ($K << K^*$).

Lane Changing Strategies

For Discretionary lane changes, several strategies have been identified by Keyvan-Ekbatani *et al.* (2016), who have found that Dutch drivers drive in accordance with at least 1 of 4 strategies (for discretionary lane changes) which are listed below:

1. Speed Leading: a driver has a certain preference speed and chooses the lane that best matches this speed
2. Speed Leading with overtaking: Same as strategy 1, but with a stronger bias to driving on the right lane and speeding up when overtaking another vehicle to decrease the time of the maneuver.
3. Lane Leading: a driver has a range of acceptable speeds and tries to stay in a lane of preference as long as the speed is in this range.
4. Traffic Leading: a driver will adapt its speed choice behavior to the behavior of other vehicles and "drive with the flow".

According to De Baat (2016), 96% of the drivers in the experiment applied a combination of at least two strategies. It was found that most drivers choose either strategy 1 or strategy 2 and switch to strategy 3 or 4 when traffic becomes more dense and congestion occurs (De Baat, 2016).

Capacity Effects of Lane Changing

A vehicle that is changing lanes needs to have a gap in the target lane and will leave a gap in the lane from whence it came. As a consequence, lane changes in dense conditions will create voids in the stream that travel downstream with the free-flow speed (Laval and Daganzo, 2006). The capacity effects of such lane changing can be particularly dramatic when they are caused in a situation in which the density in the target lane is already very high and when the void in the original lane is not filled up by another vehicle.

Despite the "keep right" rule, many drivers feel inclined to drive on the left lane during times of high demand (see Figure 2.5). According to De Baat (2016) a higher driving speed was the most important reason for this, followed by factors such as the presence of trucks in the shoulder lane and irregular flow on the shoulder lane caused by merging vehicles.

Additionally, many drivers also prefer to stay on the passing lane because they fear not being able to return to the passing lane easily after changing to the shoulder lane (De Baat, 2016). This fear is not unfounded, as it has been found that under conditions of high flow, the density in the passing lane is often high and drivers in this lane generally maintain tight spacings to anticipate on overtaking the vehicle ahead and to "ward-off" other drivers from entering their lane (Daganzo, 2002a) (Ahn and Cassidy, 2007). This behavior can particularly be expected to be present when the difference between the speed limit and the speed of traffic is large, as drivers become more irritated about their perceived travel time loss.

Lane Flow Distribution

It has been found that speed limits and the level of enforcement do have some effect on the lane flow distribution. Knoop *et al.* (2010) have found that the percentage of flow in the passing lane is positively related to the driving speed and that when lower speeds are imposed the distribution shifts away from the passing lane(s) towards the shoulder lane, which has also been found in studies by Duret *et al.* (2012) and Soriguera *et al.* (2017). Additionally, under strict speed enforcement, such as average speed checks, the lane distribution has been found to shift to the lanes on the right, leaving the leftmost lane relatively empty (Tool *et al.*, 2006).

As long as the flow distribution (in passenger car equivalents) is not sufficiently balanced, the flow will contain unnecessary voids, which will reduce the capacity of the road. Based on the research by Knoop *et al.* (2010) it may be expected that a lower speed limit will contribute to a shift in the lane flow distribution, which will be tested in section 4.5 and section 5.3.

Lane Flow Distribution at the Moment of Breakdown

In conclusion, it can be expected that lane flow fractions at the moment of breakdown may diverge quite a lot, with more vehicles present in the passing lane than in the shoulder lane. Because of this, breakdown will most likely occur in one of the lanes of the freeway facility and, subsequently, "spillback" to other lanes of the roadway. As such, plotting the breakdown flow distributions for each specific lane will not directly imply that this is the capacity of that specific lane, as the breakdown of flow may be a result of spillback from another lane in which the capacity may have been reached. It is therefore that all lane flow distributions will be referred to as breakdown flow distributions, rather than capacity distributions.

It is the expectation that the breakdown will be triggered in the passing lane (Cassidy and Bertini, 1999) (Daganzo, 2002a) and that, for this reason, breakdown flows in the passing lane can be seen as the capacity of that lane and will be strongly related to total capacity of the roadway. For the shoulder lane, on the other hand, it can be expected that flows in this lane will merely be the flow at which breakdown occurs, regardless of whether capacity has been reached. It is therefore expected, that the level of flow in this lane will only be weakly related to the capacity of the roadway.

2.4. Expected Dynamics on Two Lane Freeways

In this thesis, the behavioral "slugs and rabbits" theory by Daganzo (2002a) is used, because it has been found to apply well to the data obtained from two-lane freeways. The theory will be viewed in the light of three types of freeway layouts that are relevant to the analyses performed in chapter 4

1. Homogeneous Sections
2. On-ramp Areas
3. Lane Reduction Areas

2.4.1. Expected Dynamics on Homogeneous Sections

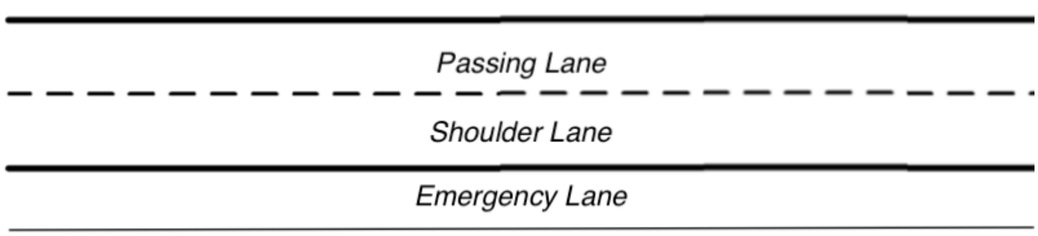

Figure 2.6. – Graphical depiction of homogeneous two-lane freeway section (Rijkswaterstaat, 2017).

In the slugs and rabbits theory, a simplification is suggested in which there are two types of drivers: "Slugs", which are vehicles that have a low preference speed (who may also be restricted by a lower legal limit for their category) and prefer to stay in shoulder lanes, and "Rabbits", which have a high preference speed and who frequently move into the passing lane to overtake slugs (Daganzo, 2002a).

When the density is very low ($K << K^*$), (almost) all vehicles drive in an unconstrained manner and all the rabbits are able to overtake with ease whenever they want to - i.e. the overtaking probability tends to 1. The mean speed on the passing lane is in this case always higher than the mean speed on the shoulder lane, since only the faster drivers will be in this lane (Daganzo, 2002a).

As the density K gradually increases towards K^* the number of rabbits that will be constrained in their speed by some other vehicle, either a slug on the shoulder lane or a slower rabbit on the left lane, will increase and passing opportunities will decrease as density K increases further towards the critical density ($K \to K^*$). Daganzo (2002a) consequently argues that, as the density increases, the rabbits will move to the passing lane as long as the speed in the passing lane V^p is higher than the driving speed in the shoulder lane V^s

($V^p > V^s$) (Daganzo, 2002a). Due to this behavior, the passing lane becomes very crowded with rabbits who have a desire to pass the slower vehicles in front of them. Daganzo goes on to postulate that, in this case, the "act of passing (or the anticipation of passing) triggers a psychological change into a 'motivated' frame of mind" (Daganzo, 2002a, p.137). With the term "motivated", Daganzo (2002a) means that drivers in the passing lane will accept a much shorter headway than they generally would, because they have the expectation that, at some point, they will be able to pass the vehicle in front of them (Daganzo, 2002a, p.137). Daganzo (2002a) also mentions that this "motivated behavior" disappears when it is no longer possible to overtake on the passing lane(s) Daganzo (2002a).

The proposed outcomes of this theory are consistent with findings for a high occurrence of short headways by Brackstone and McDonald (2007). Moreover, the large share of vehicles that will shift to the passing lane under high density conditions is consistent with the large number of drivers using a speed-leading strategy, as concluded by De Baat (2016). Additionally, the expectation of a high proportion of flow in the passing lane at the moment of breakdown is consistent with the findings in chapter 4 and chapter 5.

As a consequence of these dynamics, the pre-breakdown traffic situation (where $K \leq K^*$) is characterized by a two pipe regime, with different fundamental diagrams for each of the lanes (as displayed in Figure 2.7). Since the speed limit affects the speed at which drivers prefer to drive, it can be expected that, under a speed leading strategy, a stronger tendency to drive in the left lane will prevail under conditions of high flow. Changes in lane choice behavior, induced by a speed limit change, will affect the lane flow distribution which may, in its turn, potentially affect the capacity of the roadway.

2.4.2. Dynamics in On-ramp Areas

In an on-ramp area (such as in Figure 2.8) it is assumed that, during periods of high demand, the flow on the main road is neatly separated in a two pipe regime, where the rabbits are in the passing lane and the slugs are in the shoulder lane. From the on-ramp a stream of vehicles emerges, consisting of both slugs and rabbits, who are entering the merging lane and merge into the shoulder lane (see Figure 2.9).

Though a physical baffle (as presented in Figure 2.9), preventing the movement of vehicles between both lanes, is not always present on Dutch freeways, it helps to understand the fact that some time is spent by the shoulder lane rabbits to find a suitable gap in the passing lane (as long as the speed in this lane is higher than in the shoulder lane). During this time, the rabbits move downstream in the shoulder lane with the speed of the slugs. In accordance with the assumptions of the Lane change Model with Relaxation and Synchronization (LMRS) by Schakel *et al.* (2012), it is expected for the shoulder lane rabbits that, the longer they stay in the shoulder lane, the higher their lane change desire will become, hence reducing their gap acceptance threshold in the passing lane. Therefore, as time progresses, it is expected that the shoulder lane rabbits will attempt to synchronize with the passing lane and merge into a gap, if one is created, or force themselves into a space between two passing lane rabbits that they deem sufficient.

The merging procedure, especially if it is a forced entry, will create a speed disturbance in the passing lane, which will threaten the stability of the vehicle chain in this lane as defined in Equation 2.6. Depending on the headways and reaction times of the vehicles upstream of the merging procedure, as well as the heterogeneity of the vehicle chain in this lane, the

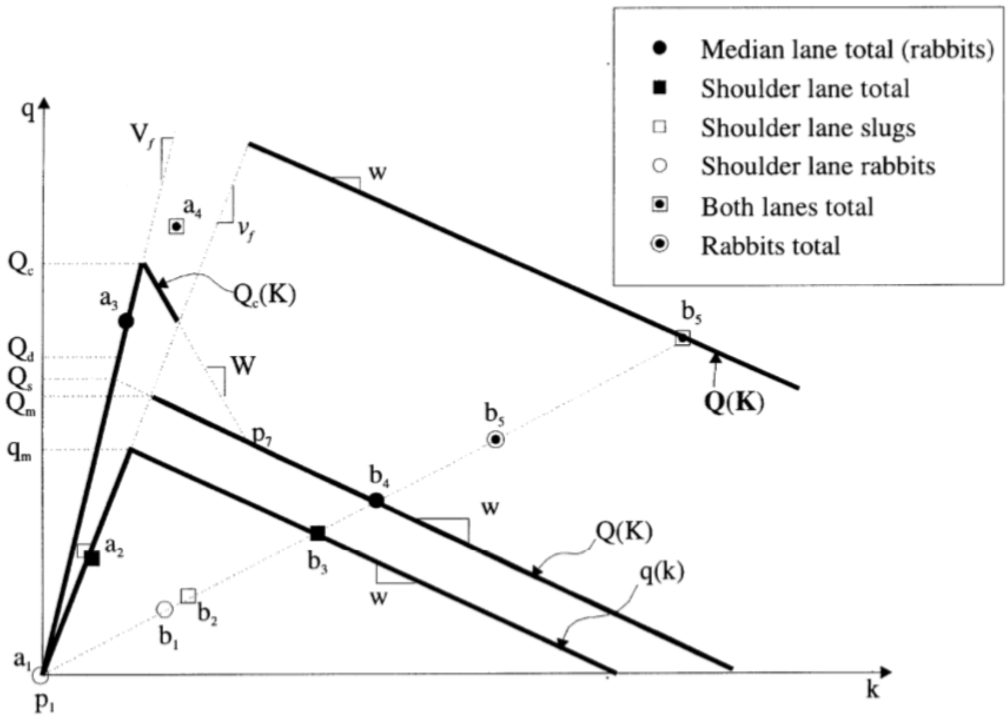

Figure 2.7. – Fundamental diagrams in the two pipe regime; the small diagram for the shoulder lane and the larger diagram (with capacity drop) for the passing lane (Daganzo, 2002a, p.139)

disturbance will either be absorbed, and disappear, or amplified, resulting in a disturbance of growing magnitude, propagating upstream against the direction of flow with the wave speed of the characteristic (Knoop *et al.*, 2018).

For a backward propagating characteristic of sufficiently large magnitude, the initial lane change may even induce additional lane changes upstream, resulting in a further loss of stability (Ahn and Cassidy, 2007). In accordance with Kerner (2004) it is expected that, first, a transition from the free flow state to the synchronized state will occur with the initial lane change, as is described by the collapse of the two-pipe regime into the one-pipe regime by Daganzo (2002a), and that, second, backwards propagating jam waves may occur as a result of additional lane changes caused by a change in the regime. In this thesis only the $(F \rightarrow C)$ transition from the free flow state to the synchronized flow state will be considered, as this transition, despite potentially higher values of flow in the synchronized state, will induce travel time losses that are expected to be significant.

Lastly, the fact that the shoulder lane rabbits need some time to find a gap in the passing lane, combined with the fact that after merging into the passing lane, their followers will engage in close following for a short time before "relaxing" back into a longer headway (Schakel *et al.*, 2012), explains why it has been found by Beinum (2018) that levels of turbulence are highest in a region of 300 to 900 meters downstream of the gore of an on-ramp. Also, by means of investigating speed-contour graphs at various locations, it has been verified that most traffic breakdowns originate somewhat downstream of the merging area.

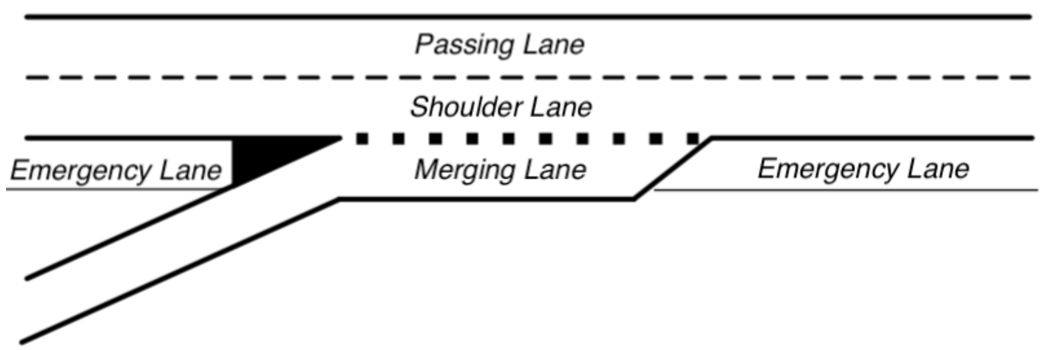

Figure 2.8. – Graphical Depiction of On-Ramp Area on Two Lane Freeway Section (Rijkswaterstaat, 2017).

This knowledge will prove to be useful for choosing appropriate detector locations, in areas where no inflow data for the on-ramp is available (see chapter 3).

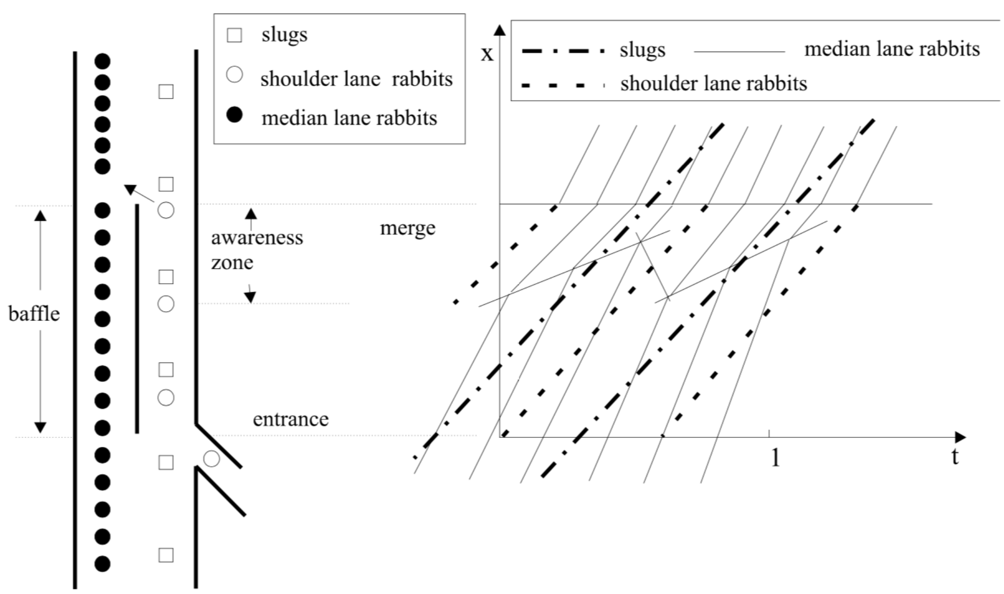

Figure 2.9. – Schematic depiction of Slugs and Rabbits entering main road from on-ramp with corresponding trajectory plot (Daganzo, 2002b, p.163).

2.4.3. Dynamics near Lane Reductions

A third, and last, freeway layout that is considered, is a three-to-two lane reduction. Different dynamics are expected in such a layout, because the merging lane will primarily be filled with rabbits instead of a mix. Though Daganzo (2002a) has not explicitly formulated this situation, the premise of the "slugs and rabbits" theory is clear enough to make a prediction about the type of dynamics that can be expected in a section such as presented in Figure 2.10.

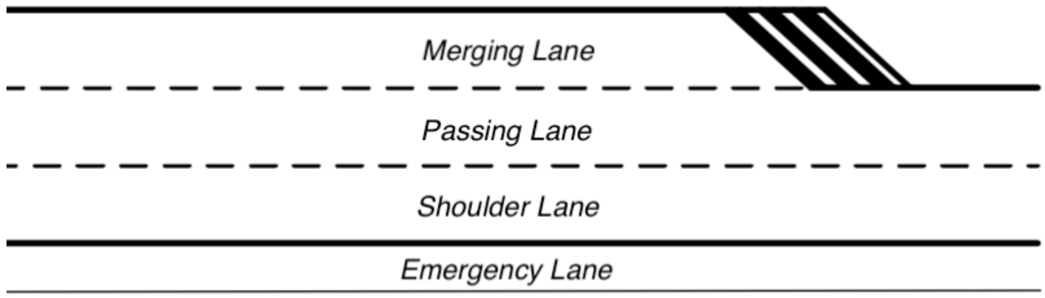

Figure 2.10. – Graphical depiction of lane reduction area on two-lane freeway section (Rijkswaterstaat, 2017).

As verified by Knoop *et al.* (2010), Yuan (2016) and Beinum (2018), in a regular, homogeneous three lane freeway section near capacity, the highest proportion of flows are often on the median (left-most) lane and center (middle) lane, with the median lane generally having a slightly higher proportion, and the lowest proportion of flow on the shoulder lane (for an example see Figure 2.11).

In the situation depicted in Figure 2.10 it is expected that the passing lane will be the busiest lane, because most vehicles will already merge from the merging lane into the passing lane upstream of the lane reduction. It is expected that in the three lane area, the shoulder lane will only contain slugs, the passing lane will contain some slugs, but mostly rabbits and the merging lane will only contain rabbits. On the basis of the findings by De Baat (2016) it is expected that, just before the lane reduction, the rabbits in the merging lane will move into the passing lane, leading to a saturation of the passing lane, while the shoulder lane is relatively empty. Next, after passing the lane reduction point, some of the slugs in the passing lane will start feeling uncomfortable at the short following distances caused by the saturation of the passing lane and will move into the shoulder lane whenever they see an opportunity to do so.

If the saturation rate in the passing lane is already very high before the lane reduction location, there is a large probability that a merging lane rabbit that forces itself into the passing lane, will trigger a disturbance in the passing lane somewhere downstream of the merging point (due to relaxation effects). If this disturbance is of sufficient magnitude to cause further lane changes upstream and a transition into the synchronized flow regime, it will lead to traffic breakdown.

2.5. Other Factors that are known to affect capacity

An extensive overview of factors that are known to affect capacity, including supporting literature, is provided by Rijkswaterstaat in the form of a highway capacity manual (Heikoop *et al.*, 2015). In this section a description will be given of the relevant factors and how they are accounted for in this thesis.

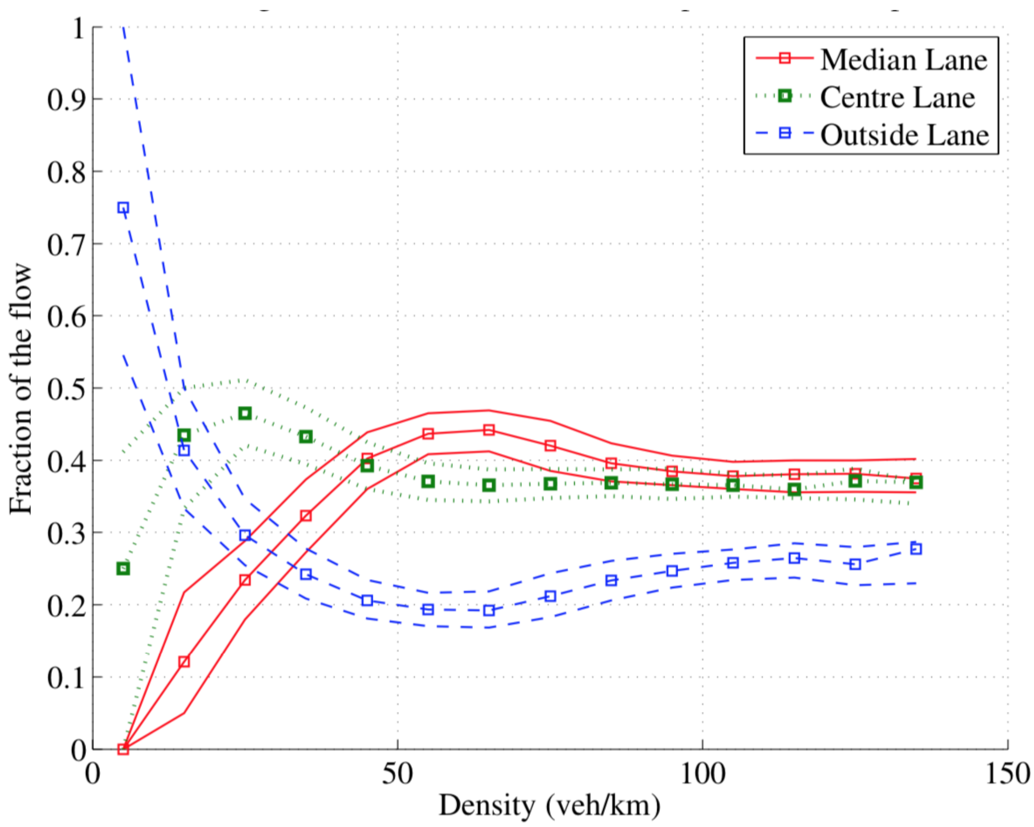

Figure 2.11. – Lane flow distribution on homogeneous three lane section (Knoop et al., 2010)

Infrastructural Factors

As can be seen from Table 2.1, it is primarily lane width that can aid in increasing capacity per lane. The number of lanes does positively influence the capacity from one to two lanes because it allows overtaking, making it more likely that gaps in the stream will be filled, while any increase from two to five lanes will negatively affect the capacity per lane by a small amount (Heikoop *et al.*, 2015, p.31). Any increase beyond five lanes has been found to be inefficient, as the left-most lanes are underutilized (Heikoop *et al.*, 2015). As such, it is better to use a main-parallel construction in these cases. The remainder of the infrastructural factors reduce the capacity per lane for obvious reasons such as reduction of sight line, narrow field of vision, close object distances and reduced acceleration power. The type of roadway surface (Closed Asphalt Concrete vs. Open Asphalt Concrete) matters primarily in times of precipitation and a reduction in capacity for Closed Asphalt Concrete is caused by a reduction in sight distance (see Figure 2.12).

Differences in infrastructural factors are implicitly accounted for in the analysis of chapter 4, because they are taken into account when comparing locations on the basis of a before and after period, in which only an exogenous change in the speed limit has taken place, and they are explicitly accounted for in the analysis in chapter 5 through the use of location-specific dummy variables (see subsection 3.2.2).

Factor	Effect (per lane)
Infrastructural	
Number of Lanes	Positive*
Lane Width	Positive
Object Distance	Negative
Rush Hour Lane Presence	Negative
Vertical Inclines	Negative
Horizontal and Vertical Arcs	Negative
Type of roadway surface	Negative**
Tunnel	Negative
Meteorological	
Rain	Negative
Mist	Negative
Illuminence	Positive
Traffic Management	
Ramp Metering Installation	Positive
Motorway Traffic Management System	Positive
Truck overtaking ban	Positive***
Traffic Composition	
Truck Presence	Negative
Weekend	Negative

* *Capacity per lane increases from 1 to 2 lanes; gradually decreases from 2 to more lanes*
** *Negative for closed asphalt concrete*
*** *Positive at low truck traffic levels, negative at high truck traffic levels*

Table 2.1. – Overview of capacity factors (Heikoop et al., 2015)

Meteorological Conditions

Meteorological conditions such as rain (Calvert, 2016) and mist have a negative effect on capacity, while daylight conditions (degree of illuminence) have a positive effect on capacity (Heikoop *et al.*, 2015). Rain and mist conditions are not directly accounted for in this thesis and it could be the case that in the before and after comparison of capacities, different amounts of precipitation have occurred (mist is relatively rare and its effect is therefore assumed to be negligible).

Illuminence conditions have been accounted for by taking the same study period of several months one year apart from each other. In this way it can be expected that illuminance conditions are approximately the same in both samples. It is the expectation that, by taking relatively long periods of measurement and by choosing the same period of the year to compare with each other across different years, the effects from weather conditions on the generated capacity distributions will be small. Nonetheless, one has to take into account that some noise may be induced by meteorological variables into the analysis.

Figure 2.12. – Effect of rain on Open Asphalt Concrete (left) and Closed Asphalt Concrete (right)

Traffic Management

Traffic Management measures will generally have a positive effect on the capacity per lane (Heikoop *et al.*, 2015). It will be checked for each location whether the traffic management measures have stayed the same in the before and after period, to make sure that these effects do not interfere with the results.

Traffic Composition

In weekends there are more people driving to places they do not frequently visit and, as such, drivers are less familiar with the road they drive on, leading to less efficient behavior and lower capacities. Weekends are taken into account in this sample for completeness, though they are generally expected to be represented to a much lesser extent than weekdays, simply because congestion occurs more frequently during weekdays than during weekends.

Additionally, the number of trucks that is present on the road has a very large negative influence on the capacity per lane (Heikoop *et al.*, 2015). It is not certain whether the percentage of trucks from one year to the other may stay constant on the locations under study. As such, truck presence should be accounted for in the analysis to the best extent possible. For locations for which truck percentage data is available, data will be presented.

2.6. Summary

In this chapter it was defined that there are two states of traffic (F and C) that are distinguished on the basis of the average driving speed. It was determined that the breakdown flow $Q^{F \to C}$ is the flow that occurs at the moment at which traffic transitions from the free flow state F to the congested state C, which can be viewed as the realization of an instance of the "free-flow" capacity. The $F \to C$ transition can occur for a range of flows, which entails that the free-flow capacity is a stochastic variable that follows a distribution.

Longitudinal Driving Behavior

It was discussed that both longitudinal and lateral behavior can affect the height of the capacity distribution. It was shown that increased vehicle lengths have a negative effect on the level of flow, meaning that increased levels of truck traffic will reduce capacity, while the speed has a positive effect on the level of flow, meaning that higher speed limits could potentially increase capacity.

It was determined that the breakdown flow in a single lane is determined by an interaction between the size of average headways and the frequency and magnitude of disturbances in the traffic stream. Observed headways in literature were found to be lower than those recommended by government guidelines and it was found that large variation between drivers, as well as within driver behavior, is present for headway choice. It was proposed that this is due to a combination of the effects of multi-anticipation in car-following as well as differences in perception bounds between drivers.

Platoon Formation and Stability

It has been explained that during periods of high traffic demand, a number of slow vehicles will become moving bottlenecks in the stream, which will lead to fast-moving queues of vehicles in the passing lane who want to overtake these moving bottlenecks. Generally, very short headways will be observed in these fast-moving queues. For this reason, it is expected that breakdown will occur in the passing lane first, before spilling back to the shoulder lane.

Additionally, it was discussed that, on the one hand, short headways are necessary for achieving high flows, while on the other hand, short headways increase the sensitivity of traffic to disturbances in the stream. Mean headways have been proven to become shorter for higher driving speeds (see Figure 2.3), but it is uncertain to what extent this will decrease the overall stability of the traffic stream.

Lane Choice Behavior

In addition to longitudinal behavior, lane choice behavior is also important for the determination of capacity. It was discussed that most drivers drive in accordance with a speed-leading strategy, in which they choose the lane in which the driving speed most closely matches their preferred speed. Despite the keep-right rule, it was found that, as more vehicles are present on a given road, the fraction of flow on the passing lane, which has a higher average speed, will become much higher than on the shoulder lane. Additionally, drivers in the passing lane are unlikely to move back into the shoulder lane because they do not want end up "stuck" between slower vehicles in this lane. If the height of the speed limit affects lane change behavior, it may be the case that a higher speed limit leads to inefficient roadway use, which may result in a lower roadway capacity.

Slugs and Rabbits

It was discussed how the behavioral "slugs and rabbits" theory from Daganzo (2002a) explains many features of the traffic flow characteristics that can be observed in the data for the two lane freeway samples in chapter 4 and chapter 5. The theoretical two-pipe fundamental diagram was presented (see Figure 2.7), which can be reproduced by plotting the measured speed and flow data from study location in this thesis in the flow-density plane (which has been done in Figure 4.2). Descriptions were given of how traffic was expected to behave in the types of bottlenecks that are investigated in this study, so that hypotheses could be formulated from these expectations, which are presented in chapter 3.

Other factors known to affect capacity

An overview was provided of other factors that are known to affect capacity from literature. It was discussed that infrastructural factors and traffic management factors are implicitly accounted for, by comparing a given location by means of a "before" and "after" measurement, between which no changes to these type of factors have occurred. Lastly, traffic composition cannot simply be expected to be the same for the "before" and "after" period. As such, for locations were data is available, truck traffic data will have to be included into the analysis to the best extent possible.

3. Methodological Framework

In this chapter, the research question and related sub-questions are discussed. From these sub-questions a total of 8 hypotheses have been derived which will be tested in chapters 4 and 5. In addition to this, relevant methods that will be applied in this thesis are described and explained. Especially the Product Limit Method and Least Squares Regression theory are discussed, which are the primary methods of analysis that have been applied in this thesis. Also some statistical tests are discussed which are applied for performing means and variance testing, as well as a non-parametric test (willcoxon signed rank sum test), for the evaluation of incomplete capacity distributions. Subsequently, the data collection process and locations at which data is collected are discussed and an overview is given of detector locations and measurement periods. Lastly, a short section on findings regarding speed choice behavior under different limits (100, 120 and 130 km/h) is presented.

3.1. Research Question and Sub-Questions

In chapter 1, the research question of this thesis was posed which stated:

To what extent does the height of the speed limit affect freeway capacity?

To answer this question, five sub-questions have been formulated:

1. Given a change in the speed limit, can a change in the capacity distribution be observed at a given location?
2. Given a change in the capacity distribution between measurement periods, can this change also be related to changes in the traffic composition at a location?
3. Given a change in the capacity distribution between measurement periods, are significant changes in lane choice behavior visible?
4. When controlling for other relevant variables and location specific factors, can a general change in breakdown flows be attributed to a change in the legal limit?
5. Is there a significant relationship between the speed limit and the utilization rate of the passing lane?

3.1.1. Sub-Question 1: Given a change in the speed limit, can a change in the capacity distribution be observed at a given location?

To assess this question to the best extent possible, eight locations will be analyzed where a speed limit change from 120 km/h to 130 km/h has occurred. In chapter 4 a before and after examination will be performed for the years surrounding a speed limit change, where flow and speed data will be used from the same period of the year (see Table 3.2). Breakdown flow probability distributions will be generated through the application of the product limit method so that changes to the breakdown probability, and thus the capacity, can be compared for both measurement periods, to obtain an indication of the likelihood that the speed limit change has led to a capacity change.

To provide an answer to this question the following alternative hypotheses (for which the null hypothesis is their negation) will be tested in sections 4.1, 4.2 and 4.3 respectively:

Hypothesis 1a:

- H_1*: The capacity distribution for the complete roadway is significantly different for the before and after period*

Hypothesis 1b:

- H_1*: The breakdown flow distribution for the passing lane is significantly different for the before and after period*

Hypothesis 1c:

- H_1*: The breakdown flow distribution for the shoulder lane is significantly different for the before and after period*

3.1.2. Sub-Question 2: Given a change in the capacity distribution between measurement periods, can this change also be related to changes in the traffic composition at a location?

In case that it has been found in sub-question 1 that the breakdown probability distribution has changed between measurement periods, it is important to investigate whether no significant changes to the traffic composition have occurred at a particular location between measurement periods, as it has been found in chapter 2 that truck traffic levels have a large effect on capacity. To this end, truck traffic data has been gathered over each measurement period and have been used in section 4.4 to determine whether changes in truck traffic could have caused changes to the capacity distribution.

As such, the alternative hypothesis (for which the null hypothesis is its negation) to be tested in section 4.4 is:

Hypothesis 2:

- H_1*: A significant change to the level of truck traffic has occurred between the before and after period*

3.1.3. Sub-Question 3: Given a change in the capacity distribution between measurement periods, are significant changes in lane choice behavior visible?

As was discussed in chapter 2, it can be expected that a change in the speed limit may cause subsequent changes in lane choice behavior. For this reason, it is deemed of interest to investigate whether significant differences in the utilization rate of the passing lane can be found for the locations tested in chapter 4. Changes to the lane flow distribution are relevant because they may be indicative of inefficient lane use. A disproportionately large proportion of total flow in either the passing or shoulder lane must be less efficient than a more equal distribution, which entails that there must be some optimal division of flow which maximizes capacity. This does not necessarily have to be a 50/50 division (especially when traffic counts are not corrected for PCE values) as some degree of dynamism is needed in traffic to make sure that inevitable gaps that emerge in the stream are filled up again.

To answer this question, the following alternative hypothesis (for which the null hypothesis is its negation) is tested in section 4.5:

Hypothesis 3:

- H_1*: The passing lane utilization rate has significantly changed between the two measurement periods.*

3.1.4. Sub-Question 4: When controlling for other relevant variables and location specific factors, can a general change in breakdown flows be attributed to a change in the legal limit?

Just comparing before and after measurements per location does not give a general idea of the effect of a change in the speed limit per se. Moreover, one would like to estimate the effect of a particular speed limit whilst controlling for other variables, such as the lane flow distribution and truck traffic, and location specific effects. By using the categorization process of the Product Limit Method for generating a set of breakdown flow measurements and using this set in a sample in which also lane flow data and truck traffic data is included for these observations, regression theory can be applied to analyze the effect of the speed limit more clearly and to discover a more general case which applies to multiple locations.

To this end, in sections 5.1 and 5.2, regressions have been performed on an 8-location sample (including truck traffic) and a 17-location sample (excluding truck traffic) respectively. In these regressions the following alternative hypotheses (for which the null hypothesis is their negation) will be tested:

Hypothesis 4a:

- H_1*: The speed limit variable of the eight-location sample has a significant effect, when controlling for location specific factors and other relevant variables.*

Hypothesis 4b:

- H_1*: The speed limit variables of the seventeen-location sample have a significant effect, when controlling for location specific factors and other relevant variables.*

3.1.5. Sub-Question 5: Is there a significant relationship between the speed limit and the utilization rate of the passing lane?

As mentioned before, it is very likely that there is some optimal distribution of flows over the lanes, as a very unequal distribution must indicate inefficient use of the roadway. If it is found that the speed limit affects lane choice behavior, this may be an important mechanism behind why capacity may be different for different speed limits. To this end, it will be tested whether a significant relationship between the speed limit and passing lane utilization rate can be found, even when controlling for location specific effects.

As such, the following alternative hypothesis (for which the null hypothesis is its negation) will be tested:

Hypothesis 5:

- H_1: *The speed limit has a significant effect on the lane flow distribution, when controlling for location specific effects.*

3.2. Relevant methods

To effectively assess the hypotheses that were presented in the previous section, several methods will be applied, which will be outlined in the remainder of this section.

3.2.1. PLM

As was argued in chapter 2, capacity should be treated as a stochastic variable that follows a distribution that can be estimated. A method which is able to estimate a capacity probability distribution from available roadway data is the Product Limit Method (PLM). In this paper the PLM, as applied by Brilon *et al.* (2005), will be used for generating all of the capacity probability distributions that are presented in chapter 4 and chapter 5.

Philosophy of the method

Using the concept of capacity as "the maximum flow that occurs (just) before breakdown", Brilon *et al.* (2005) argue that we know that a flow measurement is a capacity measurement if it is sufficiently high to cause traffic breakdown in the next measurement interval. The dynamic at play here is that, in the meta-stable free-flow state F_{ms} (Knoop *et al.*, 2010), headways are condensed to below-equilibrium values as a consequence of the high flow. This will lead to smaller buffer space between vehicles and will cause a drop in the average speed U when a disturbance in the flow occurs that exceeds the critical threshold of one or more vehicles in the stream (Kerner, 2004).

Consequently, Brilon *et al.* (2005) propose that one should look for flow measurements Q_t at a given location for which the speed in the current interval U_t is above or equal to the critical speed U^* and for which the speed in the next interval U_{t+1} is below the critical speed U^*, to indicate that a $F \rightarrow C$ transition has taken place. In summary, the method determines that the flow Q_t at time t is a capacity value if the condition in Equation 3.1 is the case:

$$U_{t+1} < U^* \leq U_t \tag{3.1}$$

It is paramount for the determination of the capacity distribution at a selected bottleneck location that the drop in speed is caused by an $F \rightarrow C$ transition that has occurred in the bottleneck and is not caused by a jam wave coming from another bottleneck downstream of the study area.

To this end, an upstream- and downstream detector should be used for determining whether the drop in speed which is measured at the upstream detector can be attributed to an $F \rightarrow C$ transition in the study area by taking speed data from the detector downstream of the congestion formation area and checking whether a drop in speed at the upstream detector is preceded by a drop in speed at the downstream detector in a preceding interval (see Figure 3.1).

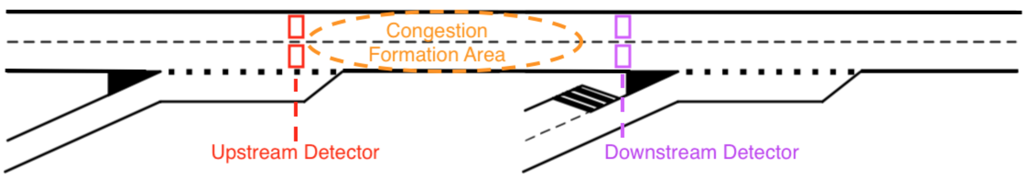

Figure 3.1. – Locations of upstream detector, downstream detector and congestion formation area (edited from Heikoop et al. (2015, p.32)).

Categorization Process

The first step in the PLM as proposed by Brilon *et al.* (2005) is the categorization process for which three series of measurements are needed, which follow a time-series structure with subsequent measurements:

- Flow measurements from the upstream detector (Q_t^{up})
- Speed measurements from the upstream detector (V_t^{up})
- Speed measurements from the downstream detector (V_t^{down})

One of four categories is assigned to each flow measurement Q_t^{up} based on the following conditions:

- **IF** at observation time t the measured speed at the upstream location (V_t^{up}) is below the critical speed ($V_t^{up} < V^*$) then the measurement Q_t^{up} must be categorized as a congested measurement, which is denoted as **category** $C1$ (which indicates traffic state C)
- **ELSE IF** at observation time t and at observation time $t+1$ the measured speeds at the upstream location are above or equal to the critical speed ($V_t^{up} \geq V^*$ **AND** $V_{t+1}^{up} \geq V^*$), then the measurement Q_t^{up} must be categorized as **category** F (which indicates traffic state F).
- **ELSE IF** at observation time t **OR** at observation time $t-1$ the measured speeds at the downstream location are below the critical speed ($V_t^{down} \leq V^*$ **OR** $V_{t-1}^{down} \leq V^*$)

then the measurement Q_t^{up} must be categorized as **category** $C2$ (which indicates traffic state J, which is a sub-state of C, for a jam coming from downstream of the study area).

- **ELSE** the measurement Q_t^{up} should be defined as a capacity measurement of **category** B (which indicates the $F \to C$ transition), because $U_{t+1} < U^* \leq U_t$ holds and the jam does not originate from downstream of the study area.

Distribution Estimation Process

The beauty of the method proposed by Brilon *et al.* (2005) lies in the fact that it acknowledges that not every period of high flow causes a traffic breakdown. It is very tempting to take just the "breakdown" measurements of category B and generate a capacity distribution from these values in a conventional way. Doing this, however, would be incorrect because it negates all the values of flow (from category F) for which the traffic flow did not break down, which may be higher than some of the breakdown measurements, and, as such, the distribution obtained by applying the conventional method is false.

Application of the method

For the determination of the capacity probability function, therefore, Brilon *et al.* (2005) propose the following function:

$$P_c(Q) = 1 - \prod_{Q_t \leq Q} \frac{k_t - d_t}{k_t}; \, t \in B \tag{3.2}$$

Where k_t is the number of observations from categories B and F that exhibit higher flows Q than Q_t, where d_t is the number of observations from category B that exhibit a flow equal to Q_t and where all observations t belong to category B. By plotting all values $P_c(Q)$ against corresponding values Q a plot of the capacity probability function can be obtained.

Emergent Issues of the PLM

The first issue is that it is not always possible to estimate the full empirical capacity distribution. This is due to a combination of factors.

Firstly, there is the issue that higher capacity values are less likely to be measured than lower ones, because the higher the flow becomes, the more sensitive it will become to the natural oscillations that occur in dense traffic (Kerner, 2004), so that breakdown may have occurred long before the highest possible capacity value has been reached (Brilon *et al.*, 2005).

Secondly, the method is structured in such a way that all breakdown measurements from set B are compared to all measurements in both set B and set F. If any flow value Q_t in set F (which has not lead to a breakdown) is higher than the highest flow value Q_t in set B, the maximum (theoretical) capacity value will not be reached - this is because the numerator in Equation 3.2 will not become zero for the highest flow value Q_t in set B (i.e. $k_t - d_t > 0$) because $k_t > d_t$.

Lastly, one has to make an important choice for two fundamental factors: the critical speed (U^*) and the time aggregation interval (t) that constitute the determination of breakdown flows and which influence the position and the spread of the capacity probability function ($P_c(Q)$).

Time Aggregation Period

The minimum aggregation level that can be obtained from loop detectors in the Netherlands is 1 minute. As such, a time period must be chosen that is higher than or equal to this value. In the determination of the time interval that is used, a trade-off exists between Resolution and Stability. Resolution is important because one wants to identify the moment at which breakdown occurs as accurately as possible for identifying lane flow and truck traffic conditions in the data. Stability is important because a capacity flow has to be sustained for at least some amount of time - a short burst of high flow, after which a breakdown immediately occurs, is not a good representation of the actual value of capacity.

When using time interval windows of 1 minute, the resolution is very high because the moment of breakdown can be very accurately identified. However, because traffic patterns are generally very dynamic, short periods of high flows are often alternated with short periods of lower flows which causes 1-minute flow measurements to be fairly unstable and exhibit a large variety of values. Furthermore, sudden drops below the critical speed may also occur from time to time without actually leading to severe congestion and may therefore not be representative of the actual capacity.

When applying longer time interval windows of, for instance, 5 minutes, stability is positively affected because the variation that is experienced on a minute-to-minute basis is averaged out, leading to more stable flow measurements. On the other hand, however, resolution is negatively affected because the breakdown may occur anywhere within those five minutes and, as such, it becomes more likely that a 5-minute measurement window containing the breakdown will average the data of both the free flow capacity state before the breakdown as well as data of the congested state after the breakdown.

In order to solve this trade-off between resolution and stability, it has been chosen to use a 5-minute rolling window which is calculated for each minute by summing the flows of the past five minutes (see Equation 3.3) and taking a harmonic average of the speeds (for an explanation see Appendix A) of those past five minutes (see Equation 3.4), which is very similar to the approach taken by Calvert (2016).

$$q_t^{5min.} = q_t + q_{t-1} + q_{t-2} + q_{t-3} + q_{t-4} \tag{3.3}$$

$$u_t^{5min.} = \frac{q_t^{5min.}}{(\frac{q_t}{u_t}) + (\frac{q_{t-1}}{u_{t-1}}) + (\frac{q_{t-2}}{u_{t-2}}) + (\frac{q_{t-3}}{u_{t-3}}) + (\frac{q_{t-4}}{u_{t-4}})} \tag{3.4}$$

In this way it is possible to maintain a resolution of 1 minute by updating the values for each minute whilst ensuring stability in the measurements by using a moving average over a period of five minutes.

Critical Speed

The critical speed is the boundary between the Fluent traffic state F and the Congested traffic state C. The choice of the critical speed U^* has strong implications for the determination of the capacity value in the Product Limit Method and will affect the capacity distribution. For higher critical speeds, a larger fraction of the capacity distribution will be estimated, as the number of observations in category F decreases, which will increase the probability that a category B measurement will be the highest value that is observed.

In this thesis it has been chosen to set the critical speed at 85 km/h. There are two reasons for doing this:

Firstly, when assessing the fundamental diagrams that have been plotted in Appendix B for the locations under study, it can be seen that 85 km/h corresponds relatively well to the values found at the top of the free flow branch and that the category B measurements generally seem to be where one would expect them to be in the fundamental diagram.

Secondly, the freeway speed limit for trucks in The Netherlands is 80 km/h but many trucks drive somewhere between 80 and 90 km/h, because there is some margin before getting a fine. It is therefore expected that, once the 5-minute average speed of the roadway drops below 85 km/h, truck drivers (the slowest road users) are most certainly driving at a speed below their preference speed and that, for this reason, congestion has set in.

One may note that setting the critical speed at a value of 85 km/h may be relatively high, when lower values are suggested in literature (Brilon *et al.*, 2005) (Geistefeldt, 2011) (Calvert, 2016). Closer examination of the data reveals, however, that speeds of both lanes are are well below 85 km/h at the moment when the average 5 minute speed drops below 85 km/h. Generally, values of 1-minute averaged lane speeds are between 30 and 75 km/h at the moment of breakdown, thus validating the fact that a critical speed of 85 km/h can be considered to be a suitable threshold for the data at hand.

The Empirical Distribution Method as a Case of the PLM

Though theoretically more correct, the PLM poses the problem that parametric testing is made impossible by the fact that capacity distributions are incomplete. One solution is to estimate the corresponding capacity distribution function by means of a Maximum Likelihood method. This approach, however, often needs extrapolation when the distribution has only been estimated to a minor extent. Consequently, there is no certainty about how reliable inferred capacity distribution functions are.

Another method for enabling parametric testing is also possible by "weakening" the restrictions imposed on the distribution estimation process by the PLM method. Instead of weighing the set of breakdown flow measurements B against measurements from sets B and F. One could also opt for generating a full capacity distribution by only using measurements from set B for the determination of the distribution. This is also known as the Empirical Distribution Method (Minderhoud *et al.*, 1996, p.33) and it can be considered to be a special case of the Product Limit Method method.

Capacity distributions that are obtained through this method will always be complete, thus allowing parametric testing. Additionally, regression theory can be successfully applied to these kind of data. Because category F flow measurements that are potentially higher are not taken into account, it is generally the case that a distribution obtained with the Empirical Distribution Method will have a lower median value than a distribution obtained by means of the Product Limit Method. Additionally, relatively low flow measurements of category B will obtain more weight in the distribution, as there are a lesser number of high flow values against which they will be compared in set B, than in the joint set of categories

B and F.

Notwithstanding these shortcomings of the Empirical Distribution Method, it is expected that when applying this method consistently across different speed limit periods for a given set of locations, that the distributions will still be comparable, as long as the sample sizes are sufficiently large.

3.2.2. Regression Theory

Regression theory will primarily be used in chapter 5 for the purpose of analyzing changes in breakdown flows under different speed limits. The following section will elaborate on basic regression theory as well as the underlying assumptions of the model, which are important for regression diagnostics.

Ordinary Least Squares Regression

OLS regression will be applied to control for the effect of, respectively, changes in truck traffic levels and lane flow fractions when performing assessments of the change in breakdown flows as a result of a change in the speed limit, in subsequent sections of chapter 5.

The primary (single-variate) function of the OLS regression model is (Stock and Watson, 2015):

$$Y_i = \beta_0 + \beta_1 * X_i + u_i \tag{3.5}$$

Where Y_i is the dependent variable, β_0 is the intercept of the regression line, β_1 is the slope of the regression line, X_i is the independent variable (of interest) and u_i is the error term. In determining the regression line, the model looks for parameter values of β_0 and β_1 for which the (squared) distance between the line and all measurement points is minimized (Stock and Watson, 2015). This can be done by means of minimization of the sum of squared errors:

$$min \sum_{i=1}^{n} (Y_i - \hat{Y}_i)^2 = min \sum_{i=1}^{n} (Y_i - \hat{\beta}_0 - \hat{\beta}_1 * X_i)^2 \tag{3.6}$$

Taking the derivatives of Equation 3.6 with respect to β_0 and β_1 and setting these derivatives equal to zero, yields the following functions for each respective variable (Stock and Watson, 2015):

$$\hat{\beta}_0 = \bar{Y} - \hat{\beta}_1 * \bar{X} \tag{3.7}$$

$$\hat{\beta}_1 = \frac{s_{xy}}{s_{x^2}} = \frac{\sum_{i=1}^{n} (X_i - \bar{X})(Y_i - \bar{Y})}{\sum_{i=1}^{n} (X_i - \bar{X})(X_i - \bar{X})} \tag{3.8}$$

Where variables with a "hat", such as $\hat{\beta}_0$, are estimates of a variable and where variables with a "bar", such as $\bar{X}$ are (sample) averages of a variable. Multivariate regression functions are also possible. Parameters are estimated as in Equation 3.8 and the functional form becomes:

$$Y_i = \beta_0 + \sum_{i=1}^{K}(\beta_k * X_{k,i}) + u_i \tag{3.9}$$

Lastly, for each regression an R^2 value is presented, which represents the amount of total variation in the data ($\sum_{i=1}^{N}(Y_i - \bar{Y}_i)$) that is explained by the model ($\sum_{i=1}^{N}(Y_i - \hat{Y}_i)$) (Stock and Watson, 2015):

$$R^2 = \frac{\sum_{i=1}^{N}(Y_i - \hat{Y}_i)}{\sum_{i=1}^{N}(Y_i - \bar{Y}_i)} \tag{3.10}$$

High values of R^2 indicate that the regression line found by estimating the parameters in Equation 3.7 and Equation 3.8 was found to be closely resembling the data (good fit) while low values of R^2 indicate that no strong relation has been found (Stock and Watson, 2015).

Fixed Effects Regression

In this thesis, breakdown flow data are gathered over multiple locations and multiple periods, thus exhibiting a panel data structure.

Suppose that the (true) population function is the following:

$$Y_{i,t} = \beta_0 + \beta_1 * X_{i,t} + Z_i + W_t + u_{i,t} \tag{3.11}$$

Where Z_i is the effect of (time-invariant) location specific characteristics and where W_t is the effect of (location-invariant) time specific characteristics.

By making sure that no other significant changes than the speed limit change have occurred to locations under study between different study periods, it is expected that W_t is implicitly accounted for - as such, $COV(w_t, X_{i,t})$ is expected to be equal to zero.

Location specific factors, however, have not been accounted for. Consequently, performing a direct cross-sectional analysis will absorb not only the effects from a speed limit change, but also the effects from the characteristics of the different locations under study. This leads to biased estimators as is shown by equations 3.12 to 3.14 (Stock and Watson, 2015).

$$\hat{\beta_1} = \frac{COV(Y_{i,t}, X_{i,t})}{VAR(X_{i,t}))} = \frac{COV(\beta_0 + \beta_1 * X_{i,t} + Z_i + W_t + u_{i,t}, X_{i,t})}{VAR(X_{i,t})} \tag{3.12}$$

$$\hat{\beta_1} = \frac{0 + \beta_1 * COV(X_{i,t}, X_{i,t}) + COV(Z_i, X_{i,t}) + 0 + 0}{VAR(X_{i,t})} \tag{3.13}$$

$$\hat{\beta_1} = \beta_1 + \frac{COV(Z_i, X_{i,t})}{VAR(X_{i,t})} \neq \beta_1 \tag{3.14}$$

To correct for the bias induced by location specific factors, Fixed Effects regression will be performed by adding $M-1$ location dummies to the regression function as shown in Equation 3.15:

$$\hat{Y}_{i,t} = \hat{\beta}_0 + \sum_{k=1}^{K}(\hat{\beta}_k * X_{k,i,t}) + \sum_{m=1}^{M-1}(\hat{\gamma}_m * D_{m,i}) \tag{3.15}$$

The coefficients $\hat{\gamma}_m$ should be interpreted as the value by which the intercept of the regression function is shifted from one location to the next.

7 Assumptions of Least Squares Regression Theory

In order to obtain unbiased and consistent estimators in Least Squares Regression, it is important that seven conditions are met (Stock and Watson, 2015):

1. The regression model has to be linear in its coefficients, as well as the error term
2. The (estimated) error ($u_{i,t}$) has to have a mean of zero ($\mu_u = 0$)
3. The (estimated) error ($u_{i,t}$) has to be unrelated to the independent variable of interest ($X_{1,i}$) - i.e. $COV(u_{i,t}, X_{1,i,t}) = 0$
4. Observations of the error term are uncorrelated with each other ($COV(u_i, u_j) = 0$), where $i \neq j$
5. The variance of the error term has to be constant for all observations ($VAR(u_{i,t}|X_{i,t}) = \sigma$) - i.e. no heteroscedasticity should be present.
6. No multicollinearity is present, which means that no combination of independent variables may be highly correlated to each other, as it makes the estimators much less precise.
7. The error term (u_i, t) has to be normally distributed.

Condition one is important because, if the (true) population model is non-linear in its coefficients, the regression model will pick up incorrect trends in the data. Satisfaction of this condition will be determined by reasoning on the basis of theory about what independent variables should be included in the analysis.

Condition two is important because a non-zero mean of the error term will induce a structural bias in the regression estimators (Stock and Watson, 2015), which should be avoided for a purposeful analysis. This condition will be checked by performing a means comparison T-test on the error term.

Condition three is important because, when the error is correlated to the independent variable of interest, the estimator of that variable will be biased. Given that:

$$\hat{\beta}_1 = \frac{COV(Y_{i,t}, X_{i,t})}{VAR(X_{i,t})} = \frac{COV(\beta_0, X_{i,t}) + \beta_1 * COV(X_{i,t}, X_{i,t}) + COV(u_{i,t}, X_{i,t})}{VAR(X_{i,t})} \tag{3.16}$$

It can be seen that when $COV(u_{i,t}, X_{i,t} \neq 0)$:

$$\hat{\beta_1} = 0 + \beta_1 + \frac{COV(u_{i,t}, X_{i,t})}{VAR(X_{i,t})} \neq \beta_1 \tag{3.17}$$

Whether the error is unrelated to the independent variable of interest, will be assessed by regressing for each regression the independent variable (X) on the error term (u), in accordance with the following regression function:

$$u_{i,t} = \delta_0 + \delta_1 * X_{i,t} \tag{3.18}$$

When δ_1 is found to be insignificant in this regression, it is considered to be proven that u is unrelated to X and that the third condition is satisfied.

The fourth condition is considered to be important because, when auto-correlation is present, it will affect the standard error of the parameter estimates, leading to inflated and biased estimates (Stock and Watson, 2015). Though originally taken from a time-series of speed and flow data, breakdown flow data are generally separated by sufficiently large time gaps, that they can reasonably be assumed to be independent from each other over time. As such, the fourth condition is assumed to be satisfied for all regression analyses in this thesis.

The fifth condition is important because the presence of heteroscedasticity will cause the standard error of an estimator to be lower than it truly is, thus leading to smaller p-values and a larger probability of falsely rejecting the null-hypothesis in favor of the alternative hypothesis (Type I error). Whether the regression model contains heteroscedasticity will be checked by means of the Breusch-Pagan test, which has a test statistic that follows a χ^2 distribution. If the test is deemed to be insignificant, it can be assumed that the data is homogeneous. Conversely, when a robust estimation procedure has been applied, it can also be taken that the estimates have taken into account the heteroscedasticity of the data set and that the condition is also satisfied.

The sixth condition is important because when two independent variables are highly related, it will obscure their individual effect and thus lead to insignificant estimates. Moreover, the sign of an estimate may even be reversed, as a consequence of the high degree of correlation between two or more variables. Perfect correlations - on the order of 1.00 - are impossible to include in the model, as it restricts the regression model in fitting any of the parameters. It is also for this reason that the number of location dummies should be $M - 1$ so that when all location dummies are equal to zero, the effect is given for the location without a dummy (Stock and Watson, 2015).

The seventh, and last, condition is deemed to be relevant, but not necessary. A normally distributed error term implies that confidence intervals are correctly defined. Without a normally distributed error term, the estimates are still unbiased, but one can be less certain about the correctness of the confidence interval. Nonetheless, with error terms that are approximately normally distributed and sufficiently high significance of estimators, it can be assumed that estimators are likely to be correctly inferred. Whether error terms are distributed normally, will be assessed by means of the Shapiro-Francia test which has been preferred over the Shapiro-Wilk test, because it puts less restrictions on sample size, and which has a test statistic that follows a Z distribution. When the test is insignificant, it implies that the error term is normally distributed.

3.2.3. T-test for equality of the means

The t-test for equality of the means will be applied for the assessment of changes in truck traffic levels, as examined in section 4.4, as well as for the comparison of mean speeds under different speed limits, as examined in Appendix C.

In the t-test for equality of the means, two means from two different samples are compared with each other to assess whether they are significantly different. The formula for the test statistic is (Keller, 2009):

$$t = \frac{(\bar{x_1} + \bar{x_2}) - (\mu_1^{H_0} - \mu_2^{H_0})}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \sim t[df] \tag{3.19}$$

With (Keller, 2009):

$$df = \frac{(\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2})^2}{\frac{(\frac{s_1^2}{n_1})^2}{(n_1-1)} + \frac{(\frac{s_2^2}{n_2})^2}{(n_2-1)}} \tag{3.20}$$

Where t is the value of the test statistic, which follows a t-distribution, distributed with df degrees of freedom. Where $\bar{x_1}$ and $\bar{x_2}$ are the means of the first and second sample respectively. Where $(\mu_1^{H_0} - \mu_2^{H_0})$ is the hypothesized difference between the means, which in all cases in this thesis is zero $(\mu_1^{H_0} - \mu_2^{H_0} = 0)$. Where s_1^2 and s_2^2 are the sample standard deviations and n_1 and n_2 are the sample sizes of sample 1 and 2 respectively.

3.2.4. Levene's F-test for equality of variances

The F-test for equality of variances will be performed to assess the degree to which the variance of the speed distribution in Appendix C has changed. The formula for the F-test statistic is (Keller, 2009):

$$F = \frac{s_1^2}{s_2^2} \sim F[df_1; df_2] \tag{3.21}$$

With (Keller, 2009):

$$df_i = n_i - 1 \tag{3.22}$$

Where F is the value of the test statistic, which follows a F-distribution, distributed with df degrees of freedom. Where s_1^2 and s_2^2 are the sample variances of samples 1 and 2 respectively and where df_i is the number of degrees of freedom for sample i.

3.2.5. Z-test for the comparison of two proportions

The Z-test for the comparison of two proportions will be performed to assess the degree to which the the flow fraction distribution has changed between the two different measurement periods in chapter 5. The formula for the test statistic is (Keller, 2009):

$$Z = \frac{\hat{p_1} - \hat{p_2}}{\sqrt{\hat{p}(1-\hat{p})(\frac{1}{n_1} + \frac{1}{n_2})}} \sim N(\mu = 0, \sigma = 1) \tag{3.23}$$

Where $\hat{p_1}$ and $\hat{p_2}$ are the sample proportions and where n_1 and n_2 are the sample sizes of sample 1 and 2 respectively. Where $\hat{p}$ is (Keller, 2009):

$$\hat{p} = \frac{n_1\hat{p_1} + n_2\hat{p_2}}{n_1 + n_2} \tag{3.24}$$

3.2.6. Willcoxon Signed Rank Sum Test for Matched Pairs

The Willcoxon Signed Rank Sum Test for Matched Pairs will be applied for the pair-wise comparison of percentiles of the breakdown distribution. For each breakdown measurement of category B, a percentual value has been determined by means of the Product Limit Method. By means of interpolation, breakdown flow values can be determined at each available percentile and different flow values experienced at a given percentile such as, for instance, the 5th percentile, can be directly compared to determine whether a significant difference exists.

For the Willcoxon test that is applied in this thesis, each available percentile of the "after" sample i_a will be compared with the same percentile of the "before" sample i_b, as long as they can both be obtained by means of interpolation.

Subsequently, absolute differences between the breakdown flow value of percentile i_a of the "after" sample and percentile i_b of the "before" sample are calculated, after which these absolute differences are ranked from low to high. After this, each ranking is multiplied by either $+1$ when the difference was positive and -1 when the difference was negative. By summing the ranks of the positive differences a positive rank sum T^+ is obtained and by summing the ranks of the negative differences a negative rank sum T^- is obtained.

Either one of these rank sums can be used to perform the test, which will produce the same result. By default, however, the positive rank sum T^+ is used. For small samples ($N < 30$) T^+ is compared against the critical bounds of a significance table as presented in Appendix D, while for large samples ($N \geq 30$) the following Z-test is performed (Keller, 2009):

$$Z = \frac{T^+ - \frac{n(n+1)}{4}}{\sqrt{\frac{n(n+1)(2n+1)}{24}}} \sim N(\mu = 0; \sigma = 1) \tag{3.25}$$

Where Z is the value of the test statistic and where n is the sample size, in this case the number of percentiles that are compared. One could argue that the choice for the comparison at percentiles of the distribution is arbitrary and may, for this reason, be unreliable. Why not take tenths or even hundreds of percentiles, one might think.

It is true that the choice for percentiles is arbitrary, but it is expected that this is unlikely to affect the potential significance of the results for the determination of the significance of differences between breakdown flow probability curves. If one would take more measurement points by, for instance, measuring at tenths or even hundreds of percentiles, it is certain that this will increase the positive rank sum T^+, but it will also increase the sample size n and, as can be verified by the reader, this will decrease the value of the Z-test statistic to such an extent (see Equation 3.26) , that significance will be harder to prove as a consequence of the larger sample size, thus necessitating a larger positive rank sum T^+.

$$\frac{\partial Z}{\partial n} = -\frac{\sqrt{6}\left(n\left(n+1\right)\left(2n^2+2n+24T^++1\right)+4T^+\right)}{4\left(n\left(n+1\right)\left(2n+1\right)\right)^{\frac{3}{2}}} < 0 \tag{3.26}$$

As such, it is expected that the Willcoxon Signed Rank Sum Test for matched pairs will produce a reliable indication for the difference between "before" and "after" distributions.

3.3. Data

3.3.1. Available Data

Vehicle counts and average (time mean) speed data is available for a large number of freeway locations in The Netherlands. Data are obtained via loop detectors in the road surface that count vehicle passages and measure vehicle speeds. These counts and speeds are subsequently aggregated to 1-minute counts and 1-minute (time) mean speeds, which can be obtained from the database. From the vehicle counts, one is able to calculate hourly flows, and from the 1-minute time mean speeds one is able to obtain an approximation of the space mean speed by using harmonic averages for all lanes on a given roadway.

3.3.2. Measurement Locations

In this section an overview of measurement locations is presented. In Figure 3.2 a layout of a two lane on-ramp is presented with distance A representing the length of the merging lane, distance B representing the distance from the gore to the upstream detector and distance C representing the distance from the gore to the downstream detector. In Table 3.1 the distances for each location are given, as well as in which sample they are included. Most of the time, loop detectors are placed somewhere before and after a merging area. Generally, in on-ramp areas, there are only loop detectors on the lanes of the main road (passing lane and shoulder lane) and no loop detectors on the on-ramp or merging lane. Because, in most of the cases, no data from the on-ramp is available, detector locations have been chosen that are either near the end of the merging area or just after the merging area.

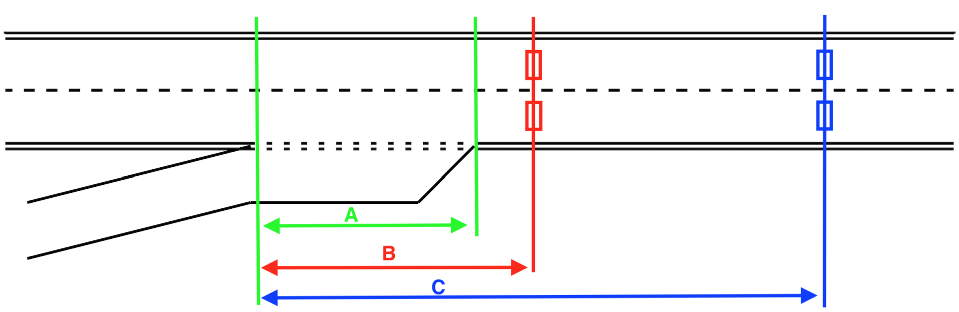

Figure 3.2. – Schematic layout of on-ramp locations

Location	Distance A	Distance B	Distance C	8-location sample	17-location sample
A2-L Valkenswaard	220	220	1330	Yes	Yes
A2-R Valkenswaard	370	240	1470	Yes	Yes
A15-L Sliedrecht-Oost	320	510	1440	No	Yes
A15-R Hardingxveld	320	350	1230	No	Yes
A15-R Sliedrecht-Oost	430	350	1270	No	Yes
A15-R Sliedrecht-West	330	420	1540	No	Yes
A20-R Nieuwerkerk	330	400	1200	No	Yes
A27-L Lexmond	310	310	1360	Yes	Yes
A27-R Lexmond	340	340	1380	Yes	Yes
A27-R Oosterhout	320	260	1410	No	Yes
A58-L Bavel	340	190	880	Yes	No
A58-L Moergestel	320	130	1320	Yes	Yes
A58-R St. Annabosch	350	350	1510	Yes	Yes
A58-R Gilze	290	210	1390	No	Yes
A58-R Goirle	N.A.	N.A.	N.A.	Yes	Yes
A58-R Oirschot	330	240	1640	No	Yes
A58-R Ulvenhout	350	290	1390	No	Yes
A67-R Leenderheide	300	310	1335	No	Yes

Table 3.1. – Positioning of detectors at different locations (distances in meters)

It is expected that this will not be a problem for capacity estimation, because drivers are likely to accept short headways at the bottleneck for some amount of time before "relaxing" into a longer headway (Schakel *et al.*, 2012), during which they move downstream with the flow. As a consequence, turbulence is more likely to occur 300 to 900 meters downstream of the gore of the on-ramp or bottleneck than at the merger location itself (Beinum, 2018) and jam formation may even occur more than a kilometer downstream (Cassidy and Bertini, 1999), increasing the probability that the $F \to C$ transition will happen in the area between the up- and downstream detectors, since all upstream detectors are within a distance of 420 meters from the gore.

Additionally, if some traffic jams that occur upstream of the detector are missed, this will not invalidate the capacity analysis. Though it should be prevented to the best extent possible by putting the upstream detector as close to the bottleneck as possible, because missing observations are considered to be a missed opportunity. Moreover, if the downstream detector is properly chosen, the upstream detector will only capture traffic jams that occurred in the bottleneck region.

For some locations the detector was chosen in the on-ramp area, because other detectors were too far downstream (this is true for every location where distance A exceeds distance B, see Table 3.1). Because only data is available from loops in the shoulder- and passing lane, there is a risk of missing vehicles on the merging lane. This risk is deemed to be acceptable, since most vehicles merge into the shoulder lane quite early when traffic is still in the free flow state. Nonetheless, if observations in the merging lane are missed from time to time, this does not pose a problem for the before- and after comparison in the analysis and neither for the regression analysis (when location fixed effects are included) as it can be expected that, on average, a similar number of merging lane vehicles will be missed for each sample.

Lastly, three locations require a special remark. Firstly, the A58-R Goirle location is not a two lane on-ramp but a three-to-two merging area. A schematic overview of this location is presented in Figure 3.3. Additionally, the locations A2-R Valkenswaard en A58-R Oirschot, have a three-to-two merging area at just a few hundred meters before the two lane on-ramp area. The benefit of this (at least for the purpose of this thesis) is that these locations are very sensitive to congestion formation, as the stream is first compressed by the merging area, after which the on-ramp flow easily triggers congestion in the on-ramp area. There is however also a slight risk that, if the traffic jam originates in the three-to-two merging area, the breakdown flow that is measured is actually the outflow of the jam, rather than the free flow capacity. It is expected that this is unlikely to strongly influence the results, as location specific effects are, generally, accounted for in this study and also because low breakdown flows obtain less weight under the distribution estimation process of the Product Limit Method.

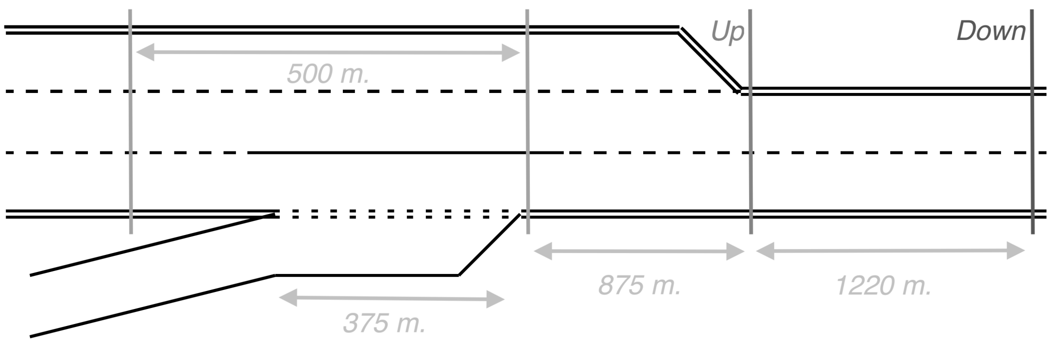

Figure 3.3. – Layout of location goirle on the A58-R

3.3.3. Measurement Periods

Two samples have been obtained for different analyses. The "eight-location" sample (see Table 3.2) has been obtained for comparing the 120 km/h limit with the 130 km/h limit, which has been done at a time when insufficient measurements for the 100 km/h limit were available. The data from this method have been used as the sample for the application of the product limit method in chapter 4. Additionally, these data have been used in the regression analyses of section 5.1 and section 5.3 together with truck traffic data from these periods. Measurements from different locations in a sample are obtained for the same period of the year to make sure that meteorological conditions and daylight intensities are approximately the same. Times of measurement were each day of the week from 06:00 until 19:00 so that only day-time is taken into account.

The second sample (see Table 3.3) has been obtained at a time when more measurements for the 100 km/h limit were available. Though it should be noted that these data were obtained during the COVID-19 lock-down period and may, for this reason, not be truly rep-

Location	Limit Change	Measurement period	120 km/h	130 km/h
A2L Valkenswaard	21-12-2018	01-March to 31-May	2018	2019
A2R Valkenswaard	21-12-2018	01-March to 31-May	2018	2019
A27L Lexmond	05-02-2016	01-March to 31-May	2015	2016
A27R Lexmond	05-02-2016	01-March to 31-May	2015	2016
A58L Bavel	05-02-2016	01-March to 31-May	2015	2016
A58L Moergestel	05-02-2016	01-March to 31-May	2015	2016
A58R St. Annabosch	05-02-2016	01-March to 31-May	2012	2016
A58R Goirle	01-09-2012	01-March to 31-May	2012	2013

Table 3.2. – Date of speed limit change (120 to 130 km/h) and measurement periods for the eight-location sample

Location	Measurement period	120 km/h	130 km/h	100 km/h
A2L Valkenswaard	01-April to 31-July	2018	2019	2020
A2R Valkenswaard	01-April to 31-July	2018	2019	2020
A15L Sliedrecht-Oost	01-April to 31-July	2019		2020
A15R Hardinxveld	01-April to 31-July	2019		2020
A15R Sliedrecht-Oost	01-April to 31-July	2019		2020
A15R Sliedrecht-West	01-April to 31-July	2019		2020
A20R Nieuwerkerk	01-April to 31-July	2019		2020
A27L Lexmond	01-April to 31-July	2015	2019	2020
A27R Lexmond	01-April to 31-July	2015	2019	2020
A27R Oosterhout	01-April to 31-July	2015	2019	2020
A58R St. Annabosch	01-April to 31-July	2012	2019	2020
A58R Gilze	01-April to 31-July	2012	2019	2020
A58R Goirle	01-April to 31-July	2012	2019	2020
A58R Moergestel	01-April to 31-July	2015	2019	2020
A58R Oirschot	01-April to 31-July	2015	2019	2020
A58R Ulvenhout	01-April to 31-July	2015	2019	2020
A67R Leenderheide	01-April to 31-July	2019		2020

Table 3.3. – Measurement periods for the seventeen-location sample

resentative of capacity values under "non-COVID" conditions, as different driving behavior may have been present. This "seventeen-location" sample has been used primarily for the regression analyses in section 5.2 and section 5.3. Again, measurements from different locations in a sample are obtained for the same period of the year to make sure that meteorological conditions and daylight intensities are approximately the same. Times of measurement were each day of the week from 06:00 until 19:00 so that only day-time is taken into account.

3.3.4. Assessment of mean driving speeds under different limits

It has been found that mean speed choice behavior across different locations seems to be fairly consistent for a given limit. For a limit of 100 km/h the mean speed was around 115 km/h in the passing lane and about 105 km/h in the shoulder lane, for a limit of 120 km/h the mean speed was around 128 km/h in the passing lane and about 112 km/h in the shoulder lane and for a limit of 130 km/h the mean speed was around 130 km/h in the passing lane and 115 km/h in the shoulder lane. Similarly, the distributions for a given limit had very similar shapes across locations as there was surprisingly little variation between the percentiles of the distribution across different locations for a given limit (for an example see Figure 3.4.

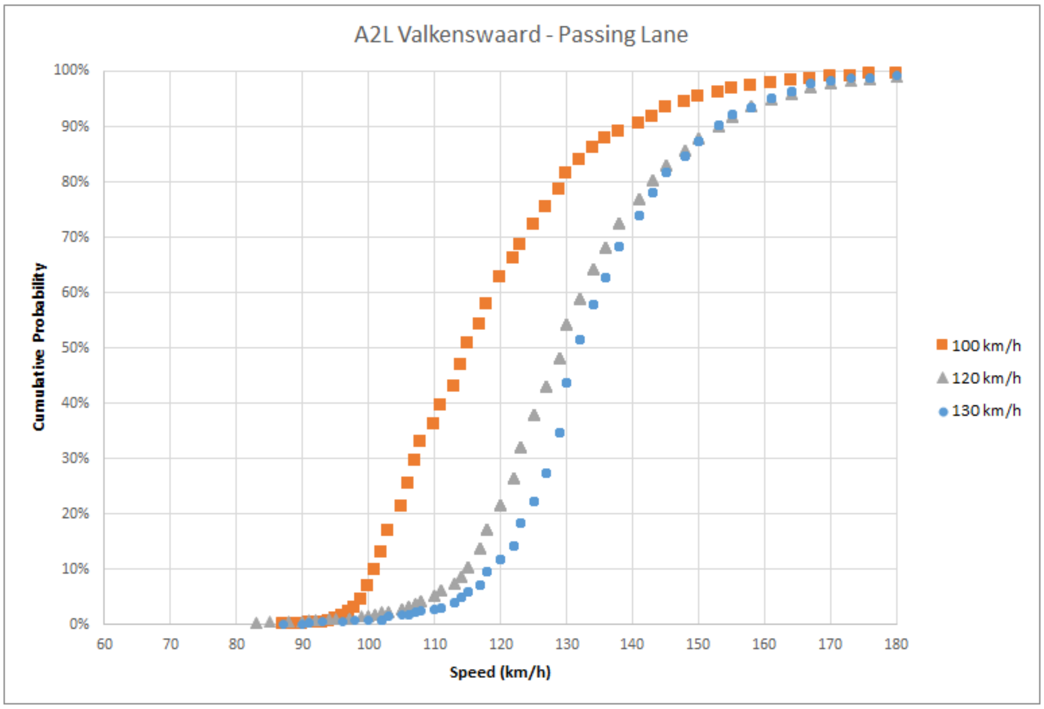

Figure 3.4. – Speed distributions derived for the passing lane of location A2-L Valkenswaard (example)

T-tests were performed to measure whether the speed limit change had a significant effect on the mean driving speed. In both lanes the mean speed under a 100 km/h limit was significantly lower (at the 1% level) than the mean speeds under 120 km/h and 130 km/h (see Appendix C). Additionally, for the passing lane, the mean speed was also deemed to be significantly higher under the 130 km/h limit with respect to the 120 km/h limit, though it is only a small increase of approximately 2 km/h, while the 100 km/h limit exhibits a decrease in the range of 10 to 15 km/h. For the shoulder lane, the mean speed under the 130 km/h limit was only found to be significantly higher than the 120 km/h limit in a small number of cases.

Additionally, an F-test was performed on both lanes to estimate whether the degree of speed variance increased as a result of the change in the legal limit. It was found for the passing lane that the variance of the speed distribution increases with a decrease in the legal limit for most locations. For the shoulder lane only a few locations experienced a change in variance as a consequence of a change in the limit.

Based on the results in Appendix C it can be concluded that changes in speed choice behavior across different locations is quite consistent and that similar speed choice behavior can be expected for a given speed limit at the different locations in this study. Furthermore, it is important to note that speed choice behavior under the 120 km/h limit is only slightly different from the 130 km/h limit, while speed choice behavior under the 100 km/h limit is very different. More details regarding this analysis can be found in Appendix C.

3.4. Summary

In this chapter several sub-questions have been posed to help provide an answer to the general research question:

To what extent does the height of the speed limit affect freeway capacity?

The first sub-question stated: "Given a change in the speed limit, can a change in the capacity distribution be observed at a given location?" To find an answer to this question, hypotheses 1a, 1b and 1c will be tested in sections 4.1, 4.2 and section 4.3 respectively, to investigate whether significant changes to the breakdown flow distribution have occurred.

The second sub-question stated: "Given a change in the capacity distribution between measurement periods, can this change also be related to changes in the traffic composition at a location?" Through analysis of average truck traffic data over each study period, hypothesis 2 will be tested in section 4.4, through which it will be investigated whether the capacity distribution may have changed as a consequence of a changed traffic composition.

The third sub-question stated: "Given a change in the capacity distribution between measurement periods, are significant changes in lane choice behavior visible?". From literature it is known that changes in speed limits can be related to changes in lane utilization rates. Given that there must be an "optimal" lane flow distribution to maximize capacity, it can be expected that changes in lane choice behavior may be relevant for the determination of capacity. As such, by means of evaluation of hypothesis 3 in section 4.5, it will be tested whether significant changes have occurred.

The fourth sub-question stated: "When controlling for other relevant variables and location specific factors, can a general change in breakdown flows be attributed to a change in the legal limit?" In section 5.1 and 5.2 this question will be answered by testing hypotheses 4a and 4b, to determine whether the height of the speed limit is significantly related to the breakdown flow, whilst taking account of other relevant variables and location specific factors.

The fifth and last sub-question stated: "Is there a significant relationship between the speed limit and the utilization rate of the passing lane?" It is proposed that the speed limit may influence the breakdown flow distribution, by inducing changes in lane choice behavior, such that the lane flow distribution is altered. In section 5.3 hypothesis 5 will be tested to assess whether a relation exists.

Overview of Methods

In the remainder of the chapter, an overview was given of methods that have been applied in this thesis to assess the hypotheses that were mentioned in the previous paragraphs. The most important method that is used in this thesis is the Product Limit Method. The measurement categorization process has been explained in this chapter, as well as the function that is used for generating the breakdown probability functions. It was discussed why the PLM has a tendency for producing incomplete capacity distribution functions and how this is affected by the choice for the critical speed U^*. It was explained that a five-minute, harmonic moving average of speeds will be applied, as well as a sum of the flows for the past five minutes, to balance the high resolution of 1-minute measurements with the stability of a 5 minute aggregation interval. Also, a motivation was provided for the choice of a critical speed of 85 km/h, which is further supported by the graphs presented in Appendix B.

Additionally, an overview has been given of basic regression theory, which will be applied in the analyses in chapter 5. It was discussed that Fixed Effects regression will be applied in this thesis, so that differences between locations can be explicitly accounted for. Moreover, an over view of general conditions for the realisation of unbiased and efficient estimators was presented (7 assumptions of OLS), which will be used to perform regression diagnostics in chapter 5. Lastly, several statistical tests and methods that will be used throughout this thesis have been presented, such as the T-test for the equality of means, The F-test for the equality of variances, the Z-test for the comparison of proportions and the Willcoxon signed rank sum test for the comparison of matched pairs.

Data Description

Furthermore, a description of the data was presented. It was discussed that 1-minute data is obtained from loop detectors in the roadway, which are generally only installed on the passing and shoulder lane and not on the merging lane. As such, it has been chosen to measure the flow at detectors just downstream of the merging point, so that the complete inflow into the bottleneck can be measured. It was explained that, due to a delayed relaxation effect, the turbulence causing traffic breakdown is expected to occur 300 to 900 meters downstream of the gore. Given that the up-stream detectors at on-ramps are, in all cases, placed closer to the gore than 420 meters, it is expected that only few breakdowns will be missed by the detectors. To illustrate the location of detectors, a layout with a corresponding table with distances was provided for all locations in this study, as well as a table in which the measurement periods for all samples were presented.

Lastly, an investigation was performed for the driving speeds that were observed under the different speed limits of 100, 120 and 130 km/h. In this analysis it was found that mean speeds were much lower under the 100 km/h limit (10 to 15 km/h lower) than under the

130 km/h limit, while between the 120 and 130 km/h limit only a difference of about 2 to 3 km/h was present. In addition to this, it was found by means of a t-test that most mean speeds were significantly different for all speed limits and it was found by means of a F-test that the variance in speeds under the 100 km/h limit was higher than under the 120 km/h, which was subsequently higher than under the 130 km/h limit.

4. Results of the application of the Product Limit Method

In this chapter, results have been tabulated for the "breakdown flow" ($Q^{F \to C}$) percentiles that were found by plotting the breakdown flow probability functions through the application of the Product Limit Method. An example of such breakdown flow probability plots, can be seen in Figure 4.1. In contrast to the Empirical Capacity Distribution method, the PLM tends to produce incomplete capacity distributions (see Figure 4.1), because it takes into account both breakdown and free-flow measurements, as was discussed in subsection 3.2.1. In the remainder of this chapter, all inferences made will be about the eight-location sample and an analysis will be performed on how the breakdown flow probability distributions have shifted from the 120 km/h to the 130 km/h limit.

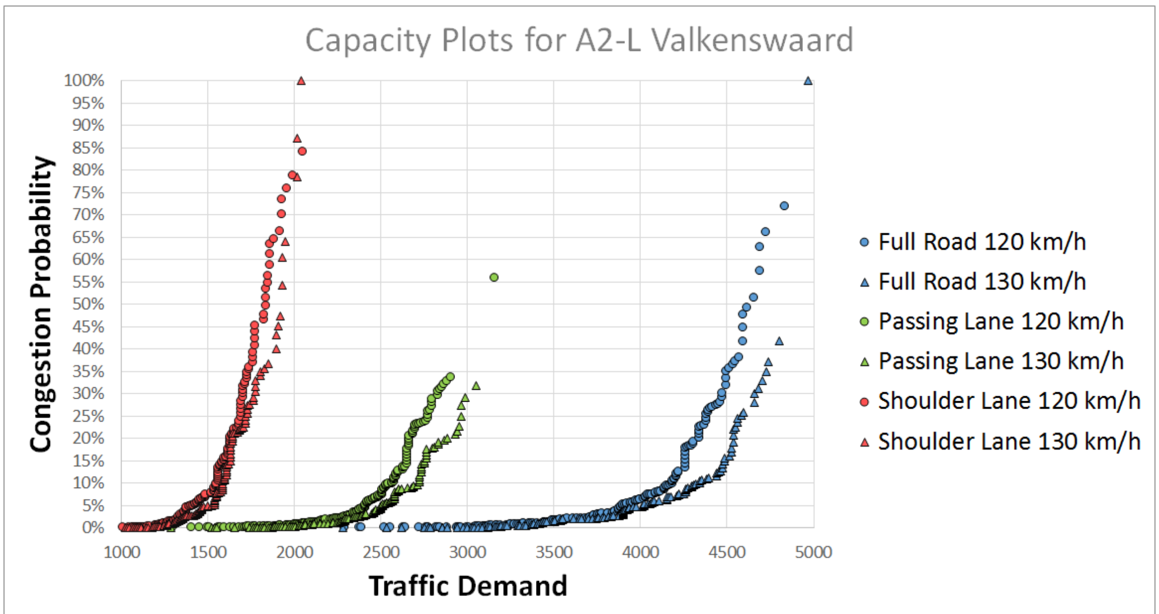

Figure 4.1. – Capacity distribution plots for the 120 km/h and 130 km/h limit at on-ramp location A2-L Valkenswaard

In sections 4.1, 4.2 and 4.3, the results for differences between the capacity/breakdown flow distributions under the 120 km/h and 130 km/h limit will be discussed for the complete roadway, passing lane and shoulder lane respectively. After this, truck traffic data will be analysed to investigate whether changes in mean truck traffic levels could have an effect on the results which obscure the effects from the speed limit change. Additionally, an analysis is performed on how the lane flow distribution is affected by different speed limits and what this says about underlying lane choice behavior. Lastly, it will be discussed in what respects the dynamics in the shoulder lane and the passing lane differ from each other, which will give some further insight into how the height of the breakdown flow is influenced by these dynamics.

4.1. Complete Roadway

In this section, hypothesis 1a will be tested, which states:

- H_0: *The capacity distribution for the complete roadway is similar for the before and after period*
- H_1: *The capacity distribution for the complete roadway is significantly different for the before and after period*

To this end, results for the capacity (breakdown flow distributions) of the complete roadway have been plotted for several percentiles of the distribution in Table 4.1. As can be seen from this table, the number of percentiles of the capacity distribution that have been estimated per location and measurement period varies a lot. For the Bavel location, for instance, most of the capacity distribution has been plotted. On the other hand, for the Moergestel location, under the 130 km/h limit, only the 5th and 10th percentiles have been reached.

There are many more potential study locations that have experienced a speed limit change, but only these eight locations have, so far, been found to contain sufficient breakdown data for a good approximation of the capacity distribution. What is interesting to see is that there does not seem to be a uniform trend towards either a positive or negative direction for the data (see Table 4.1). There is also quite some variation in the magnitude of the change in terms of a percentage-wise increase/reduction.

Positive Results

Of the eight locations in this study, only one location (A2-L Valkenswaard) has seen a relatively large increase in capacity, where most percentiles are in the range of 3% to 5% higher. Another location which has seen a moderate increase is the A58-R Goirle location. Increases at this location are in the range of 1% to 2,5%.

Negative Results

Three locations have experienced small to moderate reductions in capacity: The A27-L on-ramp near Lexmond has experienced reductions in the range of -1,2% to -2.5 %, the A58-L on-ramp near Bavel has seen moderate reductions in the range of -0,6% to -2.3% (with also two small positive results) and the A58-R on-ramp near the Sint Annabosch junction has seen moderate to large reductions in the range of -0,7% to -4.8%.

Percentile	P-5	P-10	P-15	P-20	P-25	P-30	P-40	P-50	P-70	P-75
A2-L Valkenswaard (120)	3.900	4.188	4.260	4.344	4.380	4.476	4.585	4.635	4.803	-
A2-L Valkenswaard (130)	4.020	4.336	4.488	4.536	4.578	4.656	4.777	-	-	-
A2-L Valkenswaard (%)	3,1%	3,5%	5,4%	4,4%	4,5%	4,0%	4,2%	-	-	-
A2-R Valkenswaard (120)	4.068	4.178	4.356	4.419	4.460	4.476	4.505	4.577	4.764	-
A2-R Valkenswaard (130)	4.109	4.271	4.296	4.387	4.428	4.564	4.608	4.656	-	-
A2-R Valkenswaard (%)	1,0%	2,2%	-1,4%	-0,7%	-0,7%	2,0%	2,3%	1,7%	-	-
A27-L Lexmond (120)	4.278	4.364	4.423	4.512	-	-	-	-	-	-
A27-L Lexmond (130)	4.170	4.282	4.368	-	-	-	-	-	-	-
A27-L Lexmond (%)	-2,5%	-1,9%	-1,2%	-	-	-	-	-	-	-
A27-R Lexmond (120)	3.924	4.062	4.246	4.368	4.428	-	-	-	-	-
A27-R Lexmond (130)	3.840	3.981	4.296	4.423	4.498	-	-	-	-	-
A27-R Lexmond (%)	-2,1%	-2,0%	1,2%	1,3%	1,6%	-	-	-	-	-
A58-L Bavel (120)	3.648	3.792	3.864	3.888	3.936	4.021	4.116	4.200	4.524	4.524
A58-L Bavel (130)	3.600	3.732	3.780	3.804	3.852	3.936	4.092	4.260	4.421	4.560
A58-L Bavel (%)	-1,3%	-1,6%	-2,2%	-2,2%	-2,1%	-2,1%	-0,6%	1,4%	-2,3%	0,8%
A58-L Moergestel (120)	4.224	4.302	4.502	4.615	4.673	4.730				
A58-L Moergestel (130)	4.212	4.429	-	-	-	-	-	-	-	-
A58-L Moergestel (%)	-0,3%	3,0%	-	-	-	-	-	-	-	-
A58-R Sint Annabosch (120)	3.888	3.972	4.128	4.203	4.323	4.437	-	-	-	-
A58-R Sint Annabosch (130)	3.701	3.864	3.996	4.148	4.293	4.393	4.473	4.532	-	-
A58-R Sint Annabosch (%)	-4,8%	-2,7%	-3,2%	-1,3%	-0,7%	-1,0%	-	-	-	-
A58-R Goirle (120)	3.816	3.984	4.092	4.152	4.205	4.260	4.303	4.392	-	-
A58-R Goirle (130)	3.895	4.056	4.155	4.188	4.248	4.367	4.404	4.460	-	-
A58-R Goirle (%)	2,1%	1,8%	1,5%	0,9%	1,0%	2,5%	2,3%	1,6%	-	-

Table 4.1. – Estimated percentiles for the capacity distribution for complete roadway

Indeterminate Results

Yet three other locations are indeterminate, as they contain both negative as well as positive differences: The A2-R on-ramp near Valkenswaard has experienced some moderate increases for the lower and higher percentiles of the distribution, whilst also experiencing some lower percentiles for the 15 to 25 percentile range. The A27-R on-ramp near Lexmond has seen a moderate reduction in percentiles for the 5th and 10th percentiles, whilst experiencing a moderate increase in higher percentiles. Lastly, the on-ramp on the A58-L near Moergestel has only been estimated to a very small extent under the 130 km/h. As such, it is indeterminate what the effect is at this location, as there is both a small negative effect as well as a moderate positive effect.

Inter-Location Variation in Results

Additionally, it can be seen that a lot of variation in capacity results exists between different locations. For example, for the 5th percentile, measurements for the 120 km/h limit vary in the range of 3648 $\frac{veh.}{hour}$ to 4278 $\frac{veh.}{hour}$, whilst for the 130 km/h limit they vary in the range of 3600 $\frac{veh.}{hour}$ to 4212 $\frac{veh.}{hour}$. Similarly, the range of results for the 25th percentile is 3936 $\frac{veh.}{hour}$ to 4673 $\frac{veh.}{hour}$, under the 120 km/h limit, and 3852 to 4578 $\frac{veh.}{hour}$, under the 130 km/h limit. This variation in results between locations is most likely to be caused by location specific factors such as infrastructural differences and (average) truck traffic intensity. Because of the fact that these differences have not been accounted for, locations cannot be directly compared with each other.

Willcoxon Test

Because none of the capacity distributions has been fully estimated, the application of parametric tests is inappropriate for the determination of changes in the position of the capacity distribution. To solve this issue, a non-parametric test of central position, called the Willcoxon Signed Rank Sum Test (see subsection 3.2.6), is applied to compare the differences in flow at (available) percentiles of the breakdown probability distribution. Results of the Willcoxon Signed Rank Sum Test are tabulated in Table 4.2:

Location	T+	T-	N	Z-value
A2-L Valkenswaard	860,0	1,0	41	5,57***
A2-R Valkenswaard	1589,5	301,5	61	4,63***
A27-L Lexmond	0,0	190,0	19	-3,82**
A27-R Lexmond	179,5	198,5	27	-0,23
A58-L Bavel	425,5	2424,5	75	-5,28***
A58-L Moergestel	105,0	0,0	14	3,30**
A58-R St. Annabosch	0,0	561,0	33	-5,01***
A58-R Goirle	2011,0	5,0	63	6,87***

*** p<0.01, ** p<0.05, * p<0.1

Table 4.2. – Willcoxon signed rank sum test results (Complete Roadway)

When $Z > 0$ the distribution of the "after" measurement period (130 km/h) is higher than the "before" measurement period (120 km/h). Similarly, when $Z < 0$ the distribution under the 130 km/h limit is lower. As can be seen, all locations, except for the A27-R Lexmond location, show significant differences. According to the test results, four locations have experienced a significant increase in breakdown flows during the study period, while three locations have experienced a significant decrease and one location has experienced no significant change. As such, no clear direction can be seen in the data for the complete roadway results in a comparison of just the "before" and "after" data and it is uncertain what the effect has been of the speed limit on the capacity distribution of the complete roadway.

Conclusion

It has been the aim in this section to test hypothesis 1a, in order to determine whether a capacity change in any direction has occurred. First an assessment was made of percentual changes at a number of percentiles such as the 5th and 10th percentiles. It was found from this analysis that two locations exhibited definitive positive results, three locations exhibited negative results and three locations exhibited results for which the effect was indeterminate. In a non-parametric statistical test that was subsequently performed on all available percentiles it was found that four locations had a significantly higher capacity distribution, while three locations had a significantly lower distribution and one location was indeterminate. As such, it can be concluded for hypothesis 1a that the null hypothesis has been rejected for 7 out of 8 locations, which entails that, in most cases, there are differences in capacity between the before and after period. However, no uniform direction of the effect capacity effect has been found as the number of negative versus positive changes is three to four. It is also uncertain to what extent the changes in this sample are a result of a change in the speed limit rather than "noise" caused by omitted variables such as changes in truck traffic levels.

4.2. Passing Lane

In this section, hypothesis 1b will be tested which states:

- H_0: *The breakdown flow distribution for the passing lane is similar for the before and after period*
- H_1: *The breakdown flow distribution for the passing lane is significantly different for the before and after period*

To this end, results for the breakdown flows of the passing lane samples have been plotted for several percentiles of the distribution in Table 4.3.

An interesting aspect of the passing lane breakdown flow distributions is that much higher flow values are observed in this lane than in the shoulder lane. Please note that this is partly the case because the flow measurements are based on direct vehicle measurements and are not corrected to Passenger Car Equivalents on the basis of vehicle lengths. Nonetheless, the truck traffic levels, as tabulated in Table 4.7, are not sufficient to fully explain the magnitude in difference of flow between the lanes (which is almost on the order of 2 when comparing the passing lane with the shoulder lane) and, as such, it is certain that a larger proportion of traffic is present in the passing lane at the moment of breakdown.

Positive Results

Higher breakdown flows were realized for all percentiles of three locations (see Table 4.3): At the A2-L on-ramp near Valkenswaard the largest increases in breakdown flow are visible (in a range of 4% to 8%), meaning that shorter headways are maintained, which do not seem to lead to significantly more instability. For the other roadway, at the on-ramp on the A27-R near Valkenswaard, more moderate positive results have been found. The difference in flow for each percentile are in the range of 0,8% to 3,4%. Lastly, for the lane reduction location on the A58-R near Goirle, positive increases have also been found in the range of 1,3% to 6,0%.

Negative Results

In contrast to the positive differences, there are also three locations where the results are almost exclusively negative (see Table 4.3). Locations that experience large exclusive reductions are the A27-L on-ramp near Lexmond (reduction in range of -4,3% to -6,2%) and the A27-R on-ramp near Lexmond (reduction in range of -2,1% to -6,7%). Similarly, the A58-R on-ramp near Sint Annabosch experiences negative differences (reduction in range of -1,8% to -3,4%), with the exception of a positive difference in the 15th percentile (+1,3%)

Indeterminate Results

Additionally, two locations have results that seem to be indeterminate (see Table 4.3). The on-ramp on the A58-L near Moergestel varies in a range of 0% to 1,6%, though it is hard to determine whether this can be considered as an increase, as the breakdown flow distribution has only been estimated to a very limited extent under the 130 km/h limit at this location. Also the on-ramp location on the A58-L near Bavel shows some interesting results, as the difference starts off negative for the lower percentiles and then changes into a positive difference around the 30th percentile. This may entail that, at this particular location, the introduction of the 130 km/h limit has led to a relatively greater breakdown probability at the lower end of the breakdown flow distribution while, for higher levels of traffic demand, it may have led to a lower breakdown probability.

Percentile	P-5	P-10	P-15	P-20	P-25	P-30	P-40	P-50	P-70	P-75
A2-L Valkenswaard (120) [P]	2.412	2.568	2.652	2.664	2.772	2.832	-	-	-	-
A2-L Valkenswaard (130) [P]	2.508	2.720	2.750	2.878	2.964	3.009	-	-	-	-
A2-L Valkenswaard (%) [P]	4,0%	5,9%	3,7%	8,0%	6,9%	6,3%	-	-	-	-
A2-R Valkenswaard (120) [P]	2.520	2.592	2.664	2.724	2.748	2.780	2.808	2.922	-	-
A2-R Valkenswaard (130) [P]	2.541	2.640	2.715	2.748	2.820	2.852	2.904	2.943	-	-
A2-R Valkenswaard (%) [P]	0,8%	1,9%	1,9%	0,9%	2,6%	2,6%	3,4%	0,7%	-	-
A27-L Lexmond (120) [P]	2.832	-	-	-	-	-	-	-	-	-
A27-L Lexmond (130) [P]	2.760	-	-	-	-	-	-	-	-	-
A27-L Lexmond (%) [P]	-2,5%	-	-	-	-	-	-	-	-	-
A27-R Lexmond (120) [P]	2.478	2.646	2.779	2.999	-	-	-	-	-	-
A27-R Lexmond (130) [P]	2.436	2.628	2.796	-	-	-	-	-	-	-
A27-R Lexmond (%) [P]	-1,7%	-0,7%	0,6%	-	-	-	-	-	-	-
A58-L Bavel (120) [P]	2.388	2.448	2.472	2.532	2.568	2.568	2.604	2.712	2.952	-
A58-L Bavel (130) [P]	2.364	2.426	2.472	2.520	2.556	2.600	2.688	2.800	-	-
A58-L Bavel (%) [P]	-1,0%	-0,9%	0,0%	-0,5%	-0,5%	1,2%	3,2%	3,3%	-	-
A58-L Moergestel (120) [P]	2.786	2.941	3.048	3.120					-	-
A58-L Moergestel (130) [P]	2.785	2.988	-	-	-	-	-	-	-	-
A58-L Moergestel (%) [P]	0,0%	1,6%	-	-	-	-	-	-	-	-
A58-R Sint Annabosch (120) [P]	2.268	2.424	2.439	2.653	-	-	-	-	-	-
A58-R Sint Annabosch (130) [P]	2.191	2.352	2.471	2.604	2.670	2.703	2.751	2.767	-	-
A58-R Sint Annabosch (%) [P]	-3,4%	-3,0%	1,3%	-1,8%	-	-	-	-	-	-
A58-R Goirle (120) [P]	2.628	2.724	2.784	2.868	2.892	2.928	3.000	3.059	3.221	-
A58-R Goirle (130) [P]	2.680	2.796	2.845	2.904	2.959	3.102	3.126	3.156	-	-
A58-R Goirle (%) [P]	2,0%	2,6%	2,2%	1,3%	2,3%	6,0%	4,2%	3,2%	-	-

Table 4.3. – Estimated percentiles for the capacity distribution for the passing lane

Inter-Location Variation in Results

It can be seen that again a lot of variation exists between results for different locations in the passing lane, as the lowest value for the 120 km/h in the 5th percentile is 2268 $\frac{veh.}{hour}$ and the highest is 2832 $\frac{veh.}{hour}$, while for the 130 km/h the lowest breakdown flow is 2191 and the highest is 2785 $\frac{veh.}{hour}$. This entails that a reduction of approximately 5% in variation was found between different locations for the 130 km/h limit. Similarly, for the 120 km/h limit at the 25th percentile the range of flows is between 2568 $\frac{veh.}{hour}$ and 2892 $\frac{veh.}{hour}$, while for the 130 km/h it is between 2556 $\frac{veh.}{hour}$ and 2964 $\frac{veh.}{hour}$. As such, in contrast to the 5th percentile, the variation in breakdown flows between locations for the 25th percentile has increased by approximately 26% for the 130 km/h limit. Again, variation in results is most likely to be caused by location specific factors

Willcoxon Test

In Table 4.4 an overview of results for the Willcoxon Signed Rank Sum Test for the passing lane samples has been provided. For the passing lane samples, the pattern is similar to the complete roadway samples: four locations exhibit significant positive results, three locations exhibit significant negative results and one location (again 27-R Lexmond) exhibits an insignificant negative result. Also in this sample, a uniform direction of the effect of the speed limit cannot be distinguished, which makes it uncertain how the breakdown flow in this lane is affected by the speed limit.

Location	T+	T-	N	Z-value
A2-L Valkenswaard (Passing Lane)	496,0	0,0	31	4,86***
A2-R Valkenswaard (Passing Lane)	1273,0	2,0	50	6,13***
A27-L Lexmond (Passing Lane)	0,0	36,0	8	-2,52**
A27-R Lexmond (Passing Lane)	70,0	120,0	19	-1,01
A58-L Bavel (Passing Lane)	1560,0	585,0	65	3,02***
A58-L Moergestel (Passing Lane)	55,0	0,0	10	2,80***
A58-R St. Annabosch (Passing Lane)	0,0	210,0	20	-3,92**
A58-R Goirle (Passing Lane)	1888,0	3,0	61	6,77***

*** $p<0.01$, ** $p<0.05$, * $p<0.1$

Table 4.4. – Willcoxon signed rank sum test results (Passing lane)

Discussion

The results for the passing lane samples are found in a range of approximately 2200 $\frac{veh.}{hour}$ to 3200 $\frac{veh.}{hour}$, which implies that very short-distance following is the case. For example, a flow of 2200 $\frac{veh.}{hour}$ implies an average gross headway time ($\langle h^g \rangle$) of 1,64 seconds, which is below the recommendations set by the Dutch government, and a flow of 3200 $\frac{veh.}{hour}$ implies an average gross headway time of only 1,13 seconds! Though no specific mentioning is made in the Dutch Highway Capacity manual, the minimum experienced flow at the 5th percentile of the distribution, already exceeds the 2150 $\frac{veh.}{hour}$ (50th percentile) capacity per lane that is indicated in the manual for a dual lane freeway (Heikoop *et al.*, 2015, p.31).

Conclusion

In order to formulate an answer to hypothesis 1b, it has been the goal in this section to analyze the breakdown flow distribution results of the passing lane samples. An assessment was performed on a set of percentiles of the distributions and the percentual change in the flow between the "before" and "after" period was analyzed. It was found that three locations experienced an increase in breakdown flows, three locations experienced a decrease and two locations remained indeterminate. Subsequently a statistical test was performed to analyze the significance of these observed changes. It was found that five locations exhibited a significant positive change, while two locations exhibited a significant negative change and one location exhibited an insignificant negative change. The null hypothesis is rejected in favor of the alternative hypothesis in 7 out of 8 cases, which entails that a significant change to the breakdown flow distribution of the passing lane has occurred. The change in breakdown flows seems to be primarily in the direction of an increase in the passing lane breakdown flow, with a ratio of 5 to 2 samples recording a positive change. This is an indication that under higher speed limits, more traffic seems to be present in the passing lane at the moment of breakdown, which would imply a positive relationship between the passing lane utilization rate and the speed limit.

4.3. Shoulder Lane

It is the aim of this section to test hypothesis 1c, to determine whether a significant change in shoulder lane breakdown flow distributions has occurred between the 130 km/h and 120 km/h limit. Generally, the breakdown flows in the shoulder lane are much lower than the flows in the passing lane, making it likely to be the case that the breakdown will occur in the passing lane rather than in the shoulder lane, after which it subsequently spills back to the shoulder lane. As such, the breakdown flow distribution of the shoulder lane flows (as tabulated in Table 4.5) is not necessarily an indication of the capacity of this lane, but rather a representation of the flows that occur in this lane at the moment of breakdown.

Positive Results

As can be seen from Table 4.5 there are two locations that seem to experience positive effects, such as the on-ramp on the A2-L near Valkenswaard (with a range of 0,7% to 7,7%) and the lane reduction area on the A58-R near Goirle (with a range of 0,1% to 5,8%).

Negative Results

Additionally, there are also three locations for which the effect is negative. Namely, the on-ramp on the A27-L near Lexmond (which varies in a range of -4,3% to -6,2%), the on-ramp on the A27-R near Lexmond (which experiences a range of -2,1% to -6,7%) and the on-ramp on the A58-L near Bavel (which experiences a reduction in the rang of -0,4% to -6,4%, with one positive difference at the 40th percentile).

Percentile	P-5	P-10	P-15	P-20	P-25	P-30	P-40	P-50	P-70	P-75
A2-L Valkenswaard (120) [S]	1.418	1.550	1.596	1.632	1.692	1.704	1.771	1.836	1.932	1.949
A2-L Valkenswaard (130) [S]	1.528	1.584	1.632	1.644	1.718	1.772	1.896	1.925	-	-
A2-L Valkenswaard (%) [S]	7,7%	2,2%	2,3%	0,7%	1,6%	4,0%	7,1%	4,8%	-	-
A2-R Valkenswaard (120) [S]	1.541	1.584	1.648	1.692	1.692	1.740	1.774	1.776	-	-
A2-R Valkenswaard (130) [S]	1.563	1.584	1.641	1.703	1.704	1.716	1.776	1.824	1.906	-
A2-R Valkenswaard (%) [S]	1,5%	0,0%	-0,4%	0,6%	0,7%	-1,4%	0,1%	2,7%	-	-
A27-L Lexmond (120) [S]	1.590	1.632	1.674	-	-	-	-	-	-	-
A27-L Lexmond (130) [S]	1.492	1.562	1.572	1.611	1.697	1.726	-	-	-	-
A27-L Lexmond (%) [S]	-6,2%	-4,3%	-6,1%	-	-	-	-	-	-	-
A27-R Lexmond (120) [S]	1.440	1.500	1.560	1.584	1.608	1.644	-	-	-	-
A27-R Lexmond (130) [S]	1.344	1.416	1.466	1.500	1.575	1.596	1.714	1.740	-	-
A27-R Lexmond (%) [S]	-6,7%	-5,6%	-6,0%	-5,3%	-2,1%	-2,9%	-	-	-	-
A58-L Bavel (120) [S]	1.320	1.344	1.380	1.404	1.421	1.440	1.464	1.584	-	-
A58-L Bavel (130) [S]	1.236	1.284	1.320	1.368	1.380	1.434	1.523	1.548	1.596	1.624
A58-L Bavel (%) [S]	-6,4%	-4,5%	-4,3%	-2,6%	-2,9%	-0,4%	4,0%	-2,3%	-	-
A58-L Moergestel (120) [S]	1.452	1.512	1.545	1.548	1.571	1.668	-	-	-	-
A58-L Moergestel (130) [S]	1.410	1.500	1.588	-	-	-	-	-	-	-
A58-L Moergestel (%) [S]	-2,9%	-0,8%	2,8%	-	-	-	-	-	-	-
A58-R Sint Annabosch (120) [S]	1.560	1.656	1.680	1.716	1.728	1.786	1.822	1.824	-	-
A58-R Sint Annabosch (130) [S]	1.476	1.548	1.608	1.632	1.764	1.812	-	-	-	-
A58-R Sint Annabosch (%) [S]	-5,4%	-6,5%	-4,3%	-4,9%	2,1%	1,5%	-	-	-	-
A58-R Goirle (120) [S]	1.200	1.272	1.296	1.332	1.383	1.428	1.488	1.571	-	-
A58-R Goirle (130) [S]	1.232	1.284	1.332	1.392	1.464	1.488	1.540	1.572	-	-
A58-R Goirle (%) [S]	2,6%	0,9%	2,8%	4,5%	5,8%	4,2%	3,5%	0,1%	-	-

Table 4.5. – Estimated percentiles for the capacity distribution for the shoulder lane

Indeterminate Results

The effect for the remaining three locations (Valkenswaard, A2-L; Moergestel, A58-L; Sint Annabosch, A58-R) is indeterminate, as all of these locations exhibit both positive and negative differences.

Inter-Location Variation in Results

Again, a lot of variation exists between different locations. For the 5th percentile, for instance, the values of flow for the 120 km/h limit vary between 1200 $\frac{veh.}{hour}$ and 1590 $\frac{veh.}{hour}$, while for the 130 km/h limit it varies between 1232 $\frac{veh.}{hour}$ and 1563 $\frac{veh.}{hour}$. Similarly, for the 25th percentile, the values of flow for the 120 km/h limit vary between 1383 $\frac{veh.}{hour}$ and 1728 $\frac{veh.}{hour}$, while for the 130 km/h limit it varies between 1464 $\frac{veh.}{hour}$ and 1764 $\frac{veh.}{hour}$. Consequently, under the 130 km/h limit, inter-location variation for breakdown flows in the 5th percentile is reduced with approximately 15%, while in the 25th perentile it is reduced with approximately 13%. This may be an indication that the flow in the shoulder lane becomes more homogeneous as a result of the higher limit.

Willcoxon Test

Location	T+	T-	N	Z-value
A2-L Valkenswaard (Shoulder Lane)	2013,0	3,0	63	6,88***
A2-R Valkenswaard (Shoulder Lane)	1525,5	619,5	65	2,96***
A27-L Lexmond (Shoulder Lane)	0,0	153,0	17	-3,62**
A27-R Lexmond (Shoulder Lane)	0,0	780,0	39	-5,44***
A58-L Bavel (Shoulder Lane)	283,5	1369,5	57	-4,31***
A58-L Moergestel (Shoulder Lane)	153,0	0,0	17	3,62**
A58-R St. Annabosch (Shoulder Lane)	118,0	548,0	36	-3,38***
A58-R Goirle (Shoulder Lane)	1342,0	36,0	52	5,95***

*** $p<0.01$, ** $p<0.05$, * $p<0.1$

Table 4.6. – Willcoxon signed rank sum test results (Shoulder Lane)

Results for the differences in shoulder lane breakdown flows are tabulated in Table 4.6. As can be seen from these results, all samples have been tested to be significantly different in the "before" and "after" period. Four locations have shown a positive change in shoulder lane breakdown flows, while four other locations have shown a negative change. As such, the breakdown flow results in the shoulder lane samples are more pronounced than the changes in the other samples. Nonetheless, a clear direction is not visible in the data as the ratio of positive to negative changes is four to four.

Discussion

Most interesting about the data in Table 4.5 is the fact that the 50th percentiles of the breakdown flows are all below the 1900 $\frac{veh.}{hour}$ that is proposed for the (50th percentile) capacity of a single lane with no overtaking opportunities (assuming 15% truck traffic) in the Dutch Highway Capacity Manual (Heikoop *et al.*, 2015, p.31). This single lane capacity is already considered to be quite low, as inefficiency in the stream is created by slow vehicles leaving large gaps. Generally, on multi-lane roads, capacities per lane are assumed to be higher, because vehicles can fill in the gaps that naturally emerge in the stream. Considering that for most of the locations in this study the truck percentage is around this assumed value of 15%, it is surprising to see that almost none of the estimated breakdown flows reaches this proposed single lane capacity. This entails that driving behavior in the shoulder lane is more inefficient on average than in a single lane facility. This, in turn, means that vehicles in this lane are driving, on average, at larger headways than is necessary to oblige to the 2 seconds rule and that gaps are not filled by drivers on the passing lane. At the same time, vehicles in the passing lane are driving at much shorter average headways than are safe (see Table 4.3). Furthermore, as displayed in Figure 4.2, flows in the passing lane seem to decrease after the $F \rightarrow C$ transition, while flows in the shoulder lane seem to increase. This is an indication that, once congestion has set in, drivers on the passing lane will start moving to the shoulder lane to fill in the gaps in the stream that existed before the breakdown.

Conclusion

In an attempt to determine the answer to hypothesis 1c, percentual changes for given percentiles of the shoulder lane breakdown flow distributions have been tabulated in Table 4.5. It was found from this analysis that two locations exhibited positive changes, three locations exhibited negative changes and three other locations were indeterminate. Through performing a non-parametric statistical test on all available percentiles of the distribution, it was found that four location showed significant positive changes while four other locations showed significant negative changes. As such, the null hypothesis can be rejected in favor of the alternative hypothesis in 8 out of 8 cases, meaning that for each location a significant change in shoulder lane breakdown flows has occurred. Nonetheless, the ratio of positive to negative changes is 4 to 4, which means that, again, no uniform direction can be seen from the data and a general effect of the speed limit on capacity cannot be derived.

4.4. Presence of Truck Traffic

To assess whether changes in truck traffic levels can be attributed to changes in the breakdown flow distribution, it is the aim in this section to determine an answer to hypotheses 2, which states:

Hypothesis 2:

- H_0: *No change to the level of truck traffic has occurred between the before and after period*
- H_1: *A significant change to the level of truck traffic has occurred between the before and after period*

Truck traffic data was obtained for several locations by obtaining vehicle length data from loop detectors in the vicinity of those particular locations. Passenger cars were defined as any vehicle with a length of less than 5,60 meters, while trucks were defined as any vehicle with a length exceeding those 5,60 meters. Only locations where reliable 1-minute vehicle length data was available were used in this analysis. The exact times at which traffic breakdown has occurred are known from the breakdown flow data (which have been produced by the Product Limit Method). These exact times were matched to the 1-minute truck traffic data to identify the level of truck traffic at that particular moment. To match the 1-minute truck traffic data to the 5-minute flow data, a 5-minute average was taken of the values of truck traffic in the minutes surrounding the moment of breakdown. Data from a total of 4 out of 8 locations were deemed to be sufficiently reliable for performing this analysis and a mean truck traffic level was derived for each location, as well as the standard deviation and sample size (which is equal to the number of breakdowns). Subsequently a t-test has been performed to test whether significant changes to the level of truck traffic have occurred. The results for this test have been presented in Table 4.7.

From Table 4.7 it can be seen that at two locations no significant change to the mean truck traffic level has occurred, while a two other locations a significant change to the truck traffic level has occurred. Therefore, hypothesis 2 can be rejected in favor of the alternative hypothesis in 2 out of 4 cases. To examine the effects of these changes in truck traffic in relation to the capacity effects that were found in sections 4.1, 4.2 and 4.3 an effects table has been presented in Table 4.8.

Location	T	μ_{130}	μ_{120}	σ_{130}	σ_{120}	n_{130}	n_{120}
A27-L Lexmond	0.64	0.1528	0.1470	0.0498	0.0374	54	40
A58-L Bavel	0.71	0.1467	0.1440	0.0441	0.0431	265	252
A58-L Moergestel	2.75***	0.1849	0.1568	0.0654	0.0497	66	62
A58-R Goirle	4.18***	0.0896	0.0675	0.0572	0.0307	148	163

*** $p<0.01$, ** $p<0.05$, * $p<0.1$

Table 4.7. – T-test for the comparison of mean truck traffic levels

From Table 4.8 it can be seen that locations which experienced an increase in total capacity, also experienced increases in breakdown flows in both the passing lane and the shoulder lane. At location A58-R Goirle capacity seems to have increased under the 130 km/h, despite a significant increase in truck traffic. It may be the case that the effect of trucks in this bottleneck is less relevant, because it is a left-lane reduction bottleneck, rather than an on-ramp (see Figure 3.3). For the A58-L Moergestel location a significant increase in truck traffic was also visible, but the capacity effect at this location is indeterminate, perhaps that the 130 km/h limit may have compensated for the capacity reduction caused by the trucks at this location.

On the other hand, there are two locations with an insignificant change in truck levels which have experienced a reduction in total capacity. This would be an indication that perhaps in some cases the 130 km/h limit could also lead to a reduction in capacity, even when traffic composition is the same.

In conclusion, despite estimating truck traffic levels for various locations, it is difficult to distinguish a general trend for capacity under the 130 km/h limit compared with the 120 km/h limit. For this reason, it has been chosen in chapter 5 to apply least squares regression

	Capacity Difference	Passing Lane	Shoulder Lane	Change in Truck Traffic
A2-L Valkenswaard	+	+	+	
A2-R Valkenswaard	0	+	0	
A27-L Lexmond	-	-	-	0
A27-R Lexmond	0	0	-	
A58-L Bavel	-	0	-	0
A58-L Moergestel	0	+	0	+
A58-R Sint Annabosch	-	-	0	
A58-R Goirle	+	+	+	+

Table 4.8. – Effects table ("+" is primarily positive, "0" is indeterminate and "-" is primarily negative)

theory to more directly account for the effects of truck traffic and location specific factors.

4.5. Lane Flow Distribution at moment of Breakdown

In this section, it is the goal to test hypothesis 3, which states:

- H_0: *The passing lane utilization rate has stayed the same between the two measurement periods.*
- H_1: *The passing lane utilization rate has significantly changed between the two measurement periods.*

The passing lane utilization rate (lane flow fraction) is an indication of how traffic is distributed over the lanes on a two lane freeway. The higher this fraction becomes, the more traffic is present on the passing lane and the less traffic is driving on the shoulder lane. The sum of the breakdown flows on both lanes determine the capacity and these breakdown flows are, in turn, affected by the degree to which each of the lanes is utilized. It is therefore relevant to assess whether significant changes have occurred to the lane flow fraction.

Summary statistics have been plotted for the proportion of vehicles that is present in the passing lane at the moment of breakdown in Table 4.9. As can be seen from this table, the mean flow proportion of vehicles in the passing lane is high and relatively constant in a range of approximately 60% to 65%. Furthermore, the standard deviations are also very small, meaning that not much variation around this range is present in the data.

To determine whether a significant change in flow proportion has occurred as a consequence of the change to the 130 km/h limit, a Z-test for the comparison of proportions has been performed in Table 4.10 for the mean flow fraction in the passing lane. Though only three of the eight samples are significant at at least the 10% level, most results indicate a positive relation (z values are positive) between the height of the speed limit and the flow fraction in the passing lane. This is consistent with findings from (Knoop *et al.*, 2010), who have found that the flow-fraction of traffic in the left-most lane on a freeway will become higher as the speed limit is increased. As such, there is evidence that for a higher speed limit

	μ_{120}	μ_{130}	σ_{120}	σ_{130}	n_{120}	n_{130}
A2-L Valkenswaard	0.609	0.612	0.028	0.028	195	150
A2-R Valkenswaard	0.615	0.616	0.025	0.025	99	104
A27-L Lexmond	0.628	0.629	0.049	0.059	40	55
A27-R Lexmond	0.625	0.631	0.031	0.031	185	232
A58-L Bavel	0.647	0.653	0.025	0.027	253	268
A58-L Moergestel	0.647	0.656	0.023	0.031	62	66
A58-R Sint Annabosch	0.571	0.583	0.026	0.028	153	189
A58-R Goirle	0.684	0.681	0.028	0.031	163	148

Table 4.9. – Summary statistics of flow proportions at breakdown measurements

a larger fraction of vehicles will be present in the passing lane at the moment of breakdown than under a lower limit.

	n_1	n_2	Z
A2-L Valkenswaard	195	150	1.14
A2-R Valkenswaard	99	104	0.06
A27-L Lexmond	40	55	0.17
A27-R Lexmond	185	232	2.38***
A58-L Bavel	253	268	3.43***
A58-L Moergestel	62	66	1.21
A58-R Sint Annabosch	153	189	4.09***
A58-R Goirle	163	148	-1.28

*** $p<0.01$, ** $p<0.05$, * $p<0.1$

Table 4.10. – Proportion comparison for flow proportions at breakdown measurements.

Consequently, it is proposed that the null hypothesis of hypothesis 4 can be rejected in favor of the alternative hypothesis for at least 3 out of 8 cases, which entails that it can be stated with reasonable certainty that the height of the legal limit is positively related to the flow fraction in the passing lane. Despite the fact that passenger car equivalents were not corrected for, the high fraction of flow in the passing lane is unlikely to merely be caused by a larger proportion of trucks in this lane, given that at the location of A58-L Bavel, for instance, the lane flow fraction in the passing lane has increased (see Table 4.10), despite a similar level of truck traffic (see Table 4.7). This may be an indication that under-utilization of the shoulder lane has increased under the 130 km/h limit, with respect to the 120 km/h limit. Again, this confirms the findings from chapter 4 and corroborates the theories of Daganzo (2002a) and Kerner (2004) regarding lane choice behavior in the critical density region. Furthermore, the under-utilization of the shoulder lane is also consistent with the lane choice behavior found by De Baat (2016) and Keyvan-Ekbatani *et al.* (2016).

4.6. Discussion

A direct comparison was made between capacities under different speed limit regimes by means of a simple before-and-after analysis. It has been shown that flows in the passing lane are much higher in all cases than in the shoulder lane and it has been argued that this cannot be exclusively caused by the presence of trucks in the shoulder lane. This entails that lane choice behavior, as discussed in chapter 2, is of importance in explaining traffic breakdown and provides some evidence for the fact that traffic does exhibit dynamics of the "slugs and rabbits" theory posed by Daganzo (2002a) as well as some proof for the existence of a two-capacity phenomenon (Banks, 1991), in which the capacity is reached in the passing lane first and, subsequently, spills back to the shoulder lane (Daganzo, 2002a).

In accordance with the two-pipe regime from Daganzo (2002a) and the observation by Cassidy and Bertini (1999) that the breakdown often occurs in the left-most lane, it is expected that this is also the case for the data at hand, as the headways in the passing lane are much more likely to be sufficiently short to violate the weak string stability condition as in Equation 2.6. This makes sense, because the breakdown flows in the passing lane, as tabulated in Table 4.3, are much more representative of the meta-stable state described by Kerner (2004) than the flows experienced in the shoulder lane (see Table 4.5).

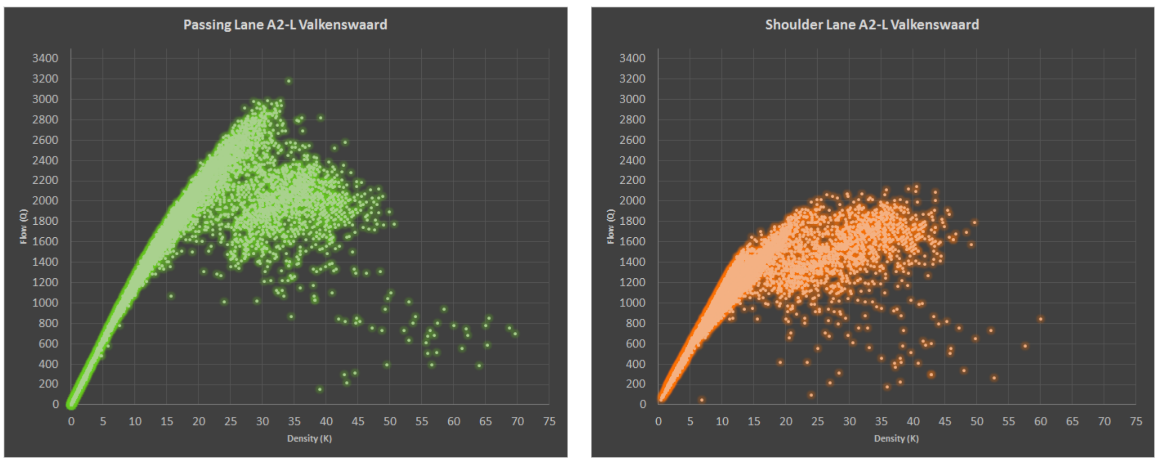

Figure 4.2. – Fundamental diagram per lane (Passing lane on the left; Shoulder lane on the right) for 5-minute aggregation intervals for on-ramp location Valkenswaard A2-L (March, April and May 2018).

Furthermore, when plotting fundamental diagrams per lane, it can be seen that the passing lane is characterized by a tall free flow branch with a large capacity drop, which resembles the fundamental diagram as formulated by Wu (2002) - see the left picture of Figure 4.2 - while the shoulder lane is characterized by a more gradual and more graceful degradation when transitioning into the congested state, almost resembling the fundamental diagram by Greenshields (1934) - see the right picture of Figure 4.2. These differences in the shapes of the fundamental diagrams are consistent with the theories posed by Daganzo (2002a) as well as Kerner (2004), who both argue for a capacity drop in the left-most lane caused by a disturbance exceeding the critical threshold. A critical threshold, which, has become sufficiently low as a consequence of "motivated behavior" (Daganzo, 2002a) which has caused the free flow state (F) to become meta-stable (F^{ms}) (Kerner, 2004).

Aforementioned results imply that there exists an inefficiency for both speed limits in which a large share of the driver population drives in the left lane, while there are still gaps to merge into on the shoulder lane. This is consistent with findings from De Baat (2016) who has found that most drivers use a "speed leading" strategy in free flow traffic and, as such, prefer to drive in the fastest lane, which is generally the passing lane. Most drivers will therefore choose to move to the passing lane, which is confirmed by the data, and many drivers will not be willing to move back to the shoulder lane, to fill the gaps. The reason most drivers will not fill the gap is caused by factors as reduced driving comfort for performing a lane change, fear of being caught between slower vehicles and fear of not being able to merge back into the passing lane again, as a result of overcrowding (De Baat, 2016).

4.7. Summary

As can be seen from Table 4.8 and, as discussed before, there seem to be two locations that have experienced a positive change in overall capacity, three locations for which the results are indeterminate and three locations at which a negative change was experienced. For A2-L Valkenswaard and A58-R Goirle all results were positive, while for A27-L Lexmond all results were negative. For the A2-R Valkenswaard and A58-L Moergestel location it can be seen that, despite a positive effect for the passing lane, the overall result is indeterminate in both the overall capacity as well as the shoulder lane. A27-R Lexmond is indeterminate, despite the negative flow in the passing lane and A58-L Bavel and A58-R are negative overall, despite the indeterminate result in the passing lane and shoulder lane respectively.

When tested for significance, the null hypothesis of hypotheses 1a and hypothesis 1b were rejected in 7 out of 8 cases, while the null hypothesis of hypotheses 1c was rejected in all cases. As such, changes to the breakdown flow distributions have occurred to virtually all locations, for the complete roadway, passing lane and shoulder lane.

The direction of the effects for the complete roadway is indeterminate with four positive changes and three negative changes. For the passing lane the direction of the effect seems to be more frequently positive (5 positive changes against 2 negative changes), while for the shoulder lane the effect is also indeterminate (4 positive changes against 4 negative changes). As such, it was found that not all results indicate in the same direction and that a general effect on capacity as a result of the increased 130 km/h speed limit cannot be distinguished.

In section 4.4, truck traffic data have been analyzed for 4 out of 8 locations. A t-test for the comparison of means was performed and from this test it was found that two locations had similar truck traffic levels in both periods and two locations had experienced an increase in truck traffic (which means that the null hypothesis of hypothesis 2 was rejected in 2 cases). For both locations where no change in truck traffic had occurred, the effect of the 130 km/h limit on the capacity distribution seemed to be negative. However, for both locations were an increase in truck traffic did occur, capacity either stayed the same or increased under the 130 km/h limit. As such, a general effect, when taking account of truck traffic levels, can still not be determined from the data at hand.

The last analysis in section 4.5 was performed to investigate how the flow was distributed across the lanes at the moment of traffic breakdown and to what extent these flow fractions have changed between different speed limit regimes. It was found that the mean flow fraction of traffic in the passing lane was relatively stable in a range of 60% to 65% and that there are some indications for a positive relation between the height of the speed limit. As such, it was found that the null hypothesis of hypothesis 3 could be rejected in 3 out of 8 cases, thus providing some evidence for a positive relation between the speed limit and the flow fraction in the passing lane.

Lastly, it was discussed how a strong difference in characteristics was found between the shoulder lane and the passing lane by plotting fundamental diagrams and examining their shape. These findings confirm the behavioral "slugs and rabbits" theory as posed by (Daganzo, 2002a) and corroborate findings from literature regarding lane choice preference and lane choice behavior under different speed limits.

5. Breakdown Flow Regression Results

In this chapter, regression theory is applied for the purpose of analyzing breakdown flow distributions under different speed limits, whilst explicitly taking account of changes in other variables as well. In section 5.1 breakdown flow measurements from the eight-location sample, which was analyzed in chapter 4, will be used in a fixed effects regression to estimate what happens to the mean breakdown flow under different limits, whilst accounting for location specific effects, changes in the lane flow distribution and changes in truck traffic levels (hypothesis 4a). Subsequently, in section 5.2, another sample including 17 locations and three different speed limits (100, 120 and 130 km/h) will be analyzed by means of fixed effects regression, to assess whether similar results can be obtained (hypothesis 4b). Lastly, a relation between the passing lane utilization rate and the speed limit is investigated for both samples, to assess whether a significant relation exists (hypothesis 5).

5.1. Analysis of Breakdown Flows for a limit change from 120 to 130 km/h

In this section, hypothesis 4a will be tested, which states:

- H_0: *The speed limit variable of the eight-location sample is insignificant, when controlling for location specific factors and other relevant variables.*
- H_1: *The speed limit variable of the eight-location sample is signficant, when controlling for location specific factors and other relevant variables.*

In order to explicitly account for truck traffic, lane choice behavior and location specific effects, breakdown flows from the 8 location sample (see Table 3.2) have been gathered into a dataset, which will be analyzed by means of regression techniques. In Table 5.1 the summary statistics for the relevant variables in the dataset have been presented, where variable "BF" stands for the breakdown flow, where variable "V120" is a dummy variable for whether the 120 km/h limit ($V120 = 1$) or the 130 km/h limit ($V120 = 0$) applies, where "TT" stands for "Truck Traffic", representing the average truck traffic level of the 5 minutes surrounding the moment of breakdown, where "LFF" stands for Lane Flow Fraction, representing the fraction of traffic present in the passing lane at the moment of breakdown, and where "LFF^2" is the exponential term of variable LFF.

A quadratic term was chosen for this variable, because it was found to represent the relation between the breakdown flow and the lane flow fraction better than a linear or log-linear term (higher R^2 values). Moreover, a linear or log-linear relation in this term does not make sense, because that would imply (for a positive relation) that the breakdown flow would be maximized at a 100% flow fraction in the passing lane, which is clearly inefficient and therefore incorrect. As such, a quadratic term represents a more truthful relationship, because it has a value at which breakdown flow is maximized.

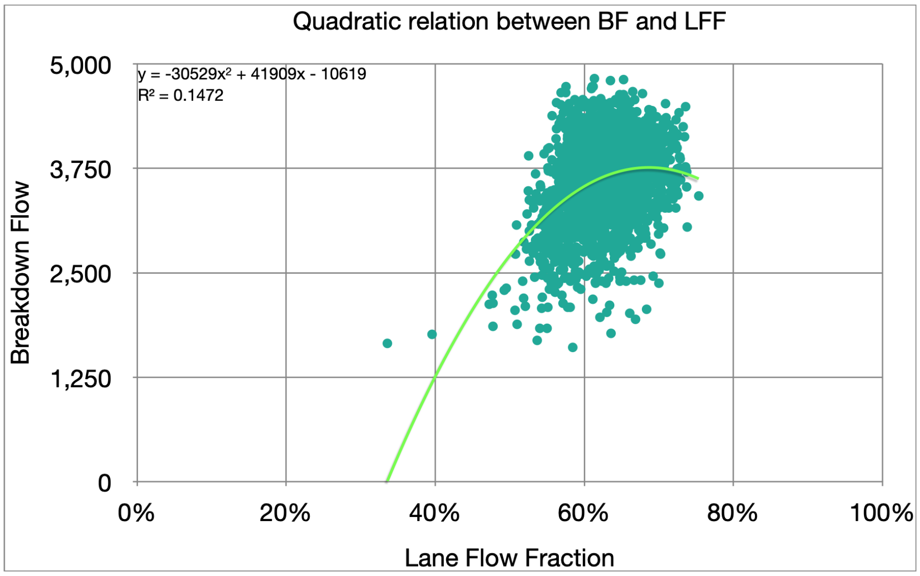

Figure 5.1. – Scatterplot with quadratic trendline for relation between breakdown flow and flow fraction in the passing lane

5.1.1. Summary Statistics

From Table 5.1 it can be seen that the number of breakdown flow measurements in this dataset is equal to 2294, with a mean of 3615 veh/hour, a minimum breakdown flow of 1608 and a maximum breakdown flow of 4824 veh/hour. From variable "V120" it can be seen that 50.26% of all measurements was observed under a limit of 120 km/h, making the dataset relatively balanced as there is an almost 50/50 division between the 120 and 130 km/h limit.

Only for some locations accurate truck traffic data from a detector loop in the vicinity of the study location was available. As such, the sample size of this variable is smaller than the rest of the dataset (1149), meaning that when this variable is taken into account in the regression, only breakdown flow measurements with matching truck traffic levels will be taken into account in the regression. With a mean truck traffic level of 12.66% in the sample and a standard deviation of 5.94%, most of the observed truck traffic levels are below 20%, though some large deviations are possible (as can be seen from the maximum observation of 0.6667%) because data is averaged over a relatively short period of 5 minutes in which a lot of variation can occur (large platoons of trucks will temporarily increase the truck traffic percentage to a very high level).

The lane flow fraction variable shows a mean flow fraction of 63% in the passing lane, which is consistent with findings in chapter 4 as well as a relatively low standard deviation (4%). A somewhat higher coefficient of variance applies to the quadratic term, LFF^2, which is derived from LFF, and the skew and kurtosis are relatively similar.

In Table 5.2 an overview is presented for the summary statistics of the location dummies in the sample. In this overview, the mean μ represents the fraction of measurements in the dataset that originate from that particular location. No dummy has been generated for the A2L Valkenswaard location, as only $M - 1$ location dummies may be included (see subsection 3.2.2. The fraction of measurements of this location is equal to one minus the sum of the other fractions, which is 15.04%.

Variable	N	μ	σ	Skewness	Kurtosis	Min	Max
BF	2294	3,615.10	505.71	-0.65	3.51	1608	4824
V120	2294	0.5026	0.5001	-0.01	1.00	0.0000	1.0000
TT	1149	0.1266	0.0594	1.25	9.10	0.0000	0.6667
LFF	2294	0.6313	0.0426	-0.44	4.24	0.3357	0.7535
LFF^2	2294	0.4003	0.0531	-0.15	3.34	0.1127	0.5678

Table 5.1. – Summary statistics for variables of eight-location sample

Location-Dummy	N	μ	Min	Max
A2R Valkenswaard	2294	0.0885	0	1
A27L Lexmond	2294	0.0414	0	1
A27R Lexmond	2294	0.1818	0	1
A58L Bavel	2294	0.2271	0	1
A58L Moergestel	2294	0.0558	0	1
A58R St. Annabosch	2294	0.1194	0	1
A58R Goirle	2294	0.1356	0	1

Table 5.2. – Summary statistics for location dummies of eight-location sample

Variables	BF	LFF	LFF^2	TT	V120
BF	1.000				
LFF	0.332	1.000			
LFF^2	0.320	0.998	1.000		
TT	-0.286	-0.079	-0.088	1.000	
V120	0.057	-0.076	-0.075	-0.149	1.000

Table 5.3. – Cross-correlation table for the evaluation of multi-collinearity in the 8 location sample

By using only the set of breakdown flow measurements, it is assumed that this set represents a full capacity distribution similar to how the distribution of breakdown flow measurements is derived in the Empirical Distribution Model (see subsection 3.2.1). It should be noted, as was mentioned before in chapter 3, that this distribution is not fully correct and should be expected to be lower than the real capacity distribution (as can be observed when comparing the values in Figure 5.2 with the values found in Table 4.1). In spite of these flaws, it is expected that a distribution such as in Figure 5.2 can still be a valuable proxy for assessing changes to the actual breakdown flow distribution that may have occurred as a consequence of a change in the speed limit. From Figure 5.2 it can already be seen that there are indications that the breakdown flow distribution under the 120 km/h limit may be somewhat higher than under the 130 km/h limit. Please note, however, that the data constituting this distribution originates from multiple locations, making the distribution sensitive to inter-location differences which may make it, potentially, biased.

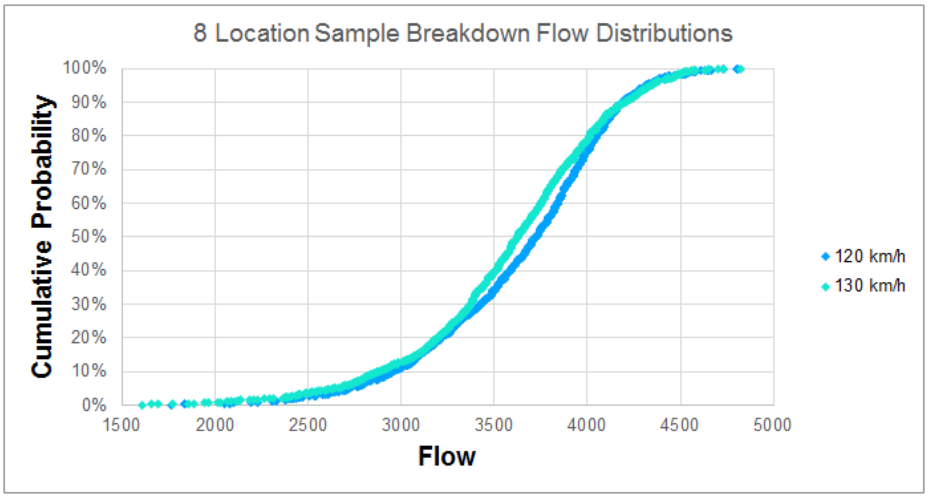

Figure 5.2. – Breakdown flow distributions as generated by means of the Empirical Distribution Method for the eight-location sample (location specific effects not included)

5.1.2. Regression Diagnostics

Before diving into the results, it is important to check whether the seven conditions for performing Least Squares regression have been met (see subsection 3.2.2).

1. **Linear Coefficients:** The variable which was expected to be quadratic of nature (LFF), has been converted to a quadratic term (LFF^2) thus linearizing the coefficient. For truck traffic there is no indication from literature that this should be a non-linear relation and it is expected that a linear coefficient would therefore be suitable. The remaining variables of V120 and location dummies are all "intercept shifters" (Stock and Watson, 2015) and can be expected to also be linear in coefficients.

2. **Error term Mean Zero (μ_u):** A T-test for a mean zero error has been performed for each regression in Table 5.4. It can be seen from the results of this test that the error term has a zero mean for most regressions with the exception of regressions 9, 10 and 11. This means that the estimators from those regressions are likely to be biased and should be interpreted with caution.

3. **Error term unrelated to variable of interest ($COV(u_{i,t}, V120_{i,t} = 0)$):** In Table 5.5 values of variable $V120_{i,t}$ have been plotted against the residuals $u_{i,t}$ generated by each regression. As can be seen from this table, the error term is in none of the regressions related to the variable of interest, $V120$. Additionally, the constant term is only significant in regressions 9 through 11, reflecting the fact that the error term is not equal to zero in those regressions, posing a risk for the unbiasedness of the estimators.

4. **Error term observations are uncorrelated with each other ($COV(u_i, u_j)$=0):** Breakdown flows are pulled from a time-series, potentially making them vulnerable to serial correlation. However, time intervals between moments of breakdown are sufficiently large to expect that auto-correlation will not be a problem, especially when location specific characteristics are accounted for.

5. **When present, heteroscedasticity should be accounted for:** The Breusch-Pagan (B.P.) test in Table 5.4 shows whether heteroscedasticity is present in the data. It can be seen that heteroscedasticity is induced in the model by the LFF and TT variables. This heteroscedasticity is reduced, though not solved, by the addition of location dummies. As such, robust regression is needed for generating the estimators because, without it, they become inefficient as a result of heteroscedasticity. For this reason robust regression has been applied in regressions 9 through 12.

6. **No multicollinearity should be present:** highly correlated variables should be avoided in each regression. In Table 5.3 correlations between relevant variables can be seen and it can be found that only the relation between LFF and LFF^2 is very high (0.998), which makes sense because LFF^2 is generated from LFF. This is, however, not a problem because we want to know the joint effect of these two variables, as they form a regression function of the type $Y = a * X^2 + b * X + c$. Additionally, the correlations with other independent variables are sufficiently low to avoid problems (< 0.70).

7. **The error term has to be normally distributed (optional):** The Shapiro-Francia test (which is a variation on the Shapiro-Wilk test) has been performed for each regression and it has been found that the error term is significantly different from a normal distribution. The problem with this non-normality is that the confidence intervals surrounding an estimator become less certain. However, when plotting histograms of the error terms it was found that they were at least approximately normally distributed. This implies that, as long as the confidence intervals are sufficiently far from zero, significance of estimators can still be judged to a reasonable extent.

5.1.3. Regression Results

The functional form for the regression in Table 5.4 is:

$$\hat{BF}_{i,t} = \hat{\beta}_0 + \hat{\beta}_1 * V120_{i,t} + \hat{\beta}_2 * LFF_{i,t} + \hat{\beta}_3 * LFF^2_{i,t} + \hat{\beta}_4 * TT_{i,t} + \sum_{i=1}^{I} (\hat{\gamma}_i * D_{i,t}) \quad (5.1)$$

Where i stands for the location from which the data is obtained and where t stands for each breakdown measurement over time at this particular location.

BF	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
V120	57.90***	85.71***	100.20***	108.30***	72.35***	87.93***	69.11**	72.82***	68.95***	85.22***	36.46	59.99**
	(21.09)	(19.49)	(26.31)	(25.11)	(19.47)	(18.66)	(26.98)	(24.75)	(18.46)	(18.41)	(23.71)	(23.88)
LFF		42,134.46***		38,424.91***		33,397.08***		31,148.57***		37,085.64***		43,778.99***
		(3,816.60)		(4,067.32)		(3,986.01)		(4,045.84)		(3,931.05)		(5,348.43)
LFF^2		-30,648.60***		-28,396.77***		-23,874.63***		-20,857.29***		-27,096.29***		-31,015.20***
		(3,063.28)		(3,222.46)		(3,234.00)		(3,245.64)		(3,189.42)		(4,216.91)
TT			-2,092.37***	-2,173.21***			-2,305.17***	-2,733.24***			-2,766.57***	-2,987.32***
			(220.82)	(211.33)			(262.38)	(243.02)			(230.64)	(234.63)
Constant	3,586.00***	-10,756.93***	3,920.24***	-9,020.67***	3,575.07***	-7,904.19***	3,956.95***	-7,257.46***	3,618.45***	-8,935.19***	4,161.23***	-10,965.99***
	(14.95)	(1,187.75)	(35.59)	(1,284.33)	(27.30)	(1,233.45)	(61.57)	(1,264.37)	(25.89)	(1,216.44)	(54.12)	(1,701.59)
Location Dummies	NO	NO	NO	NO	YES	YES	YES	YES	YES	YES	YES	YES
Robust Regression	NO	NO	NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Observations	2,294	2,294	1,149	1,149	2,294	2,294	1,149	1,149	2,294	2,294	1,149	1,148
# Parameters	2	4	3	5	9	11	10	12	9	11	10	12
R^2	0.0033	0.1543	0.0931	0.1925	0.1606	0.2334	0.1215	0.2640	0.1579	0.1938	0.0836	0.1398
B.P. test (χ^2)	1.91	92.86***	70.76***	137.34***	1.19	26.69***	110.95***	192.97***	N.A.	N.A.	N.A.	N.A.
S.F. test (Z)	8.55***	5.75***	8.53***	6.72***	9.00***	6.85***	8.12***	4.61***	9.41***	7.45***	8.61***	5.70***
T-test ($\mu_u = 0$)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-4.18***	-2.56**	-3.29***	-1.11
Implied Optimal Lane Flow Fraction	N.A.	0.6874	N.A.	0.6766	N.A.	0.6994	N.A.	0.7467	N.A.	0.6843	N.A.	0.7058

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.4. – Regression results for Breakdown Flow dependence at eight locations

The first thing to note is that the estimates for the V120 variable are all positive and in the range of 36 to 108, meaning that the average breakdown flow is about 36 to 108 vehicles per hour higher than under the 130 km/h limit, when keeping other factors constant. Additionally, with exception of regression 9 (for which the estimator is biased) all of the positive differences are significant at at least the 5% level. It does not matter much for significance whether location dummies are taken into account and/or whether the regression is executed in a robust manner. Consequently, a case is made for a higher mean breakdown flow under the 120 km/h limit than under the 130 km/h limit. This entails that the null hypothesis of hypothesis 4a can rejected in favor of the alternative hypothesis. As such, in this sample, the mean breakdown flow under the 120 km/h limit is higher than under the 130 km/h, giving an indication that capacity could also be higher under the 120 km/h limit.

Additionally, it can be seen that variables LFF, LFF^2 and TT are also significant at the 1% level for all regressions in which they occur, indicating that their effects are relevant and should be taken into account. For the truck traffic variable (TT) its interpretation is straightforward. For every percentage-point change (0.01) in TT, a decrease of 20 to 32 veh/hour in breakdown flows can be found, which is consistent with what one would expect. For variable LFF and LFF^2 the interpretation of the coefficients is less straight-forward, as the relation is quadratic. Technically, the estimator indicates that the average breakdown flow will follow a parabolic shape in relation to the lane-flow fraction.

To illustrate the meaning of these results in a clear manner, one can assume a truck traffic level of 12% and a speed limit of 120 km/h. Using the results from regression 12, the equation then becomes:

$$BF = -10,965.99 + 59.99 * [1] + 43,778.99 * LFF - 31,015.20 * LFF^2 + -2,987.32 * [0.12] \quad (5.2)$$

Taking the derivative of this equation with respect to LFF and setting this derivative equal to zero yields a passing lane flow fraction which maximizes the breakdown flow, which entails, in this case, $LFF = 0.7058$. For which the hypothesized breakdown flow is approximately 4184 vehicles per hour, which is the value for which, under these conditions, the flow is maximized. Under similar conditions, with a speed limit of 130 km/h. The model predicts a maximum breakdown flow equal to 4124 vehicles per hour. Please note that these results apply for the A2L Valkenswaard location only (as all other location dummies are set at zero) and that one has to correct for the location dummy coefficients (which are not shown in Table 5.4 to improve readibility) to approximate the "maximum" breakdown flow for other locations.

Because the regression function contains a quadratic component (LFF^2) an optimal value of the lane flow fraction for which the breakdown flow is maximized, can be calculated for each regression in which the LFF variable was included. In Table 5.4 these numbers have been calculated by taking the derivative of the regression function with respect to LFF and setting this derivative equal to zero. The range of "optimal" Lane Flow Fraction values that was obtained is relatively small, with all values positioned somewhere between 0.67 and 0.75 (see Table 5.4).

These values seem to be relatively high for achieving optimal flow. One should note, however, that in the process of generating the LFF variable, direct vehicle counts were used in which no correction for Passenger Car Equivalent values was made. Since most trucks (which represent more PCE) are driving in the shoulder lane and the flow in the passing lane consists predominantly of passenger vehicles, the LFF variable overstates the actual proportion of traffic (in PCE) that should be in the passing lane to achieve optimal flow. It is therefore expected by the author that the "optimal" LFF value of around 70%, which was found in this model, is more likely to be somewhere around 60% when PCE values are accounted for. Nonetheless, the results of this model make a clear case for the existence of an optimal flow distribution among the lanes that maximizes the (breakdown) flow on a freeway facility.

$\hat{u}_{i,t}$	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
V120	-0.00	0.00	0.00	0.00	0.00	-0.00	0.00	-0.00	5.72	2.77	33.26	8.53
	(21.09)	(19.42)	(26.00)	(24.54)	(19.35)	(18.49)	(25.59)	(23.42)	(19.41)	(18.54)	(25.73)	(23.57)
Constant	0.00	-0.00	-0.00	-0.00	-0.00	0.00	-0.00	0.00	-43.45***	-25.15*	-60.10***	-17.57
	(14.95)	(13.77)	(19.04)	(17.97)	(13.72)	(13.11)	(18.74)	(17.15)	(13.76)	(13.14)	(18.84)	(17.26)
Observations	2,294	2,294	1,149	1,149	2,294	2,294	1,149	1,149	2,294	2,294	1,149	1,149
R^2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.5. – Regression for the evaluation of error term dependence of variable V120 on error term $\hat{u}_{i,t}$

5.2. Analysis of Breakdown Flows under speed limits of 100, 120 and 130 km/h

In this section, hypothesis 4b will be tested, which states:

- H_0: *The speed limit variables of the seventeen-location sample have an insignificant effect, when controlling for location specific factors and other relevant variables.*
- H_1: *The speed limit variables of the seventeen-location sample have a significant effect, when controlling for location specific factors and other relevant variables.*

5.2.1. Summary Statistics

In Table 5.6 the summary statistics of the seventeen-location sample have been presented. A total sample size of 5998 breakdown flow measurements is available, for which no missing data are present. As can be seen from Table 5.6), the mean breakdown flow in the sample is equal to 3446 vehicles per hour, which is somewhat lower than the mean breakdown flow in the eight-location sample (see Table 5.1). The standard deviation is relatively similar at 528 vehicles per hour and the skew and the kurtosis are also similar, meaning that there are no large differences for the (average) distribution of breakdown flows under the 17 location sample compared to the 8 location sample, except for the central position.

	N	Mean	SD	Skew	Kurtosis	Min	Max
BF	5998	3445.68	527.68	-0.48	3.33	1044	4944
V120	5998	0.5458	0.4979	-0.18	1.03	0	1
V100	5998	0.1400	0.3471	2.07	5.30	0	1
LFF	5998	0.6191	0.0501	-0.76	5.14	0.3403	0.9006
LFF^2	5998	0.3858	0.0602	-0.33	4.31	0.1158	0.8110

Table 5.6. – Summary Statistics for variables of seventeen-location sample

Three different speed limits are present in this sample. 54.6% of the measurements have been obtained under a speed limit of 120 km/h, while 14% of measurements have been obtained under a speed limit of 100 km/h and the remaining 31.4% of measurements have been obtained under a limit of 130 km/h. As was displayed in chapter 3 in Table 3.3, eleven locations have data from 100, 120 and 130 km/h limits and the remaining 6 locations only have data from the 100 km/h and 120 km/h limit. The 100 km/h limit observations have all been observed in 2020, the 130 km/h limit observations have all been observed in 2019 and the 120 km/h limit observations have been observed in different years, which were always one year prior to a speed limit change (2012, 2015, 2018 or 2019; see Table 3.3).

As such, this sample is broader than the eight-location sample presented in Table 5.1, in the sense that more locations are included (see Table 5.7), more breakdown flow measurements are available and more speed limits have been observed. Additionally, for some locations in the eight-location sample, the data for the 130 km/h limit are different, as they have been observed in 2019 instead of, for instance 2013 or 2016 (see Table 3.2). This presents an opportunity for testing whether similar results can be obtained in this regression as in the regression that was presented in Table 5.4. One limitation of this sample is that no truck traffic data was gathered. This was done for reasons of time constraints as looking for and selecting locations for reliable truck traffic data is a time consuming process.

Lane flow fractions in variable LFF take on an average value of 61.91% in this dataset, which is close to the 63.13% that was found in the eight-location sample (see Table 5.1). With a standard deviation of 0.05 and similar skew and kurtosis, the LFF distributions in both samples can be expected to be reasonably similar.

	N	Mean	Min	Max
A2R Valkenswaard	5998	0.0453	0	1
A15L Sliedrecht-Oost	5998	0.0665	0	1
A15R Hardinxveld	5998	0.0760	0	1
A15R Sliedrecht-Oost	5998	0.0448	0	1
A15R Sliedrecht-West	5998	0.0810	0	1
A20R Nieuwerkerk	5998	0.0415	0	1
A27L Lexmond	5998	0.0225	0	1
A27R Lexmond	5998	0.0715	0	1
A27R Oosterhout	5998	0.0120	0	1
A58R St. Annabosch	5998	0.1085	0	1
A58R Gilze	5998	0.0222	0	1
A58R Goirle	5998	0.0664	0	1
A58R Moergestel	5998	0.0305	0	1
A58R Oirschot	5998	0.0700	0	1
A58R Ulven	5998	0.1110	0	1
A67R Leenderheide	5998	0.0420	0	1

Table 5.7. – Summary Statistics for location dummies of seventeen-location sample

When assessing the average breakdown flow distributions (ignoring that the measurements are from different locations), it can be seen that an interesting pattern emerges (see Figure 5.3) where the 120 km/h limit seems to follow a breakdown flow distribution that is much higher than the 100 and 130 km/h limit distributions. Note that this could simply be due to noise in the data caused by the inclusion of different locations in one distribution. For instance, it may be the case that locations which did not experience a 130 km/h limit may have had higher breakdown flows to start with. This is an effect that will be corrected for when taking into account location dummies in Table 5.10. Moreover, it is not certain whether the breakdown flow distribution under the 100 km/h limit is truly representative of the actual distribution under "regular" conditions. All observations of the 100 km/h limit have been obtained during the COVID-19 lockdown period, which means that the driving behavior may be more typical of a weekend day, which has been found to cause lower capacities than driving behavior on a "regular" work day (Calvert, 2016, p.57).

5.2.2. Regression Diagnostics

Again, before diving into the results, it is important to check whether the seven conditions for performing Least Squares regression have been met (see subsection 3.2.2).

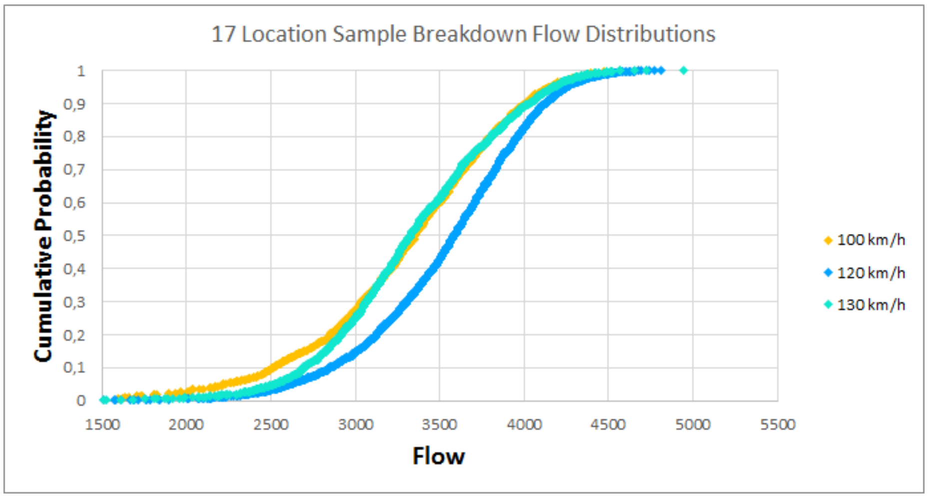

Figure 5.3. – Breakdown flow distributions as generated by means of the Empirical Distribution Method for the 17 location sample (location specific effects not included)

1. **Linear Coefficients:** Again, the variable which was expected to be quadratic of nature (LFF), has been converted to a quadratic term (LFF^2) thus linearizing the coefficient. For truck traffic there is no indication from literature that this should be a non-linear relation and it is expected that a linear coefficient would therefore be suitable. The remaining variables of V120, V100 and location dummies are all "intercept shifters" (Stock and Watson, 2015) and can be expected to also be linear in coefficients.

2. **Error term Mean Zero (μ_u):** A T-test for a mean zero error has been performed for each regression in Table 5.10. It can be seen from the results of this test that the error term has a zero mean for all regressions.

3. **Error term unrelated to variable of interest ($COV(u_{i,t}, V120_{i,t} = 0)$):** In Table 5.11 values of variables $V120_{i,t}$ and $V100_{i,t}$ have been plotted against the residuals $u_{i,t}$ generated by each regression. As can be seen from this table, the error term is in none of the regressions related to either one of these variables of interest. Additionally, the constant term is never significant and R^2 values are zero for all regressions.

4. **Error term observations are uncorrelated with each other ($COV(u_i, u_j)$=0):** Breakdown flows are pulled from a time-series, potentially making them vulnerable to serial correlation. However, time intervals between moments of breakdown are sufficiently large to expect that auto-correlation will not be a problem, especially when location specific characteristics are accounted for.

5. **When present, heteroscedasticity should be accounted for:** The Breusch-Pagan (B.P.) test in Table 5.10 shows whether heteroscedasticity is present in the data. It can be seen that heteroscedasticity in the model is the result of not including location dummies. When these location dummies are included, the Breusch-Pagan test becomes insignifcant and it is therefore that we can assume that the data in regressions

3 and 4 is homoscedastic and estimators are efficient. Consequently, robust regression is not necessary for this sample.

6. **No multicollinearity should be present:** highly correlated variables should be avoided in each regression. In Table 5.9, correlations between relevant variables can be seen and it can be found that only the relation between LFF and LFF^2 is very high (0.997), which makes sense because LFF^2 is generated from LFF. This is, however, not a problem because we want to know the joint effect of these two variables, as they form a regression function of the type $Y = a * X^2 + b * X + c$. Additionally, the correlations with other independent variables are sufficiently low to avoid problems (< 0.70).

7. **The error term has to be normally distributed (optional):** The Shapiro-Francia test (which is a variation on the Shapiro-Wilk test) has been performed for each regression and it has been found that the error term is significantly different from a normal distribution for all regressions. The problem with this non-normality is that the confidence intervals surrounding an estimator become less certain. However, when plotting histograms of the error terms it was found that they were at least approximately normally distributed. This implies that, as long as the confidence intervals are sufficiently far from zero, significance of estimators can still be judged to a reasonable extent.

Table 5.8. – Cross-correlation table

Variables	V120	V100	LFF	LFF2
V120	1.000			
V100	-0.442	1.000		
LFF	0.128	-0.270	1.000	
LFF2	0.122	-0.260	0.997	1.000

Table 5.9. – Cross-correlation table for the evaluation of multi-collinearity in the 17 location sample

5.2.3. Regression Results

The functional form for the regression in Table 5.10 is:

$$\hat{BF}_{i,t} = \hat{\beta}_0 + \hat{\beta}_1 * V120_{i,t} + \hat{\beta}_2 * V100_{i,t} + \hat{\beta}_3 * LFF_{i,t} + \hat{\beta}_4 * LFF^2_{i,t} + \sum_{i=1}^{I}(\hat{\gamma}_i * D_{i,t}) \quad (5.3)$$

Where i stands for the location from which the data is obtained and where t stands for each breakdown measurement over time at this particular location.

It can be seen from Table 5.10 that the mean breakdown flows for the 120 km/h limit are significantly higher than the mean breakdown flows for the 130 km/h limit in all regressions, further corroborating the findings from Table 5.4 that the mean breakdown flow seems to

BF	(1)	(2)	(3)	(4)
V120	189.90*** (14.98)	180.64*** (13.23)	81.64*** (14.94)	125.84*** (13.64)
V100	-39.78* (21.49)	168.48*** (19.64)	-97.25*** (20.63)	121.63*** (19.87)
LFF		33,215.19*** (1,518.56)		34,998.48*** (1,505.21)
LFF2		-23,883.93*** (1,257.86)		-25,429.45*** (1,269.17)
Constant	3,347.59*** (11.93)	-8,025.99*** (458.49)	3,394.43*** (21.22)	-8,487.48*** (450.75)
Location Dummies	NO	NO	YES	YES
Robust Regression	NO	NO	NO	NO
Observations	5,998	5,998	5,998	5,998
# Parameters	3	5	19	21
R^2	0.0369	0.2492	0.2674	0.4001
B.P. test (χ^2)	18.96***	41.70***	1.56	0.81
S.F. test (Z)	9.47***	4.44***	11.25***	8.89***
T-test (μ_u)	0.00	0.00	0.00	0.00
Implied Optimal Lane Flow Fraction	N.A.	0.6953	N.A.	0.6881

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.10. – Regression results for Breakdown Flow dependence at seventeen locations

$\hat{u}_{i,t}$	(1)	(2)	(3)	(4)
V120	-0.00	0.00	0.00	0.00
	(14.98)	(13.22)	(13.06)	(11.82)
V100	-0.00	0.00	-0.00	0.00
	(21.49)	(18.97)	(18.74)	(16.96)
Constant	0.00	-0.00	-0.00	-0.00
	(11.93)	(10.54)	(10.41)	(9.42)
Observations	5,998	5,998	5,998	5,998
R^2	0.00	0.00	0.00	0.00

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.11. – Regression for the evaluation of error term dependence of variable V120 and V100 on error term $\hat{u}_{i,t}$

be higher. This is both the case for regressions in which location dummies are accounted for (2 and 4) and for regressions in which they are not (1 and 3).

For the 100 km/h limit, the sign of the effect seems to switch depending on whether the Lane Flow Fraction is included. It has already been found that the Lane Flow Fraction is a very important variable in the determination of the height of the breakdown flow (see Table 5.4) and it is proven again by the R^2 values in the regression in Table 5.10. Whenever the Lane Flow Fraction is added to the analysis, the R^2 value increases by a lot. As such, neglecting this is likely to cause omitted variable bias, in which the estimators of V120 and V100 will become correlated with the error term (see subsection 3.2.2) and will be biased.

Note that in Table 5.9 the relation between V120 and LFF is positive and relatively weak (+0.128) while the relation between V100 and LFF is positive and stronger (-0.442). Because of this, the negative effect of V100 in regressions 1 and 3 is multiplied by the negative correlation with LFF when the LFF variable is included in regressions 2 and 4, which causes the sign of the estimator to switch, because the effects of LFF and V100 become disentangled. The fact that the heteroscedasticity disappears from the sample and that the R^2 is relatively high in the fourth regression, shows that the model in this regression is most likely to be the one closest to reality. When accounting for the effects of the lane distribution variable, the mean breakdown flows under both the 120 and 100 km/h limit seem to be higher than the mean breakdown flows under the 130 km/h limit.

Again, when calculating the "optimal" Lane Flow Fraction by taking the derivative of the regression function with respect to variable LFF and setting this derivative equal to zero, "optimal" Lane Flow Fractions are found for these regressions (around 70%), which are similar to the values found in Table 5.4, thus strengthening the case for the existence of an optimal lane flow distribution.

5.3. Relation Between the Speed Limit and the Flow Fraction in the Passing Lane

In this section, hypothesis 5 will be tested, which states:

- H_0: *The speed limit does not have a significant effect on the lane flow distribution, when controlling for location specific effects.*
- H_1: *The speed limit has a significant effect on the lane flow distribution, when controlling for location specific effects.*

5.3.1. Regression Diagnostics

To test this hypothesis, both the eight-location and seventeen-location samples will be analyzed. As many aspects of these samples have already been discussed, only issues that are relevant to this particular regression will be discussed in this subsection.

For the 8-location sample (regressions 1 and 2) heteroscedasticity has not been found to pose a large problem, as no value of the Breusch-Pagan test takes on a value that is significant at the 5% level. For this reason, no robust estimation procedure was necessary for this sample. For the 17-location sample, on the other hand (regressions 3 and 4) heteroscedasticity was found to be present with a significance level of less than 1%. Therefore, in regressions 5 and 6, robust regression has been applied to account for this heteroscedasticity.

It can be seen from Table 5.13 that for regressions 5 and 6, the mean of the error term is non-zero, which is caused by the robust estimation procedure that has been applied here, as such, one has to take account of the fact that estimators in these regressions are likely to be biased. Additionally, the assumption that the error term is unrelated to the independent variables of interest, is also violated, because the coefficient for V100 is found to be significantly different from zero for regressions 5 and 6 in Table 5.13 as well as the coefficient for V120 in regression 5. For these reasons, the results of regressions 5 and 6 should be viewed with caution.

Lastly, the error term is found to be non-normally distributed, which implies that confidence intervals of estimators are less reliable.

5.3.2. Regression Results

The following functional form has been applied for the regression in Table 5.12:

$$L\hat{F}F_{i,t} = \hat{\delta}_0 + \hat{\delta}_1 * V120_{i,t} + \hat{\delta}_2 * V100_{i,t} + \sum_{i=1}^{I}(\hat{\gamma}_i * D_{i,t}) \tag{5.4}$$

Where i stands for the location from which the data is obtained and where t stands for each breakdown measurement over time at this particular location.

In regression 1 of Table 5.12 the eight-location sample is analyzed and only the 120 km/h limit is used as an explanatory variable. It is found that passing lane use is reduced by

Sample	8-loc.	8-loc.	17-loc.	17-loc.	17-loc.	17-loc.
LFF	(1)	(2)	(3)	(4)	(5)	(6)
V120	-0.0065***	-0.0043***	0.0011	-0.0104***	0.0050***	-0.0095***
	(0.0018)	(0.0012)	(0.0014)	(0.0011)	(0.0013)	(0.0009)
V100			-0.0382***	-0.0408***	-0.0302***	-0.0359***
			(0.0020)	(0.0015)	(0.0019)	(0.0013)
Constant	0.6345***	0.6126***	0.6239***	0.6158***	0.6230***	0.6170***
	(0.0013)	(0.0017)	(0.0011)	(0.0015)	(0.0011)	(0.0013)
Location Dummies	NO	YES	NO	YES	NO	YES
Robust Regression	NO	NO	NO	NO	YES	YES
Observations	2,294	2,294	5,998	5,998	5,998	5,998
R-squared	0.0058	0.5354	0.0729	0.5926	0.0606	0.6503
B.P. test (χ^2)	2.82*	2.60	255.42***	65.19***	N.A.	N.A.
S.F. test (Z)	7.03***	10.70***	11.11***	14.67***	11.29***	14.85***
T-Test ($\mu_u = 0$)	0.00	0.00	0.00	0.00	-3.78***	-5.26***

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.12. – Regressions of Speed Limit on Lane Flow Fraction

0.65 percentage points at 1% significance when location specific effects are not included, which reduces to a (1% significant) 0.43 percentage point reduction when location effects are included. For the seventeen-location samples, the third and fifth regression estimates are very likely to be biased as the true effect effect is masked by the effects from the many different location specific factors in this sample.

By including location dummies in regressions 4 and 6, location specific effects are explicitly accounted for and, as a result, the estimators become significant again and are relatively close to each other. Regressions four and six are practically the same regressions, with the difference that the 6th regression includes controls for robustness, which reduces the significance of the estimators by a bit. Overall, it can be seen that the effect from the 120 km/h limit is most likely to be significant, with a mean reduction of passing lane use between 0.4 and 1.0 percentage points.

For the 100 km/h limit the results are even more convincing, as a reduction of passing lane use of 3 to 4 percentage points is found for this variable. Moreover, all estimators are significant at the 1% level, regardless of whether location specific effects are controlled for. When looking at the R^2 values a clear pattern is visible. When location effects are not included, the R^2 values are relatively low, meaning that a lot of unexplained variation is still present in the data. When location effects are included, however, R^2 values immediately shoot up to values of around 50% to 65%, indicating that most of the variation in the data can be explained by the model. Though it is true that adding extra variables to a regression will always increase the R^2 value (Stock and Watson, 2015), the increase is large enough to be certain that the model is significantly improved by the inclusion of the location specific effects.

$\hat{u}_{i,t}$	(1)	(2)	(3)	(4)	(5)	(6)
V120	-0.0000	0.0000	-0.0000	0.0000	-0.0039***	-0.0004
	(0.0018)	(0.0012)	(0.0014)	(0.0009)	(0.0014)	(0.0009)
V100			-0.0000	-0.0000	-0.0081***	-0.0044***
			(0.0020)	(0.0013)	(0.0020)	(0.0013)
Constant	0.0000	-0.0000	0.0000	-0.0000	0.0009	-0.0014*
	(0.0013)	(0.0009)	(0.0011)	(0.0007)	(0.0011)	(0.0007)
Observations	2,294	2,294	5,998	5,998	5,998	5,998
R-squared	0.0000	0.0000	0.0000	0.0000	0.0029	0.0021

Standard errors in parentheses
*** p<0.01, ** p<0.05, * p<0.1

Table 5.13. – Regressions of independent variables on error term $\hat{u}_{i,t}$ for regressions 1 through 6

In conclusion, the effects from both the 100 and 120 km/h limit on the lane flow distribution are very pronounced and mostly significant. As such, the null hypothesis of hypothesis 5 is rejected in favor of the alternative hypothesis and it is concluded that the speed limit seems to affect the lane flow distribution significantly.

5.4. Discussion

Based on the results in sections 5.1, 5.2 and 5.3, it can be stated with reasonable certainty that mean breakdown flows seem to be highest under the 120 km/h limit, relatively high under the 100 km/h limit and relatively low under the 130 km/h limit. It seems a little bit strange that the mean breakdown flows under the 120 km/h limit are higher than breakdown flows under either the 100 or 130 km/h limit, especially since it was found in Appendix C that speed choice behavior under the 120 and 130 km/h limit was relatively similar.

One would expect, if a relation between mean breakdown flows and speed limits would exist, that the ranking of mean breakdown flows would either be $\mu_{BF_{100}} \leq \mu_{BF_{120}} \leq \mu_{BF_{130}}$ or $\mu_{BF_{100}} \geq \mu_{BF_{120}} \geq \mu_{BF_{130}}$. There are several reasons why such an ordering was not encountered in the results of this chapter:

First of all, it is very likely to be the case that the 100 km/h breakdown flow data is not representative of "regular conditions" due to the fact that data were obtained during the COVID-19 lockdown, a period in which less traffic was in the road to begin with, which has several implications for the data:

Firstly, less freeway congestion was present in this measurement period than in the same period of the year in other years, leading to only a small fraction of "100 km/h limit" measurements in the sample (approximately 14% against 55% "120 km/h limit" measurements and 31% "130 km/h limit" measurements).

Additionally, the traffic composition is likely to be different. Truck traffic levels were not explicitly accounted for in the 17 location sample and it may well be the case that the traffic stream had larger proportions of truck traffic during the COVID lockdown period, due to the fact that "economically important" traffic such as trucks would keep driving, while many passenger cars were not on the road because their "regular drivers" were working at home. If this is the case, truck traffic percentages would have been much higher, which explains why the breakdown flows under the 100 km/h limit are so low. Additionally, it was discussed earlier that driving behavior in general may have become more like "weekend" driving in the COVID lockdown, which implies lower capacities to begin with.

Secondly, it should be noted that the breakdown flow distributions, as presented in this chapter, are not fully representative of the true capacity distribution, because only category B measurements are taken into account for the generation of the distribution. Considering that, for instance, the true capacity distribution curve of the 130 km/h limit may be characterized by a larger variance (with more observations in the tails of the distribution) it would explain why a larger share of relatively low breakdown flows are measured under this limit than under the 120 km/h limit.

In the distribution estimation process of the product limit method, these lower value measurements would not be of a large influence on the distribution, because they would also be compared with a lot of category F measurements that are (much) higher than these values. However, because no category F measurements are taken into account in the distributions that are used in this chapter, the lower breakdown flow values have a stronger (negative) influence on the central position of the distribution.

A third explanation is that, despite not affecting speed choice behavior very much, the 120 km/h limit does have an effect on the lane in which people prefer to drive under conditions of high flow. In Table 5.12 it has been found that a strong relation was present between the speed limit and the utilization rate of the passing lane. Even for the 120 km/h limit there was a difference with the 130 km/h limit that was deemed to be significant. Moreover, it has been proposed in this thesis that it is very likely that there must be some "optimal" lane flow distribution for which the breakdown flow is maximized.

In Table 5.4 and Table 5.10 evidence was found for the existence of a quadratic relation between the lane flow fraction on the passing lane and the height of the flow at which traffic breakdown occurs. Given that a strong relation was found between the speed limit and the utilization rate of the passing lane, it could very well be the case that the mean breakdown flow results of the 120 km/h limit are higher than the results of the 130 km/h limit and the 100 km/h limit, because the "average" lane flow fraction under the 120 km/h is closest to the optimal lane flow distribution of traffic, i.e. closer to the "top of the hill" of the quadratic relation. This may indicate that an optimal speed limit in two lane freeway on-ramp areas could, perhaps, be somewhere between 100 km/h and 120 km/h, as both of these seem limits seem to be characterized by relatively high breakdown flows.

5.5. Summary

In section 5.1 it was found that the mean breakdown flows under the 120 km/h limit were significantly higher than under the 130 km/h limit and that this difference was in the range of 60 to 110 vehicles per hour more. It was found that when location specific effects were accounted for that a large share of the variation was taken away and that the results would stay relatively the same. Also, when truck traffic and lane flow distribution effects were taken into account, the mean breakdown flows would still be significantly higher under the 120 km/h limit. Moreover, this pattern persisted in the regression in section 5.2, where mean breakdown flows were 80 to 190 vehicles per hour higher and significant in all regressions at at least the 1% level. For the 100 km/h limit it has been found that, when location specific factors as well as lane flow distribution effects are taken into account, the mean breakdown flow higher than under the 130 km/h limit, at a level of 1% significance, and somewhat lower than under the 120 km/h limit.

A statistically significant, positive relation, between the height of the speed limit and the fraction of flow in the passing lane has been found, which entails that the tendency to drive in the passing lane increases when the speed limit is higher. This tendency was consistent and significant across both samples and all regressions. In addition to this, it was found that the lane flow fraction could, in its turn, be related to the level at which breakdown flow occurs and it was found that a quadratic relation between the two factors generated estimates with the highest level of fit. It was argued that a quadratic relation for this variable is preferred over a linear or log-linear relation, because each of these two relations would imply the flow to be maximized when 100% of the flow is in the passing lane, which is clearly inefficient. Moreover, one would expect that some optimal distribution of traffic over the lanes must exist, which is why a quadratic relation would make sense, because it has a value at which the breakdown flow is maximized.

On the basis of results in the regressions in Table 5.4 and Table 5.10 an "optimal" passing lane flow fraction of 70% was estimated. This value is, however, not corrected for Passenger Car Equivalents, which would more likely lead to a 60/40 division of flow over the passing and shoulder lane respectively.

6. Discussion

6.1. Major Findings

Throughout this thesis it has been proven to be difficult to disentangle the effects from a speed limit change on capacity from other relevant effects. Through the application of the Product Limit Method, it has been achieved to generate (incomplete) capacity distributions for eight locations in the period surrounding a speed limit change from 120 to 130 km/h. Capacity distributions were derived for the complete roadway as well as for the passing lane and shoulder lane separately. When comparing differences between capacity distributions under the different limits of 120 and 130 km/h per location, it was found that no uniform direction of a capacity effect could be distinguished. Even when including the effect of changes in truck traffic levels between measurement periods, a general direction of the effect was still not visible.

In an effort to find a more general trend in the data, the full set of breakdown flow measurements for the eight locations surrounding the 120 to 130 km/h speed limit change, as well as a larger set, including measurements from a total of seventeen locations with speed limit changes from 120 to 130, 130 to 100 km/h and 120 to 100 km/h, have been analyzed by means of Least Squares regression techniques, so that location specific factors could be explicitly accounted for. From the analysis of these breakdown flow distributions, it was found that the 120 km/h limit did have a consistent and statistically significant positive effect on the mean breakdown flow, when compared to the 130 km/h limit. This was true for both samples and the difference with the 130 km/h limit was in the range of 60 to 190 vehicles per hour more. Additionally, the effects for the 100 km/h limit were also tested, which were slightly lower than the results for the 120 km/h limit, but higher than the mean breakdown flow results under the 130 km/h limit, when lane flow distribution effects were accounted for.

In both samples it was found that a significant positive relation was present between the proportion of flow in the passing lane and the height of the speed limit. It was estimated that the proportion of flow in the passing lane was approximately 62% under the 130 km/h limit, with a 0.4 to 1.0 percentage point reduction for the 120 km/h limit and a 3 to 4 percentage point reduction under the 100 km/h limit. When taking account of location specific factors, all estimators were significant at the 1% level and the R^2 values of the model were in the range of 0.54 to 0.65, meaning that the majority of the variation in the data can be explained by the model.

Moreover, it was found that, in both samples, the proportion of flow in the passing lane was strongly related to the height of the flow at which breakdown occurs and that a quadratic function of the type $Y = a * X^2 + b * X + c$ best represented this relation (producing the highest R^2 and making sense theoretically).

Based on these findings it is proposed that the speed limit does affect the level at which the breakdown flow occurs and that this is principally caused by changes in lane choice behavior, which alter the lane flow distribution which, subsequently, affects the breakdown flow. Assuming that the relation between the lane flow distribution and the breakdown flow is of a non-linear quadratic nature, there must be some "optimal" lane flow distribution value for which the breakdown flow is likely to be maximized. Perhaps by calculating what this (hypothetically) optimal lane flow distribution is for a given freeway layout, speed limits could be adjusted in such a way that an ideal lane flow distribution is induced.

6.2. Findings in relation to literature

6.2.1. Relation between the speed limit, the lane flow distribution and the mean breakdown flow

In this study a significant relation was found between the speed limit and the fraction of flow on the passing lane. Similar positive relations have been found in other studies, such as a study by (Knoop *et al.*, 2010) on the influence of variable speed limits on the lane distribution of a three lane freeway near an on-ramp. In this study it was found that a significantly lower fraction of traffic used the shoulder lane near the on-ramp, which may also be one of the reasons why such a relatively high value of a 70% "optimal" passing lane flow fraction has been found. Additionally, this implies that the location of the detector (see chapter 3 for the detector locations) may influence the lane flow distributions that are measured at the moment of breakdown. These effects are however accounted for in the regressions of chapter 5, when location specific effects are included. Also in studies by Tool *et al.* (2006) Duret *et al.* (2012) and Soriguera *et al.* (2017) the positive relation between the height of the speed limit and passing lane use - or, equivalently, the negative relation with shoulder lane use - has been found. The findings in this thesis add to this body of knowledge by exactly quantifying the lane flow distribution effects of different speed limits in on-ramp areas of two-lane freeways.

Subsequently, results for the relation between the level of flow and the lane flow distribution, confirms the proposed relation between the lane flow distribution and total flow on a two lane freeway, as presented in the Dutch Freeway Design Guidelines (Rijkswaterstaat, 2017, p.34). Though a relation with a log-linear shape is presented here, it is found that it may be the case that a quadratic term poses a better relation between these two terms. Moreover, given that the speed limit influences the lane flow distribution, it may be the case that there is a (location-specific) speed limit, for which a level of lane flow distribution is reached, at which the breakdown flow is maximized. In this thesis the most "optimal" speed limit that has been found for maximizing the mean breakdown flow was 120 km/h (see chapter 5).

6.2.2. Behavioral theory of Slugs and Rabbits

In this thesis evidence was found for the existence of a two pipe regime as argued by (Daganzo, 2002a). It was found that the passing lane and shoulder lane have very different dynamics section 4.6, in which traffic breakdown seems to be triggered by over-saturation in the passing lane. For most breakdown flow measurements it could be seen that the flow in the passing lane would decrease in the next minute following the moment of breakdown, while it would increase in the shoulder lane, thus presenting evidence for the collapse of the two-pipe regime into the one-pipe regime (Daganzo, 2002a). Additionally, near the moment of breakdown, relatively low shoulder lane utilization relates were found versus high passing lane utilization rates, which is an indication of the tendency for "rabbits" to prefer driving in the left most lane, even when gaps are available in the shoulder lane. The macroscopic findings in this paper also corroborate the findings regarding lane choice behavior that were found in research by (De Baat, 2016), who found that during times of high demand, drivers were reluctant to move back into the shoulder lane, which is consistent with the lane flow utilization rates in this paper.

6.2.3. Relation of findings with Dutch Highway Capacity Manual

In the Dutch highway capacity manual, a (median) capacity of 4300 vehicles per hour is given for a 2 lane freeway with on-ramp (Heikoop *et al.*, 2015, p.31). Truck traffic data is available for four locations in the eight-location sample (see chapter 4) and for two of these locations the 50th percentile has been estimated, which enables a comparison of the capacity values that have been found in this thesis with the highway capacity manual. It should be mentioned, however, that no correction has been made for location specific effects in this comparison.

For the A58-L Bavel location, a 50th-percentile capacity of 4200 vehicles per hour was found for a speed limit of 120 km/h. The truck traffic level at this location is approximately 15% and therefore at a similar level as the capacity manual. It can be seen that the value of capacity flow at this location does not deviate much from the capacity manual and that the difference in flows is likely to be explained by location specific factors. A similar story is the case at the A58-R Goirle location, where a 50th-percentile capacity of 4392 vehicles per hour was found under the 120 km/h limit. The truck traffic level at this location was between 5% and 10% and by using the conversion table from the Highway Capacity Manual (Heikoop *et al.*, 2015, p.55), a converted value of 4216 vehicles per hour is found for this location, which is also relatively close to the estimate in the manual.

Additionally, there are some other locations such as on the A2-L and A2-R near Valkenswaard, which have higher 50th percentile capacity values (around 4600 vehicles per hour), but no data is available for truck traffic at these locations. Moreover, it is expected that, when location specific effects are accounted for, most locations will have similar capacity values as presented in the highway capacity manual.

6.3. Limitations of this study

6.3.1. Data

The imposition of speed limit changes from 120 to 130 km/h has been spread out over a period from 2011 until 2019 and speed limit changes were only performed at locations where a limit change was deemed to be sufficiently safe. As such, there is already a selection effect in the data which is based on circumstantial factors such as infrastructure and traffic composition. It is expected that this does not pose a problem for analyses based on the eight-location sample, but it may potentially bias the results for the 17-location sample as some of the locations in this sample have only experienced a speed limit change from 120 to 100 km/h, whilst others have experienced a change from 120 to 130 km/h first and, subsequently, a change from 130 to 100 km/h.

The imposition of the general daytime 100 km/h limit on March 15 in 2020 has provided an example case of a fully exogenous change in the speed limit, regardless of locations. Unfortunately, however, the limit change coincided with the imposition of a nationwide lockdown to control the COVID-19 virus, which immediately affected travel behavior such that much less freeway congestion was experienced and that, potentially, driving behavior itself was also affected. As such, the data obtained under the 100 km/h limit is not necessarily representative of what would have occurred under "normal" conditions. Nonetheless, data from April to July 2020 was included in this study (see section 5.2) to investigate whether an effect was already visible.

Also for truck traffic data there are limitations in this study. Despite having access to data from more than 37.000 measurement locations (NDW, n.d.), not all locations are equipped with loop detectors that can detect vehicle lengths. Because of this, looking for detectors near measurement locations that can reliably detect vehicle lengths is a time consuming task, which is why truck data are only applied in the 8-location sample, for reasons of time constraints. Furthermore, whenever no data or only unreliable data was available, no truck traffic data was included in the regression. Additionally, the five minute average of truck traffic data surrounding a breakdown flow interval is merely a proxy of the truck traffic level at that particular moment and may not always be fully truthful to the traffic composition at the exact moment of breakdown.

6.3.2. Methods

For the application of the product limit method, the most important parameter is the critical speed. In this thesis, a critical speed of 85 km/h was chosen, which was determined on the basis of the shape of the fundamental diagrams in Appendix B. This critical speed could be considered somewhat high as a driving speed of 85 km/h does not necessarily imply congestion. When examining breakdown flow data, however, it was found that the 1-minute lane speeds were generally between 35 km/h and 70 km/h at the moment of breakdown. This makes sense, because the 85 km/h critical speed is applied to 5 minute speed averages, such that, a number of relatively low speed measurements have to be included in this average before it drops below the critical speed. As such, it is expected that the 85 km/h critical speed is a suitable threshold for the evaluation of traffic breakdown at locations where the limit is 120 or 130 km/h. For the 100 km/h limit the same critical speed was used to maintain consistency with the breakdown flows from other limits. It has, however, not been assessed in this thesis whether the 85 km/h critical speed is also suitable for the estimation of breakdown under a limit of 100 km/h.

Additionally, Brilon *et al.* (2005) have found that the level of aggregation can have a significant effect on the levels of capacity that are estimated, with higher capacity values for shorter measurement intervals and lower capacity values for longer measurement intervals. As such, the 5-minute aggregation intervals which are used in this study, imply that findings should be interpreted as 5-minute capacities and are not directly comparable to capacities which are found for shorter or longer intervals. An additional benefit of choosing 5-minute aggregation intervals in this thesis, is that most capacity values in the Dutch highway capacity manual are also 5-minute capacities (Heikoop *et al.*, 2015), which makes the values that have been found in this thesis comparable in some respects.

For the regressions in chapter 5, breakdown flow measurements have been used, which were identified by means of the categorization process of the product limit method. However, instead of subsequently applying the distribution estimation process of the product limit method, measurements have been directly used in the regressions, which implies that they should not be viewed as capacity distributions (which also take into account measurements of category F), but rather as breakdown flow distributions (which only take into account measurements from category B).

The consequence of using these breakdown flow measurement distributions directly, is that the mean of these distributions is lower than the actual capacity distribution, because lower value observations have more weight in the determination of the distribution. As such, these distributions are also more sensitive to "false" identifications of category B measurements, which have less influence when the full product limit method is applied. Consequently, the results from the regressions in chapter 5 should not be interpreted as a definitive answer regarding the capacity of the roadway, but rather as an indication of what happens to the breakdown flow distribution that indirectly constitutes the capacity distribution.

7. Conclusion

In this thesis an investigation has been performed into the relation between capacity and the height of the speed limit. Due to a number of changes in the freeway speed limit at several locations in The Netherlands, many locations have experienced at least two different speed limits and some have experienced three different limits. The limits that are studied in this thesis are the 120 km/h limit, the 130 km/h limit and, to a lesser extent (as a consequence of limited congestion data from this period), the 100 km/h limit. In order to determine whether a significant relation exists between the speed limit and capacity, the following research question was posed in this thesis:

To what extent does the height of the speed limit affect freeway capacity?

In order to answer this question, five sub-questions were posed for which a total of eight hypothesis have been formulated, which will be answered in the remainder of this chapter.

7.1. Results for Sub-Question 1: Given a change in the speed limit, can a change in the capacity distribution be observed at a given location?

- Hypothesis 1a: The capacity distribution for the complete roadway is significantly different for the before and after period (H_1)
- Hypothesis 1b: The breakdown flow distribution for the passing lane is significantly different for the before and after period (H_1)
- Hypothesis 1c: The breakdown flow distribution for the shoulder lane is significantly different for the before and after period (H_1)

When evaluating the capacity distributions for the eight locations under study in chapter 4 it was found that the capacity distributions for 4 out of 8 locations were significantly higher under the 130 km/h limit and that the capacity distributions for the remaining 4 locations were higher under the 120 km/h limit, with three of them significant at the 1% level. Similarly, for the passing lane distributions 5 out of 8 locations were significantly higher under the 130 km/h limit and 3 of them were higher under the 120 km/h limit, of which two were significant. Lastly, for the shoulder lane distributions all differences were significant, for which the distributions were higher at 4 locations under the 130 km/h limit and the remaining 4 locations were significant under the 120 km/h limit.

Based on these findings, it can be concluded that the null hypothesis in hypotheses 1a, 1b and 1c can be rejected in favor of the alternative hypothesis, which entails that significant changes to the capacity distributions have occurred at different locations between measurement periods. However, no uniform direction has been found for the effect of the speed limit on capacity. This could be due to the fact other relevant factors have significantly changed from one measurement period (120 km/h limit) to the other (130 km/h limit), such as the level of truck traffic. Other factors such as traffic management and infrastructure have been accounted for through checking whether these have remained the same at the location from one period to the next. Additionally, changes in weather may have had an effect but it is assumed in this paper that, due to the relatively long measurement periods in the same period of the year, this effect is likely to be negligible when performing a before and after comparison of distributions.

7.2. Results for Sub-Question 2: Given a change in the capacity distribution between measurement periods, can this change also be related to changes in the traffic composition at a location?

- Hypothesis 2: A significant change to the level of truck traffic has occurred between the before and after period (H_1)

In section 4.4 it has been investigated whether levels of truck traffic have significantly changed from one measurement period to the next. Reliable 1-minute truck traffic data could be obtained for four out of eight locations and this data was subsequently matched to the exact times at which traffic breakdown had occurred. To match the aggregation time of the speed and flow data, truck traffic levels were defined as 5-minute moving averages of the minutes surrounding the moment of breakdown. Subsequently, the means of truck traffic levels per measurement period and per location were calculated, such that mean traffic levels were obtained for the before (120 km/h limit) and after (130 km/h limit) period for each location. After this, the means for the before and after period were compared to each other through a t-test for the equality of means. It was found that a significant increase in truck traffic had occurred between the before and after period for two out of four locations and that no significant change had occurred for the remaining two locations. As such, it can be concluded that the null hypothesis for hypothesis 2 has been rejected in favor of the alternative hypothesis for two out of four cases, providing evidence for the fact that the capacity distributions in chapter 4 are likely to be affected by changes in truck traffic levels.

When comparing the locations for which significant increases in truck traffic have occurred, it was found that 1 location had experienced no significant increase in capacity, while another location had experienced an increase in capacity, despite the higher level of truck traffic. Given that increased levels of truck traffic should depress the observed value of capacity, it can be stated that these locations provide some evidence that the 130 km/h limit may have led to a higher capacity, in spite of a negative direction by other factors. On the other hand, there were also two locations for which no significant change in truck traffic was recorded and, for these locations, the capacity distribution was significantly lower under the 130 km/h limit, thus providing evidence for that the capacity may be higher under the 120 km/h limit. Consequently, even when comparing for truck traffic, no uniform direction for the capacity effect can be distinguished in the data.

7.3. Results for Sub-Question 3: Given a change in the capacity distribution between measurement periods, are significant changes in lane choice behavior visible?

- Hypothesis 3: The passing lane utilization rate has significantly changed between the two measurement periods. (H_1)

In literature it has been shown that the speed limit affects the lane flow distribution. Because of this, an analysis has been performed in section 4.5, to test whether significant changes to the lane flow distribution have also taken place at the locations under study. For this analysis, the fraction of flow in the passing lane as a percentage of total flow has been calculated for each breakdown flow measurement. Subsequently, the mean flow proportion was calculated for each location and each measurement period. By using a Z-test for the comparison of proportions, it was tested whether the flow fraction in the passing lane was significantly different from one period (120 km/h) to the next (130 km/h).

It was found that the proportion of flow in the passing lane was significantly higher under the 130 km/h limit for three out of eight locations. For the remaining locations the results were insignificant, though a positive change in passing lane flow seemed to dominate for all but one location. As such, the null hypothesis of hypothesis 3 can be rejected in favor of the alternative hypothesis in at least 3 out of 8 cases, meaning that there is an indication that the speed limit has a significant positive effect on the utilization rate of the passing lane.

These findings are also consistent with the findings from section 4.2, in which it was found that the larger share of locations experienced an increase in passing lane flows, indicating that a larger share of the driving population is inclined to drive in this lane during times of high demand, for a higher speed limit.

7.4. Results for Sub-Question 4: When controlling for other relevant variables and location specific factors, can a general change in breakdown flows be attributed to a change in the legal limit?

- Hypothesis 4a: The speed limit variable of the eight-location sample has a significant effect, when controlling for location specific factors and other relevant variables. (H_1)
- Hypothesis 4b: The speed limit variables of the seventeen-location sample have a significant effect, when controlling for location specific factors and other relevant variables. (H_1)

In the evaluation of sub-questions 1 through 3, effects were investigated separately. To generalize the results from these analyses, it is desired to investigate the different effects jointly. Least Squares Regression theory offers a method which makes it possible to test the significance of the speed limit as an explanatory variable of capacity in conjunction with other relevant factors, such as the level of truck traffic, lane flow distribution effects and location specific effects. Unfortunately, the capacity distributions that were produced in chapter 4 were incomplete and maximum likelihood estimation of the distributions was deemed to risky, because many distributions were estimated only to a minor extent, which would imply that a lot of extrapolation would have been necessary.

Therefore, as a proxy for capacity, the breakdown flow measurements, which constituted the capacity distributions, have been used as a sample for the evaluation of the relation between the speed limit and capacity. Please note that the distribution of breakdown flow measurements by itself is different from the capacity distribution, because the breakdown flow distribution takes into account only measurements of category B, whilst the capacity distribution takes into account measurements of categories B and F (for an explanation see subsection 3.2.1).

By using data from breakdown flow measurements, truck traffic, passing lane utilization rates and location dummies, a fixed effects regression could be performed in which the effect of the speed limit can be evaluated in (relative) isolation from the effects of other factors. Two different samples were analyzed, the eight-location sample, which was also analyzed in chapter 4 and which included truck traffic data and data from speed limits of 120 and 130 km/h, as well as a seventeen-location sample, which included data from 100 km/h, 120 km/h and 130 km/h limits but no truck traffic data.

In the eight-location sample, a significant negative relation between the speed limit and the mean breakdown flow was found, as for (practically) all of the regression results the 120 km/h limit was found to have a higher mean breakdown flow (in the range of 60 to 110 vehicles per hour). Which entails that the null hypothesis of hypothesis 4a can be rejected in favor of the alternative hypothesis.

In the seventeen-location sample, again a significantly negative relation was found for the 130 km/h limit when compared to the 120 km/h limit. In all of the regressions in this sample, the speed limit of 120 km/h was found to be characterized by a significantly higher mean breakdown flow (in the range of 80 to 190 vehicles per hour). For the 100 km/h limit the results are less obvious. In the regressions where the lane flow distribution variable was omitted, the effect of the 100 km/h limit in relation to the 130 km/h limit was tested to be negative. However, when the lane flow distribution was included, the sign of the estimator would switch and the 100 km/h limit would become positive and significant. It was found that the relation between the lane flow distribution variable and the 100 km/h limit dummy was negative, and that the lane flow distribution represented an important omitted variable when not included in the regression (as shown by the R^2 value). As such, it is expected that the negative estimates for this variable are biased and that therefore the positive effect is closer to the truth. Consequently, it was found that the 100 km/h limit was characterized by a significantly higher mean breakdown flow than the 130 km/h limit, albeit lower than the 120 km/h limit. As such, the null hypothesis of hypothesis 4b can also be rejected in favor of the alternative hypothesis.

Notwithstanding aforementioned results, it should be noted that the estimates for the 100 km/h limit may be less reliable than the estimates for the 120 and 130 km/h limit, because they were obtained during the COVID lockdown period. Moreover, it is important to be aware that the fact that the 120 km/h limit is characterized by higher breakdown flows than the 130 km/h limit does not necessarily mean that the capacity under the 120 km/h limit is higher. However, the fact that the mean breakdown flow is higher for this limit does represent an indication that it could very well be the case that the capacity under a limit of 130 km/h is lower than under a limit of 120 km/h.

7.5. Results for Sub-Question 5: Is there a significant relationship between the speed limit and the utilization rate of the passing lane?

- Hypothesis 5: The speed limit has a significant effect on the lane flow distribution, when controlling for location specific effects. (H_1)

In chapter 5 it was found that a quadratic relation best represented the relation between the Breakdown Flow and the Lane Flow Fraction in the passing lane and in section 4.5 evidence was found for a positive relation between the lane flow fraction in the passing lane and the speed limit. As such, it may be the case that the speed limit affects the capacity through inducing changes in lane choice behavior that affect the lane change distribution. To investigate whether a uniform and significant relation between the speed limit and the passing lane utilization rate can be found, an analysis has been performed in section 5.3.

From this analysis it was found that there was a significant positive relation between the speed limit and the utilization rate of the passing lane. It was found that the fraction of flow in the passing lane under the 130 km/h limit was around 62%. Additionally, it was found that the 120 km/h limit led to a reduction of 0.6 to 1.0 percentage points in the proportion of flow on the passing lane and for the 100 km/h limit this reduction was equal to 3 to 4 percentage points. As such, the null hypothesis of hypothesis 5 can be rejected in favor of the alternative hypothesis, which entails that there is sufficient proof for the existence of a positive relation between the speed limit and the passing lane.

7.6. Conclusion regarding the Research Question: To what extent does the height of the speed limit affect freeway capacity?

Altogether, it cannot be determined with certainty that capacity is affected by the height of the speed limit. It has been found in this thesis that a great variety of factors affects capacity and that it is difficult to isolate the effect of a speed limit change, without picking up changes in other factors as well. It has been found to be the case that the speed limit has a significant effect on the lane flow distribution, which in its turn will affect the capacity of the roadway. Moreover, findings in this thesis suggest that it is likely that some range of values for the lane flow distribution exists, under which the roadway is optimally utilized and the mean breakdown flow is maximized. It is expected that, since the speed limit affects the lane flow distribution, a "capacity-optimal" speed limit is likely to exist. It has been proven in this thesis that mean breakdown flows are higher under the 120 km/h limit and that it could, for this reason, very well be the case that capacity has decreased under the 130 km/h limit. As such the answer to the principal research question of this thesis is:

It is uncertain to what extent the speed limit affects capacity, but it is very likely that the speed limit does affect capacity and that the 130 km/h limit exhibits a slightly lower capacity than the 120 km/h limit, when other relevant factors are taken into account.

8. Recommendations

8.1. Scientific Recommendations

On the basis of the findings in this thesis it is recommended that a more thorough before- and after analysis be performed on the capacity of locations where a limit of 100 km/h was recently imposed. As findings from Appendix C suggest, the speed choice behavior under the 100 km/h limit is very different from speed choice behavior under higher limits of 120 and 130 km/h and, for this reason, a bigger capacity effect is expected for this limit than was found in this thesis. It is the expectation of the author that other results will be obtained for the analysis of the 100 km/h data when more data is available for "regular" traffic conditions, which will, hopefully, return after the COVID-19 pandemic.

It would also be interesting to further investigate whether the relation between the speed limit and the utilization rate of the passing lane holds for different limits of, for instance, 80, 70 and 50 km/h, which could be investigated on any two lane "flow-road" with a frequently activated bottleneck, to see whether a general relation between these two factors holds.

Additionally, many more cases of speed limit changes can be expected to be found both nationally as well as internationally. It would be interesting to investigate how capacity distributions in other countries are shaped under a given limit and how it would change as a result of a change in the limit. It would be interesting to see whether significant differences in lane utilization rates exist at the moment of breakdown for countries with different driving cultures and also whether this affects capacity.

8.2. Recommendations for practitioners

In this thesis it was found that it is hard to distinguish a capacity effect for a limit change from 120 to 130 km/h. Additionally, it is uncertain to what extent the new 100 km/h limit has affected the reliability of certain bottlenecks. It may be interesting for an organisation such as Rijkswaterstaat, to design a programme that can (automatically) obtain roadway data from a given location of the network and can, subsequently, generate for each detector location a capacity distribution function, in accordance with the product limit method. In this way, reliability of the network could be periodically reviewed by monitoring the capacity probability distributions of problematic bottlenecks in the network to see if capacity changes over time. Also, when implementing new measures at a location, capacity effects could be quickly evaluated, without the need to perform extensive (and expensive) capacity studies.

Additionally, it is important to consider that a positive relation between the speed limit and the flow fraction in the passing lane has been found. When designing roadway layouts for a given speed limit or deciding on what kind of speed limit to impose on a certain section of freeway, it may be relevant to consider these lane flow distribution effects. Policy makers must take account of the fact that a stronger bias in driving towards the left is likely to occur for a higher limit, while a stronger bias in driving towards the right is likely to occur for a lower limit. Depending on the situation at hand, policy makers should judge which of these effects is desired.

Lastly, it is recommended that policy makers take into account that a higher speed limit will not automatically lead to better travel times on busy sections of road, as it has been proven in this paper that the mean breakdown flow for the 120 km/h limit, and perhaps also the 100 km/h limit, is higher than the mean breakdown flow for the 130 km/h limit, thus causing congestion at lower levels of flow.

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Appendices

Appendix A.

Space Mean Speeds vs. Time Mean Speeds

Imagine two vehicles (A and B) driving a stretch of road that is 20 kilometers long. Vehicle A drives and vehicle B drive consistently with 100 km/h and 50 km/h respectively. Imagine that we would stand somewhere along this 20 km road with a laser gun and measure the speeds of the passing vehicles. We would measure the aforementioned speeds and conclude that the average speed is:

$$\frac{50 \text{ km/h} + 100 \text{ km/h}}{2} = 75 \text{ km/h} \tag{A.1}$$

At first this would seem like a reasonable estimate. However, if we look at how long each vehicle takes over the section we will see that this estimate is wrong.

With a speed of 100 km/h vehicle A would spend a total of 12 minutes to pass this stretch. Vehicle B, driving at a speed of 50 km/h would take twice as long to cover the distance, 24 minutes. The total duration that both vehicles have spent on crossing this stretch of road is therefore 36 minutes and the total amount of kilometers that they have passed is 40. Consequently:

$$\frac{40 \text{ km}}{36 \text{ minutes}} = 66.67 \text{ km/h} \tag{A.2}$$

Which is the true average speed of both vehicles.

This difference between the first average (the arithmetic mean) which is also called the "time mean speed" and the second average (the harmonic mean) which is also called the "space mean speed" can be very large and is dependent on two factors: 1) the number of vehicles in the measurement sample and the 2) (differences in) speed at which the vehicles drive.

Let U^t be the time mean speed and U^s the space mean speed. Then the following is the case:

$$U^t = \sum_{i=1}^{N} v_i \tag{A.3}$$

$$U^s = \frac{N}{\sum_{i=1}^{N} \frac{1}{v_i}} \tag{A.4}$$

If we let D be the difference between the two means we can derive the following:

$$U^t - U^s = D = \sum_{i=1}^{N} v_i - \frac{N}{\sum_{i=1}^{N} \frac{1}{v_i}} = N * \langle v \rangle - \frac{N}{\frac{N}{\langle v \rangle}} = N - \frac{1}{\langle v \rangle} \tag{A.5}$$

As can be seen from Equation A.5 $D \geq 0$ because $N \geq 0$ and $v \geq 0$ therefore it is always the case that $U^t \geq U^s$.

The difference D between the two mean speeds is linearly increasing with the number of vehicles in the sample N (Equation A.6) and is exponentially decreasing with the true mean speed $\langle v \rangle$ (Equation A.7).

$$\frac{\partial D}{\partial N} = +1 > 0 \tag{A.6}$$

$$\frac{\partial D}{\partial \langle v \rangle} = \frac{-1}{(\langle v \rangle)^2} < 0 \tag{A.7}$$

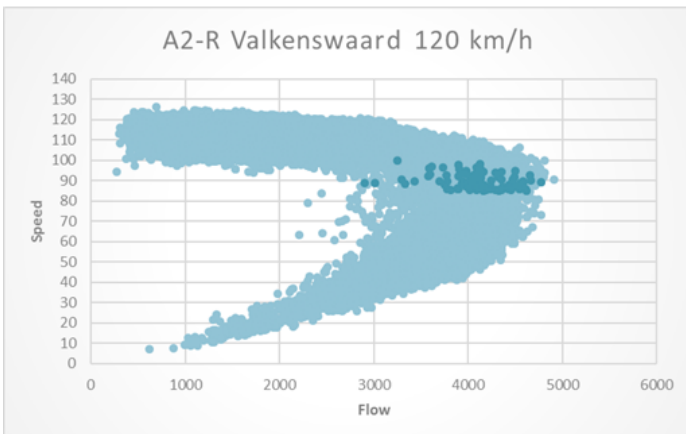
Consequently, due to Equation A.7 the difference between the two means is small if the measured speeds are high, while it increases exponentially if the speed decreases to low levels. This entails that average speed measurements at low real speeds are very unreliable if the average speeds are calculated by means of time mean speeds (which is the case for the Dutch network). This entails that travel time calculations as well as density calculations at these low speeds - i.e. high densities - are unreliable.

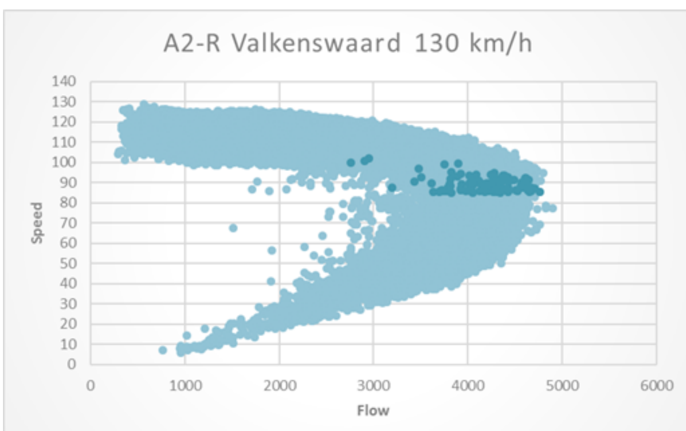
Furthermore, if the measurement period increases the probability of it containing a larger number of vehicle observations increases as well. As a consequence, the difference between the space mean speed and the time mean speed will be larger if the measurement aggregation interval increases (see Equation A.6) and the average speed is measured as a time mean speed.

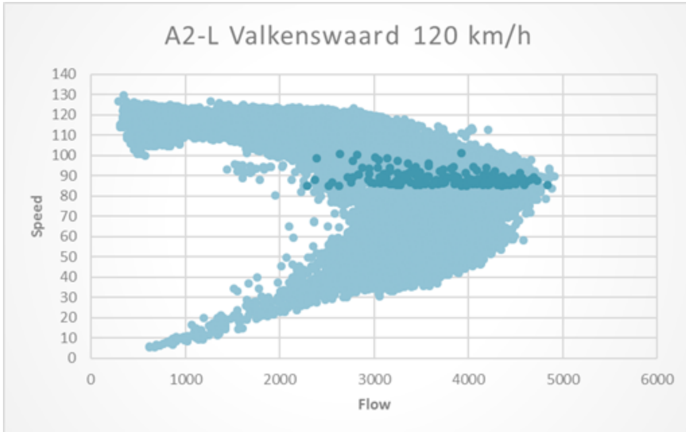
Lastly, as was portrayed by the introductory example, the difference in travel times also shows that even driving short distances at a low speed will disproportionately pull down the average speed of a trip for any given vehicle. This is why any traffic jam, even very short ones, will have a disproportionately large impact on the travel time.

Appendix B. Fundamental Diagrams

In this appendix the fundamental diagrams for each location in the eight-location sample have been plotted in the Speed-Flow plane. In dark blue, the breakdown measurements of the *B* category are highlighted. As can be seen from these diagrams, the choice for a critical speed of 85 km/h is an appropriate one. All breakdown measurements are around the speed for which maximum flow occurs at all locations. These graphs therefore serve as a supporting argument for the critical speed choice in subsection 3.2.1.






A2-L Valkenswaard 120 km/h
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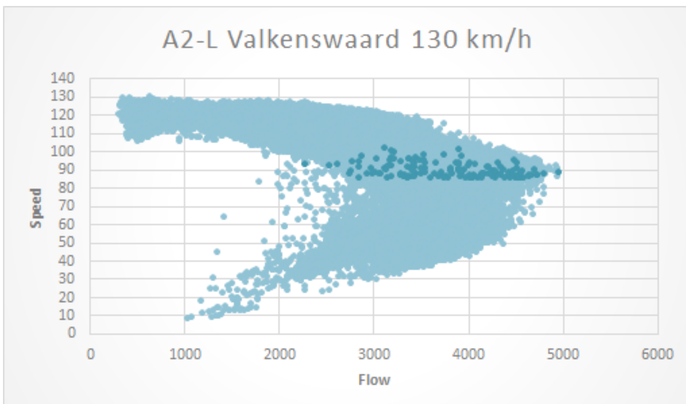
A2-L Valkenswaard 130 km/h
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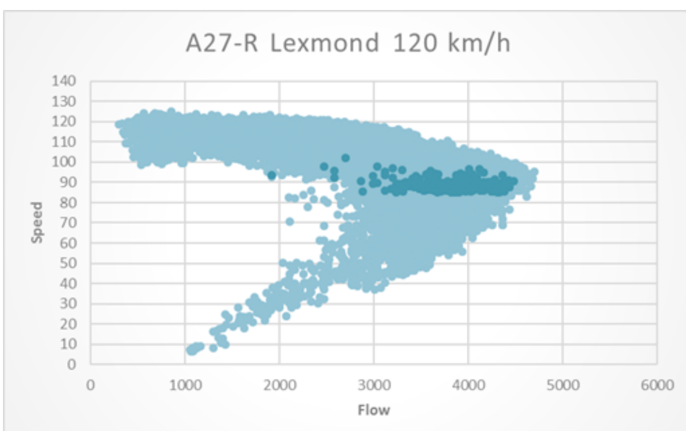
A27-R Lexmond 120 km/h
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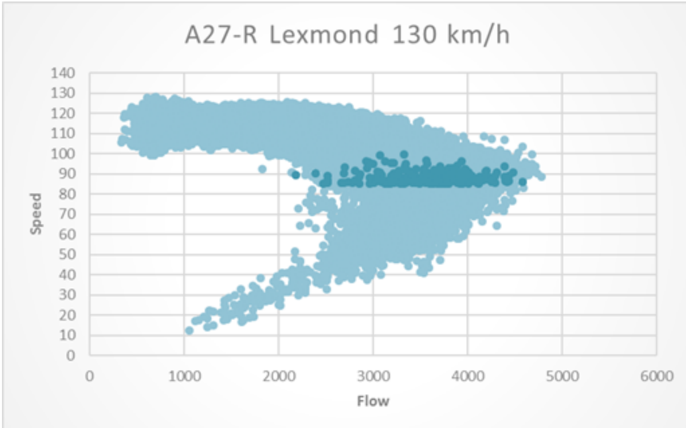
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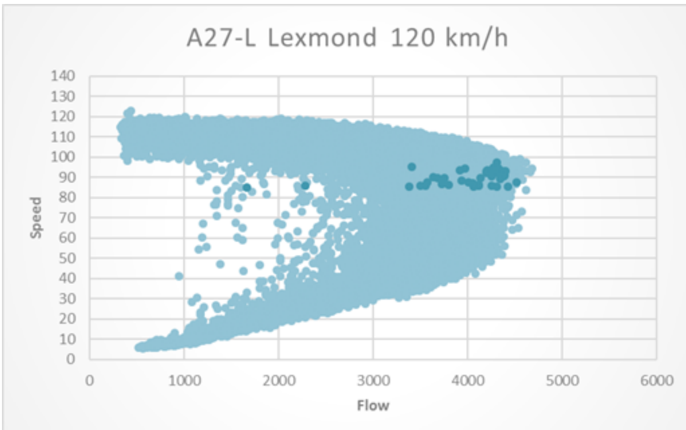
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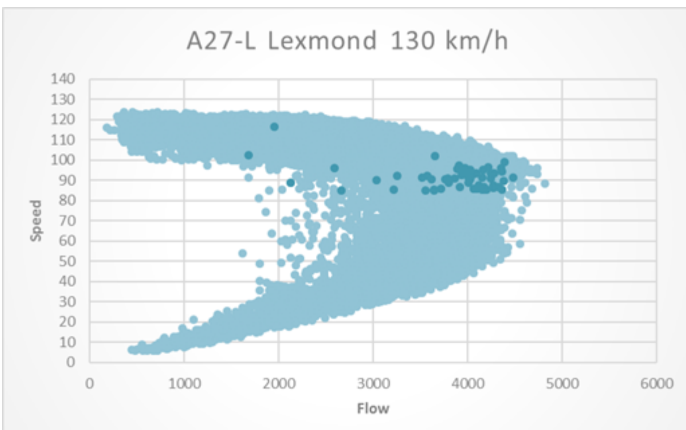
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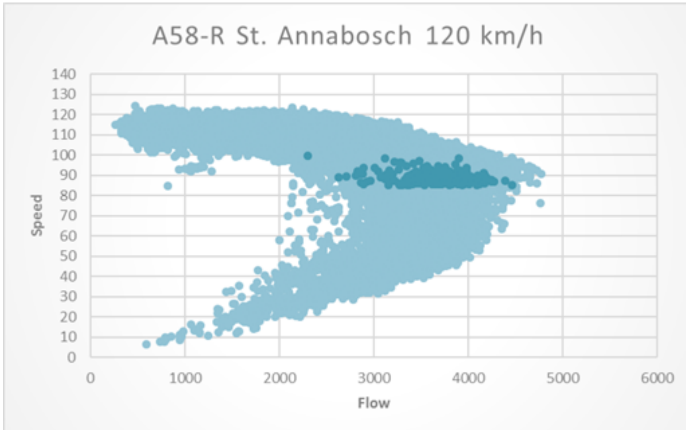
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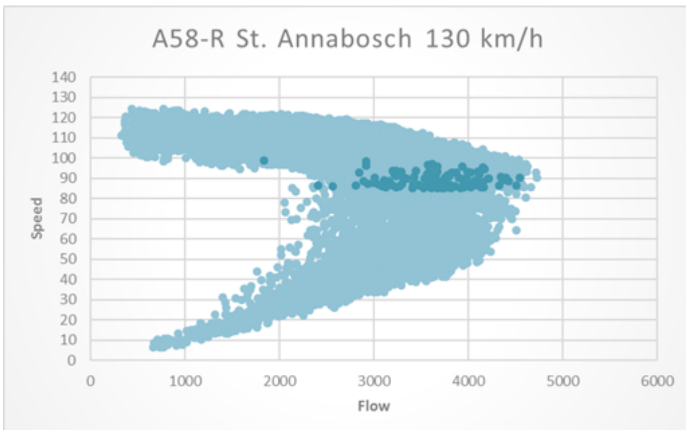
A58-R St. Annabosch 130 km/h
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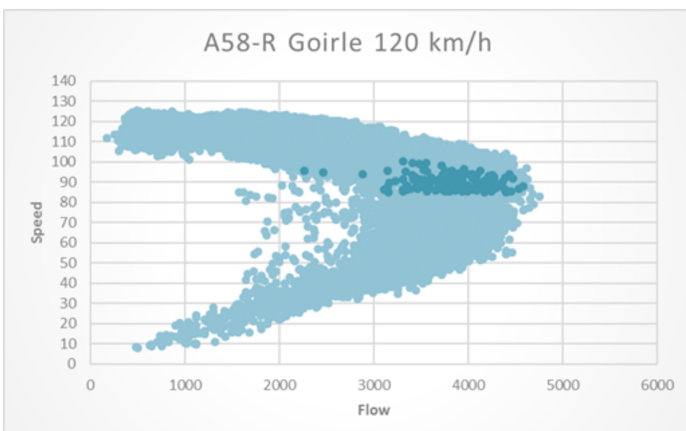
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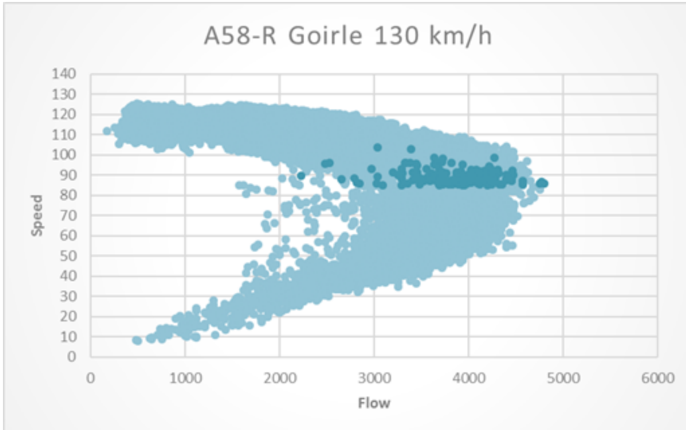
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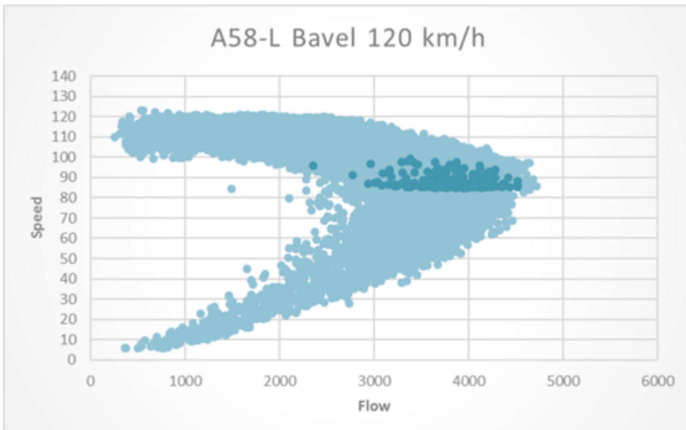
A58-L Bavel 120 km/h
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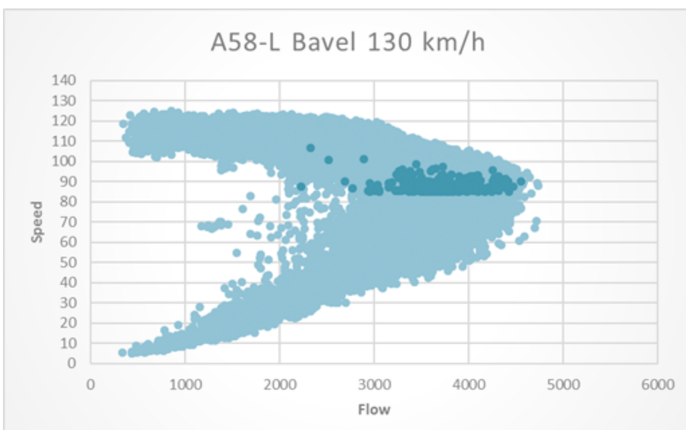
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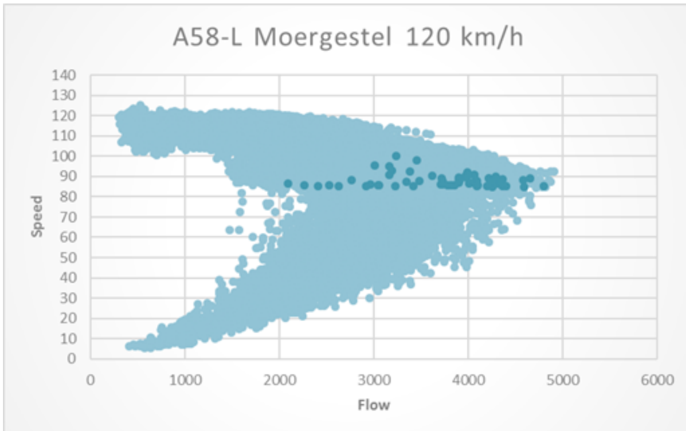
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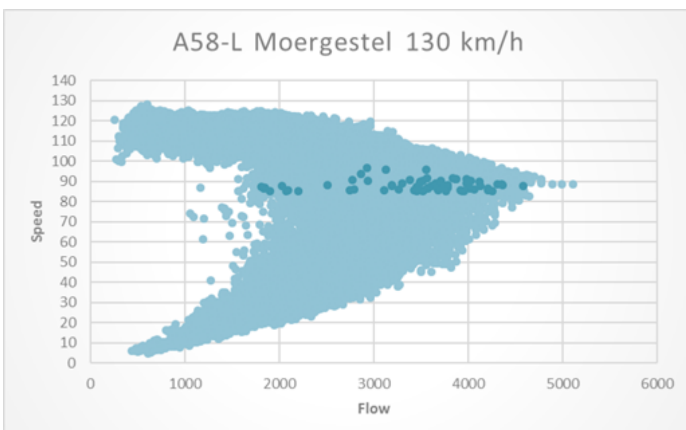
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Appendix C.

Speed Distribution Results

Differences in how a roadway is experienced and the extent to which a limit is enforced have a strong influence on the speeds that are actually driven under a given limit. To investigate whether a change in speed choice behaviour has truly taken place, an analysis of driving speed distributions will be performed for periods with very low levels of traffic. Though the estimations in this chapter may not be perfect representations of the speed distribution function in reality, the internal consistency of the method enables the comparison of different speed limits at a given section of roadway. As such, results in this chapter will be viewed primarily as a gauge of the degree to which speed choice behavior is altered as a consequence of a change in the legal limit.

C.1. Factors Affecting Preference Speed

The preference speed of an individual (v_i^p) is a consequence of a variety of both individual factors and environmental factors. According to the Task-Capability model by Fuller (2005) each individual has certain aptitude for learning driving tasks (constitutional features). Together with training and experience, a driver gains competence for the driving task and, in real driving situations, human factors such as comfort, fatigue and distraction will consequently determine the level of capability of that driver (see Figure C.1). On the other hand, there is the level of task demands which is a function of the environment, other vehicles and vehicle handling difficulty. The speed is the great mediator in determining the task demands, as it sets the pace (and thus reaction time) at which all of the tasks that the driver faces have to be dealt with.

Balancing of Capability and Demands

Fuller (2005) argues that control of the vehicle can only be achieved as long as the capability of the driver exceeds the task demands imposed by the traffic situation. It is further assumed that a driver will attempt to maintain control at all times and will do so by adapting his/her speed to balance the demands of the task to his/her driving capability (Fuller, 2005).

As such, it can be expected that the preference speed of any driver (v_i^p) is both time and location dependent. Drivers vary in their capability and, as such, for any time and location it can be expected that the preference speed of a population of drivers follows a distribution of $v^p \sim (\mu_{v^p}.\sigma_{v^p})$. with a mean μ_{v^p} and a standard deviation σ_{v^p}.

In reality, however, it can be expected that this (theoretical) preference speed distribution is constrained by the speed limit as well as the level of enforcement of that speed limit (SWOV, 2019) (SWOV, 2020), leading to an actual (observed) speed probability distribution

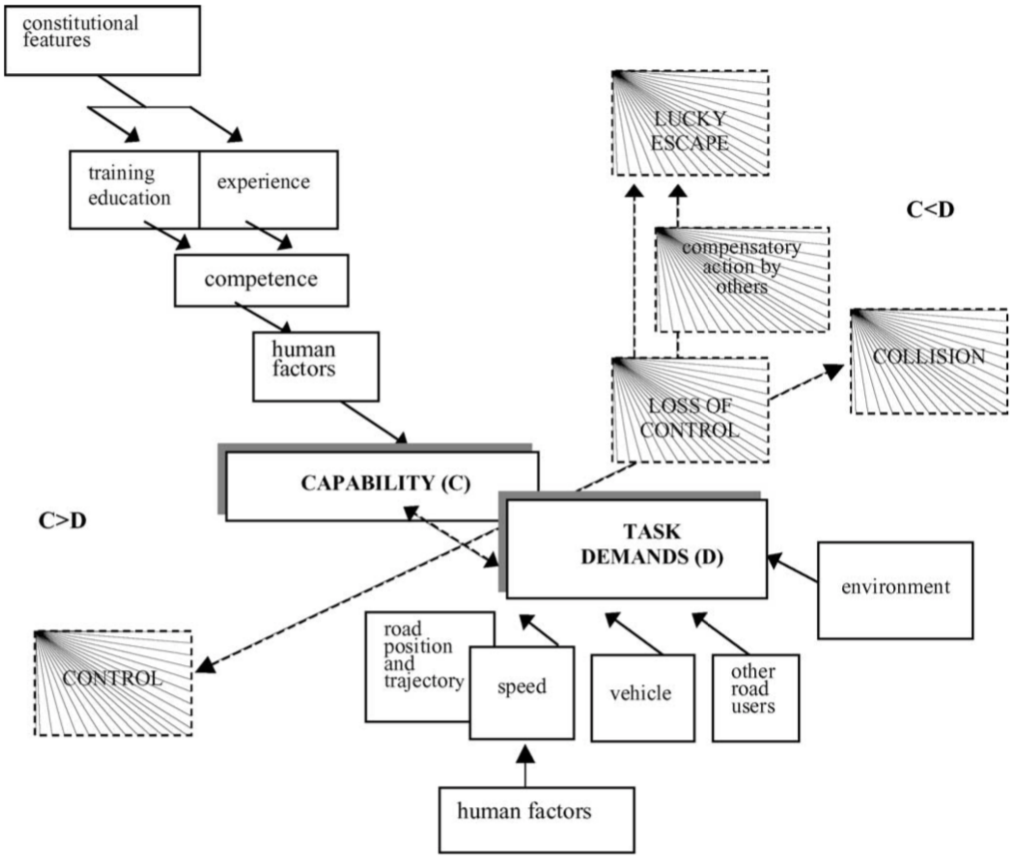

Figure C.1. – Task-Capability model (Fuller, 2005, p.464)

$v^a \sim (\mu_{v^a}.\sigma_{v^a})$, which can be inferred by measuring individual vehicle speeds at a given location.

Firstly, it should be noted that the speed of vehicle i is dependent on whether the vehicle is constrained by a predecessor or not. If vehicle i is constrained by a predecessor with speed v^a_{i+1} the speed of vehicle i (v^a_i) will be strongly dependent on the speed of the predecessor (see chapter 2). If, however, the vehicle is driving in an unconstrained matter, the speed will most likely be higher. It can therefore be expected that the shape of distribution of observed speeds $v^a \sim (\mu_{v^a}.\sigma_{v^a})$ is strongly dependent on the traffic conditions. As such, for the determination of the extent to which drivers comply with a speed limit, it is important to measure only cases in which the driver is driving in a truly unconstrained manner (only during periods of very low traffic demand).

C.2. Estimation Method

As was explained in chapter 3, the minimum interval at which measurements can be retrieved from loop detector data in The Netherlands is one minute. The unavailability of individual vehicle data makes it difficult to approximate a true speed distribution v^a. Nonetheless, a proxy of this distribution can be estimated by filtering the data for one minute intervals in which only 1 vehicle was measured (making the average speed of the interval equal to the speed of that particular vehicle) and in which the speed was higher than 60 km/h (to filter out congestion). Measurements were taken between 06:00 and 19:00 to make sure that the speed limit was the same throughout the measurement period (as many limits are based on a daytime window) and the measurement periods for the 120 km/h and 130 km/h limits were from 1st of March until the 31st of May in subsequent years surrounding a speed limit change from 120 km/h to 130 km/h (see Table 3.2). Also the speed distribution effect of the 100 km/h limit has been included, albeit for a shorter period of 23rd of March until the 28th of April 2020.

Biased distribution

A benefit of this estimation method is that any measurement interval with only 1 vehicle is extremely likely to be measured in a traffic situation in which there is a very low traffic density, meaning that the observed speed of that vehicle will not be influenced by a predecessor. It should be taken into account, however, that this method is very restrictive and that many "free driving" measurements will be missed. Furthermore, because measurements are taken for each lane, because a keep-right policy is attended to on Dutch highways and because measurements were only taken during daytime, a large share of measurements will be observed on the passing lane, while only a very small share of measurements will be observed on the shoulder lane. It is for this reason that for each location, the passing lane and shoulder lane have been independently estimated and plotted.

Because the passing lane has more measurements and because it is also very unlikely that slower vehicles will use this lane during the observed intervals, the cumulative probability functions of the passing lanes are more smooth. Moreover, because only faster vehicles drive in the passing lane and "free driving" measurements in the shoulder lane are frequently missed, the estimated speed probability function $\hat{v^a}$ is likely to have a higher mean than the true speed probability function ($\hat{v^a} > v^a$).

C.3. Results for driving speeds in the passing lane

In Table C.1 an overview of the summary statistics for the free speed distributions in the passing lane has been presented. As can be seen, approximately 1000 to 2000 measurements are included per sample, which allows for parametric statistical tests to be reliably performed (Stock and Watson, 2015). As can be seen from Table C.1 the difference in mean speeds between the 120 km/h limit and the 130 km/h limit is very small for each location, indicating that speed choice behavior is affected only to a small extent. A much larger contrast is visible between the mean speeds under the 100 km/h limit in comparison to the 120 and 130 km/h limits. On average, the mean speeds for the 130 km/h limit are in the range of 125 to 133 km/h, the mean speeds for the 120 km/h limit are in the range of 125 to 130 km/h and the mean speeds for the 100 km/h limit are in the range of 116 to 118 km/h (see Table C.1).

As can be seen from the T-test results that are used to compare the means across different samples (see Table C.2), the differences in mean speeds between the different limits found

	Sample Size	Mean	St.Dev	Variance	Skew	Kurt.
A2LValk100P	1.641	117,39	16,27	264,67	1,43	3,64
A2LValk120P	1.219	130,60	15,83	250,55	0,85	1,84
A2LValk130P	874	133,12	13,99	195,71	0,62	1,59
A2RValk100P	2.162	116,49	16,05	257,50	1,12	2,77
A2RValk120P	1.114	127,56	14,98	224,33	0,29	0,96
A2RValk130P	1.020	129,04	14,31	204,72	0,12	2,27
A27LLex100P	1.669	118,42	16,07	258,21	0,70	0,07
A27LLex120P	1.110	126,16	14,23	202,45	0,36	0,34
A27LLex130P	1.003	131,02	13,53	183,09	0,09	0,39
A27RLex100P	1.920	116,80	15,15	229,51	0,81	0,39
A27RLex120P	1.148	128,35	13,85	191,74	0,36	0,42
A27RLex130P	958	130,38	13,12	172,25	-0,20	1,25
A58LBavel100P	1.663	116,12	14,94	223,33	0,74	0,25
A58LBavel120P	1.335	128,54	14,46	209,09	0,15	0,37
A58LBavel130P	1.026	131,00	13,17	173,42	0,17	0,44
A58RGoirle100P	1.359	115,24	17,08	291,81	1,36	4,19
A58RGoirle120P	1.118	126,52	15,55	241,94	0,84	2,30
A58RGoirle130P	1.191	127,86	14,60	213,30	1,05	3,72
A58LMoergestel100P	1.658	115,73	14,80	218,95	0,92	0,50
A58LMoergestel120P	1.224	129,23	13,35	178,29	0,42	0,22
A58LMoergestel130P	970	131,81	13,20	174,16	0,17	0,15
A58RST.AN100P	1.747	117,62	16,00	256,01	0,68	0,08
A58RST.AN120P	809	127,11	14,39	206,97	0,37	0,05
A58RST.AN130P	967	125,17	14,17	200,86	0,35	0,24

Table C.1. – Summary Statistics Table for Passing Lane Speeds

in Table C.1 are all significant at 1% significance[1]. Though it should be noted that, with respect to the 130 km/h limit, the mean speed for the 100 km/h limit is much lower (10 to 15 km/h) than the mean speed for the 120 km/h limit (only 2 to 3 km/h).

Regarding the standard deviation of the speeds, it is found that the variance in speeds in the passing lane in most cases seems to be larger under the 100 km/h limit than under the 120 km/h limit (first column, Table C.3). Additionally, when compared with the 130 km/h limit, the speed variance in the passing lane at all locations is higher under the 100 km/h limit (second column, Table C.3). When comparing the 120 km/h with the 130 km/h limit (third column Table C.3) the variance is in some cases higher for the 120 km/h limit, though often at a lesser degree of significance. On the basis of the results in Table C.3 it can be concluded that the speed variance in the passing lane seems to be negatively related to the height of the speed limit.

[1] with the exception of a 5% level of significance in the 120 to 130 km/h limit change at Goirle

	t-value(100 vs. 120)		t-value(100 vs. 130)		t-value(120 vs. 130)	
A2LValkP	-21.81	***	-25.35	***	-3.85	***
A2RValkP	-19.55	***	-22.20	***	-2.34	***
A27LLexP	-13.34	***	-21.71	***	-8.05	***
A27RLexP	-21.58	***	-24.81	***	-3.43	***
A58LBavelP	-23.03	***	-27.01	***	-4.31	***
A58RGoirleP	-17.19	***	-20.11	***	-2.12	**
A58LMoergestelP	-25.62	***	-28.81	***	-4.53	***
A58RST.ANP	-14.96	***	-12.68	***	2.85	***

* = 10% significance. ** = 5% significance. *** = 1% significance

Table C.2. – T-Test for equality of means for the passing lane samples (100 km/h compared with 120 and 130 km/h as well as 120 km/h compared with 130 km/h)

	F-value(100 vs. 120)		F-value(100 vs. 130)		F-value(120 vs. 130)	
A2LValkP	1.06		1.35	***	1.28	***
A2RValkP	1.15	***	1.26	***	1.10	*
A27LLexP	1.28	***	1.41	***	1.11	*
A27RLexP	1.20	***	1.33	***	1.11	**
A58LBavelP	1.07		1.29	***	1.21	***
A58RGoirleP	1.21	***	1.37	***	1.13	**
A58LMoergestelP	1.23	***	1.26	***	1.02	
A58RST.ANP	1.24	***	1.27	***	1.03	

* = 10% significance. ** = 5% significance. *** = 1% significance

Table C.3. – F-Test for the comparison of variances for the passing lane (100 km/h compared with 120 and 130 km/h as well as 120 km/h compared with 130 km/h)

Whether this may negatively affect the stability of the flow under the 100 km/h limit remains to be seen, as the Skew is slightly positive in almost all locations, indicating that it is a relatively small number of high speed measurements that have a disproportionate effect on the mean (Table C.1). Lastly, the kurtosis is mostly low, except for the Valkenswaard locations, which indicates that at these locations there are a relatively high number of outliers in the speed measurement data.

When looking at the percentile data that is plotted in Table C.4, it is interesting to note the degree to which drivers seem to drive in accordance with the speed limit. Due to the fact that there is a speed correction of about 5-8 km/h on freeways before getting a ticket, only speeds that are more than 5 km/h above the speed limit will be considered to be in excess of the limit.

For the 130 km/h limit, about 50% to 75% percent of the drivers in the sample seem to comply with the limit, for the 120 km/h limit the number of compliant drivers decreases to about 50% and for the 100 km/h limit the number of compliant drivers decreases to

	15th perc.	25th perc.	50th perc.	75th perc.	85th perc.
A2LValk100P	102,00	105,00	114,00	125,00	132,00
A2LValk120P	117,00	120,00	129,00	138,00	145,00
A2LValk130P	122,00	125,00	130,00	141,00	148,00
A2RValk100P	101,00	104,50	114,00	125,00	132,00
A2RValk120P	115,00	118,00	125,00	136,00	143,00
A2RValk130P	117,00	122,00	129,00	136,00	143,00
A27LLex100P	102,00	106,00	117,00	129,00	134,00
A27LLex120P	114,00	118,00	125,00	134,00	141,00
A27LLex130P	118,00	123,00	130,00	138,00	145,00
A27RLex100P	102,00	105,00	114,00	127,00	132,00
A27RLex120P	117,00	120,00	127,00	136,00	143,00
A27RLex130P	118,00	123,00	130,00	138,00	143,00
A58LBavel100P	101,00	105,00	114,00	125,00	132,00
A58LBavel120P	115,00	118,00	127,00	136,00	143,00
A58LBavel130P	120,00	123,00	130,00	138,00	145,00
A58RGoirle100P	99,00	102,00	113,00	125,00	132,00
A58RGoirle120P	114,00	117,00	125,00	134,00	141,00
A58RGoirle130P	115,00	118,00	127,00	134,00	141,00
A58LMoergestel100P	101,00	103,00	113,00	125,00	130,30
A58LMoergestel120P	117,00	120,00	127,00	136,00	143,00
A58LMoergestel130P	120,00	123,00	130,00	138,00	145,00
A58RST.AN100P	101,00	105,00	115,00	127,00	134,00
A58RST.AN120P	114,00	117,00	125,00	136,00	143,00
A58RST.AN130P	113,00	115,00	123,00	134,00	141,00

Table C.4. – Percentile Table for Passing Lane Speeds

somewhere around 25% (Table C.4).

Another interesting point to note is the consistency of speeds at a given percentile of the distribution across locations. For example, the 25th percentile of the 100 km/h limit for all locations is around 105 km/h, the 50th percentile of the 120 km/h limit seems to be around 125 to 127 km/h and the 85th percentile for the 100, 120 and 130 km/h limits are all in the same range across locations (see Table C.4). This indicates that road user speed choice behavior is relatively consistent among the locations in this study. Furthermore, one can observe that speed choice behavior for a speed limit of 120 and 130 km/h is relatively similar and that especially drivers in the upper quartile of the sample are relatively insensitive to whether the speed limit is 120 or 130 km/h.

C.4. Results for driving speeds in the shoulder lane

In comparison to the passing lane samples in Table C.1, the samples for the shoulder lanes are relatively small (40 to 340 measurements per sample). The reason for this is that the measurement method is very restrictive and that it is less likely that a single vehicle per minute will be counted on the shoulder lane, between the times of 06:00 and 19:00, as a result of the keep right rule.

	Sample Size	Mean	St.Dev	Variance	Skew	Kurt.
A2LValk100S	248	107,59	15,48	239,55	1,38	5,05
A2LValk120S	133	118,01	17,00	288,87	-0,12	0,33
A2LValk130S	55	118,04	21,55	464,55	-0,41	-0,56
A2RValk100S	338	105,07	17,41	303,10	1,14	2,76
A2RValk120S	164	110,24	17,22	296,57	0,08	-0,12
A2RValk130S	153	115,62	17,39	302,38	-0,02	0,03
A27LLex100S	312	105,35	16,88	285,00	0,70	0,11
A27LLex120S	115	110,24	17,22	296,64	-0,05	0,24
A27LLex130S	99	112,21	18,23	332,35	0,09	-0,38
A27RLex100S	218	104,19	15,78	248,91	0,98	1,11
A27RLex120S	111	114,09	15,88	252,12	-0,31	-0,04
A27RLex130S	80	118,49	16,68	278,33	-0,40	-0,27
A58LBavel100S	228	104,43	15,25	232,48	0,96	1,57
A58LBavel120S	75	109,12	19,00	360,92	-0,30	-1,11
A58LBavel130S	42	117,21	15,03	225,93	-0,70	-0,06
A58RGoirle100S	180	104,03	16,70	278,85	0,37	0,17
A58RGoirle120S	195	115,89	20,03	401,25	0,03	-0,38
A58RGoirle130S	196	116,79	18,85	355,40	-0,11	-0,40
A58LMoergestel100S	263	107,83	17,72	314,13	0,86	0,64
A58LMoergestel120S	82	116,15	14,43	208,23	-0,49	0,60
A58LMoergestel130S	76	113,45	16,83	283,32	-0,59	-0,17
A58RST.AN100S	204	104,83	17,05	290,56	0,58	-0,02
A58RST.AN120S	59	109,02	18,83	354,64	-0,21	-0,75
A58RST.AN130S	48	114,00	17,25	297,53	0,55	1,04

Table C.5. – Summary Statistics Table for Shoulder Lane Speeds

As can be seen from Table C.5, mean speeds in the shoulder lane are a lot lower than in the passing lane samples. Just as with the passing lane samples, the mean speeds across locations for a given speed limit are relatively constant, with speeds in a range of 105 to 107 km/h under the 100 km/h limit, speeds in a range of 110-118 km/h under the 120 km/h limit and speeds in a range of 113 to 118 km/h per hour under the 130 km/h limit (Table C.5).

When testing for significance of the differences between the means, it can be found that significant differences are found when the mean speeds of the 100 km/h limit are compared

	t-value (100 vs. 130)		t-value (100 vs.120)		t-value (120 vs. 130)	
A2LValk130S	-5,88	***	-3,41	***	-0,01	
A2RValk130S	-3,14	***	-6,22	***	-2,76	***
A27LLex130S	-2,62	***	-3,32	***	-0,81	
A27RLex130S	-5,36	***	-6,65	***	-1,83	*
A58LBavel130S	-1,94	*	-5,05	***	-2,54	**
A58RGoirle130S	-6,25	***	-6,96	***	-0,45	
A58LMoergestel130S	-4,31	***	-2,53	**	1,08	
A58RST.AN130S	-1,53		-3,32	***	-1,43	

* = 10% significance. ** = 5% significance. *** = 1% significance

Table C.6. – T-Test for equality of means for the shoulder lane samples (100 km/h compared with 120 and 130 km/h as well as 120 km/h compared with 130 km/h)

with the 120 km/h limit (first column in Table C.6) and the 130 km/h limit (second column in Table C.6). However, when the 120 km/h limit is compared to the 130 km/h limit, there are only three locations that show a significant difference, while for the other five locations no significant difference can be found (third column in Table C.6).

From Table C.7 it can be seen that, despite some differences in variance between the different limits, only a few locations experience significant changes. In contrast to the passing lane, the speed variance in the shoulder lanes seems to increase with a rise in the speed limit. The variance in speed is already higher than in the passing lane, because the vehicle population is more heterogeneous, but the fact that variance in the shoulder lane increases with speed, while it decreases with speed in the passing lane, gives an indication that as the speed limit increases, the degree of heterogeneity in the passing lane decreases and while it goes up in the shoulder lane. This is therefore deemed indicative of a change in lane change behaviour.

	F-value (100 vs. 130)		F-value (100 vs.120)		F-Value (120 vs. 130)	
A2LValk130S	0,83		0,52	***	0,62	**
A2RValk130S	0,98		1,00		0,98	
A27LLex130S	0,96		0,86		0,89	
A27RLex130S	0,99		0,89		0,91	
A58LBavel130S	0,64	**	0,97		0,63	*
A58RGoirle130S	0,69	***	0,78	**	0,89	
A58LMoergestel130S	0,66	**	0,90		0,73	*
A58RST.AN130S	0,82		0,98		0,84	

*= 10% significance. ** = 5% significance. *** = 1% significance

Table C.7. – F-Test for the comparison of variances for the shoulder lane (100 km/h compared with 120 and 130 km/h as well as 120 km/h compared with 130 km/h)

Additionally, when looking at the percentiles in the shoulder lane it can be seen that, especially at the lower end of the distribution, the speeds are much lower in the shoulder lanes (Table C.8). In contrast to the findings for the passing lanes, there is much larger variation across locations for the percentile values under a given limit. Furthermore, there are many cases in which a percentile of the 120 km/h limit or even the 100 km/h limit has a higher speed value than the 130 km/h limit percentile. Especially for the lower percentiles there is much variation across locations for the same limit and, seemingly, this variation decreases when looking at higher percentiles such as the 75th and 85th percentiles of the distriution (Table C.8).

The degree of heterogeneity found in the shoulder lane in combination with the degree of homogeneity in the passing lane is in accordance with the "slugs and rabbits" theory from Daganzo (2002a), where the passing lane will only contain fast vehicles during periods of low flow, which is a relatively homogeneous driver population, while the shoulder lane will contain a mix of slugs and rabbits under these conditions, and does therefore represent a much more heterogeneous driver population. It is also for this reason that a more bi-modal speed distribution is apparent in the speed distribution graphs of the shoulder lanes, while the distributions of the passing lane seem to be uni-modal.

	15th perc.	25th perc.	50th perc.	75th perc.	85th perc.
A2LValk100S	94,35	99,00	105,00	114,75	124,30
A2LValk120S	97,00	109,00	120,00	127,00	132,00
A2LValk130S	86,00	98,00	125,00	132,00	138,00
A2RValk100S	85,00	91,00	103,00	115,00	122,15
A2RValk120S	87,75	98,00	114,00	122,00	125,00
A2RValk130S	94,10	103,00	118,00	127,00	130,00
A27LLex100S	87,00	93,00	102,00	116,50	125,00
A27LLex120S	88,40	100,00	113,00	122,00	127,00
A27LLex130S	90,00	97,00	114,00	125,00	129,00
A27RLex100S	88,85	96,00	100,50	111,00	122,00
A27RLex120S	97,60	106,00	117,00	125,00	130,00
A27RLex130S	99,00	106,25	122,00	130,00	134,00
A58LBavel100S	91,00	95,00	101,00	113,00	119,30
A58LBavel120S	83,40	92,00	114,00	125,00	129,00
A58LBavel130S	100,45	106,00	121,00	127,00	132,20
A58RGoirle100S	88,00	93,25	103,00	111,00	123,00
A58RGoirle120S	92,40	100,00	118,00	130,00	136,00
A58RGoirle130S	93,00	103,00	120,00	130,00	136,00
A58LMoergestel100S	89,60	97,00	103,00	118,00	129,00
A58LMoergestel120S	100,00	109,50	118,00	125,00	129,00
A58LMoergestel130S	93,65	100,25	117,50	125,00	130,00
A58RST.AN100S	86,00	89,25	103,00	115,00	122,25
A58RST.AN120S	87,00	91,00	110,00	123,00	129,00
A58RST.AN130S	96,70	105,00	114,00	122,00	128,95

Table C.8. – Percentile Table for Shoulder Lane Speeds

Appendix D.

Significance table for the small sample Willcoxon Signed Rank Sum Test

From Keller (2009, B-25):

	5%-sig (two-tail)		*10%-sig (two-tail)*	
n	***T(l.b.)***	***T(u.b.)***	***T(l.b.)***	***T(u.b.)***
6	1	20	2	19
7	2	26	4	24
8	4	32	6	30
9	6	39	8	37
10	8	47	11	44
11	11	55	14	52
12	14	64	17	61
13	17	74	21	70
14	21	84	26	79
15	25	95	30	90
16	30	106	36	100
17	35	118	41	112
18	40	131	47	124
19	46	144	54	136
20	52	158	60	150
21	59	172	68	163
22	66	187	75	178
23	73	203	83	193
24	81	219	92	208
25	90	235	101	224
26	98	253	110	241
27	107	271	120	258
28	117	289	130	276
29	127	308	141	294
30	137	328	152	313