Quantifying Saturation Flow of Right Turning Lanes on Intersections

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Master thesis

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Abstract

Saturation flow is a fundamental value in the dimensioning of signalized intersections, specifically in the allocation of green time to the intersection lanes. In response to identified overestimation of saturation flow by certain projection methods that base on Swiss standards, this research analyzes the accuracy of projection methods found in the relevant Swiss, US and German standards. These projection methods, as well as some modified methods, are compared to measured saturated flows, which are obtained from detector data on intersection lanes in Zürich in collaboration with the city's traffic operator, 'Dienstabteilung Verkehr'.

The standards' presented methods to predict saturation flow by adjusting an ideal flow for real conditions with respective factors are shown to be fairly accurate. None of the standards present an adjustment methodology that consistently yields better results than the methodologies of the other two countries' standards. Modifications to these methods, to better account for presumed synergies between the effects of the infrastructure's gradient and the presence of heavy vehicles, cannot be shown to consistently improve accuracy.

The Swiss alternative methodology, which uses a fixed average value without adjustment to estimate saturation flow, is shown to consistently and substantially overestimate saturation flow. Its usage on right turning lanes is therefore strongly advised against and an adjustment or removal of the relevant entries in the Swiss standards is recommended.

Keywords

Saturation Flow, Signalized Intersections, Projection Methods, Comparison, Zürich

Preferred citation style

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Abbreviations

Considered Intersections

RR	Röschibachstrasse - Rosengartenstrasse
PD	Pfingstweidstrasse - Duttweilerbrücke
WH	Witikonerstrasse - Hofackerstrasse

Modifications on Projection Methods

base	no modification
a1	Calculation of the heavy vehicle adjustment factor as a function of the gradient based on PCE-values for trucks without trailers by VSS (1999)
a2	Calculation of the heavy vehicle adjustment factor as a function of the gradient based on PCE-values for trucks with trailers by VSS (1999)
b	Calculation of the heavy vehicle adjustment factor as a function of the gradient and the heavy vehicle share based on PCE-values by Skabardonis et al. (2014)

Terms

DAV	Dienstabteilung Verkehr
FGSV	Forschungsgesellschaft für Strassen- und Verkehrswesen
GAM	General Additive Model
HBS	Handbuch für die Bemessung von Strassenverkehrsanlagen
HCM	Highway Capacity Manual
TRB	Transportation Research Board
VSS	'Schweizerischer Verband der Strassen- und Verkehrsfachleute', formerly known as 'Vereinigung Schweizerische Strassenfachleute'

Units

PCE	passenger car equivalents
veh	vehicles

1 Introduction

1.1 Background

"The roadway right-of-way is allocated among the modes through the provision of facilities that ideally serve each mode's needs. However, in many urban situations, the right-of-way is constrained by adjacent land development, causing transportation engineers and planners to consider trade-offs in how to allocate the right-of-way." (TRB (2010), Chapter 3, Modal Characteristics, p. 3-1)

Urban areas are characterized by a high population density, a high density of opportunities for activities and, consequently, a high demand for mobility. High demand for mobility induces high traffic volumes, which in turn requires a regulatory body to guarantee both safe and smooth operation. A main task of this traffic operator is to allocate available space for mobility between transportation modes and traffic streams. Optimal allocation of mobility is particularly critical at intersections, where multiple traffic streams cross and limit each other's capacity.

At signalized intersections the traffic operator provides the right-of-way to the various traffic streams by allocating green time. This is done by considering each approach's ratio of demand to saturation flow. Saturation flow is thus a fundamental component in the planning and operation of intersections. Therefore, it is essential to determine it as realistically as possible, especially since demand depends on long-term developments that are often difficult to predict.

For existing intersections, saturation flow can be measured. For planned intersections, however, it must be calculated using projection methods. Zürich's traffic operator, the *Dienstabteilung Verkehr* (DAV, 2017), is highly interested in a correct calculation of saturation flow. Especially since they have dealt with a number of cases in which congestion resulted from overestimation of saturation flow. In particular, a widespread method using a fixed 'average' saturation flow value, presented by current Swiss standards, seems to have led to these overestimations.

1.2 Research Target

In response to the mentioned overestimation of saturation flow, this thesis aims to present a scientific foundation for discussions on the suitability of various projection methods to estimate saturation flows. Accuracy of the projection methods is determined by comparing projected and measured saturation flows for suitable intersections in Zürich. The thesis further considers potential shortcomings of projection methods and tests the performance of alternative approaches on the relevant aspects.

1.3 Limitations

Intersections exist in designs of varying complexity. In order to keep this thesis within the preset limits, mainly defined by the time frame, this analysis exclusively focuses on purely homogeneous right-turning lanes. The exclusion of mixed lanes allows for easier and more reliable data acquisition on measured saturation flows. Right-turning traffic streams have relatively simple relations to other streams while the turning movement provides an interesting aspect to the calculation of saturation flows. The results of this analysis may be indicatory for other types of traffic streams as well, but should not be applied directly.

1.4 Structure

Chapter 2 reviews relevant concepts of traffic planning and discusses literature that addresses the variability of saturation flow.

In Chapter 3, the methodology of both the projection methods and the empirical determination of saturation flow is presented.

The results on the accuracy of the projection methods are shown and discussed in Chapter 4.

Chapter 5 supplements the discussion by providing information on some secondary aspects of the analysis and challenging the used methodology.

Finally, Chapter 6 summarizes the findings of this research, discuss possible changes to relevant Swiss standards and suggest potentially interesting fields for further research.

2 Literature Review

This chapter reviews traffic planning concepts that are fundamental to this research. It further discusses influences on saturation flow based on reviewed literature and presents a selection of relevant adjustment factors as documented in considered standards.

2.1 Basics

This section discusses the concepts of *saturation flow*, *green time allocation*, *passenger car equivalents*, *heavy vehicles* and *effective green time*.

2.1.1 Saturation Flow

The reviewed standards define saturation flow as follows:

VSS	(VSS, 1997)	The highest discharge flow from an approach lane that is observable during the lane's effective green time.
HCM	(TRB, 2010)	The expected average discharge flow for a saturated approach lane during the lane's green time.
HBS	(FGSV, 2015)	The amount of vehicles that could discharge from an approach lane in an uninterrupted green phase of one hour.

Albeit similar, the definitions differ in an important aspect. The US 'Highway Capacity Manual' (HCM) accounts for variability of saturation flow with changes in prevailing conditions such as weather, vehicle mix and numerous others[1]. The German 'Handbuch für die Bemessung von Strassenverkehrsanlagen' (HBS) does not specifically mention any variability of saturation flow but by considering the average flow over a full hour it implicitly smoothes variation of saturation flow over sixty minutes. The Swiss VSS definition, on the other hand, neither mentions variability nor considers smoothening over a substantial time period. The consensus of the three definitions is the understanding that saturation flow addresses the flow rate during a lane's green phase only. Also, all three reviewed standards distinguish between ideal and real saturation flows.

[1]Consider Section 2.2 for an overview over relevant influences.

Ideal Saturation Flow

The ideal saturation flow is the saturation flow reached under 'ideal conditions'. These ideal conditions relate to pure straight-going lanes, a grade of about 0%, absence of bus stops and on-street parking near the intersection, sufficient storage space on the approaching lane, and, dependent on the source, a lane width between 3.0 and 3.7 m (VSS, 1997; TRB, 2010; FGSV, 2015). The ideal saturation flow is usually indicated in PCE per unit of time. In the reviewed literature, it ranges between 1900 PCE/hr (TRB, 2010) and 2000 PCE/hr (VSS, 1997; FGSV, 2015) for traffic streams on urban intersections.

Real Saturation Flow

The real saturation flow is the saturation flow reached under given conditions. It is calculated by multiplying the ideal saturation flow with adjustment factors which account for the real conditions on an intersection (see Equation 1). The real saturation flow is usually indicated in vehicles per unit of time (TRB, 2010; FGSV, 2015) or PCE per unit of time (VSS, 1997). The Swiss VSS standards state an average value of 1800 PCE/h for real saturation flows (VSS, 2008).

$$s_{\text{real}} = s_{\text{ideal}} \times \prod_i f_i \tag{1}$$

where

s_{real}	=	real saturation flow
s_{ideal}	=	ideal saturation flow
f_i	=	adjustment factor for influence i

Relation to Capacity

Saturation flow is a key component in the calculation of a lane's capacity. This is shown in Equation 2 which is fundamental to the allocation of green time (see following section).

$$C_i = s_i \times \frac{g_i}{z} \tag{2}$$

where

C_i	=	capacity of approach lane i
s_i	=	saturation flow on lane i
g_i	=	mean effective green time for lane i
z	=	mean cycle time

2.1.2 Green Time Allocation

At intersections, mobility opportunity is distributed by the traffic operator by providing green time to the approaches. This allocation of green time is critical when total traffic demand on the intersection is approaching (or exceeding) capacity. The green time that is given to each approach is commonly calculated by comparing the approaches' ratios of demand to capacity (TRB, 2010). This is best explained at the example of a fictional merge, where the two approaches (a) and (b) experience high traffic demand.

As both lanes are critical, the green time is now typically calculated so that the ratio of demand to capacity of the two lanes is equal[2], as shown in Equation 3. Including the information Equation 2 gives us and dividing both sides of Equation 3 by the cycle time z, we receive Equation 4. This equation can be transformed to Equation 5, which shows that the distribution of green time is entirely based on the relations of demand to saturation flow on the relevant lanes.

$$\frac{q_a}{C_a} = \frac{q_b}{C_b} \tag{3}$$

$$\frac{q_a}{s_a \times g_a} = \frac{q_b}{s_b \times g_b} \tag{4}$$

$$\frac{g_b}{g_a} = \frac{q_b/s_b}{q_a/s_a} \tag{5}$$

where

q_i	=	demand on approach lane i
C_i	=	capacity of approach lane i
s_i	=	saturation flow on lane i
g_i	=	mean effective green time for lane i
z	=	mean cycle time

2.1.3 Passenger Car Equivalents

Passenger Car Equivalents (PCE) are used to express the impact of vehicle types on traffic performance in relation to regular passenger cars. Commonly differentiated vehicle types include bicycles, motorcycles, passenger cars, vans, buses and trucks with and without trailers. Table 1 provides an overview of PCE-values valid at intersections according to the reviewed

[2] For the sake of conciseness, we assume that no approach is prioritized over the other.

literature. As stated in the table, the effect of heavy vehicles is the highest, due to their size and difference in operating capabilities compared to regular cars (TRB, 2010).

Table 1: Passenger car equivalents at intersections

Transportation Mode		VSS (1997)	HCM (2010)	HBS (2015)
Passenger Cars	including pick-ups	1	1	1
Motorcycles		$^1/_2$	-	1
Bicycles		$^1/_4$	-	0
Heavy Vehicles	average heavy vehicle mix	2	2	1.9

Source: Own representation based on VSS (1997), TRB (2010) and FGSV (2015) in order of publication

Theoretically, PCE-values depend on the circumstances. For example, a bicycle may have a different weight at an intersection than at a roundabout (VSS, 1997, 2006). Similarly, a heavy vehicle's impact may depend on the grade of the road infrastructure and the share of heavy vehicles on the total traffic stream (VSS, 2010; TRB, 2010; Skabardonis et al., 2014). Furthermore, PCE-values may vary globally due to differences in typical driver behaviour. However, the reviewed standards do not discuss any variability relevant to intersections.

2.1.4 Heavy Vehicle Definition

The reviewed literature provides several definitions and PCE-values for heavy vehicles such as trucks, buses and agricultural vehicles. Since the distinction between different heavy vehicle types is impractical for applied planning, the HCM offers a pragmatical definition by assigning the attribute *heavy vehicle* to any vehicle with more than four tires touching the ground. This definition refers to an average mix and is used in this thesis as well, due to its practicality. The respective PCE-values in the literature range from 1.9 to 2.0 PCE (see Table 1).

2.1.5 Effective Green Time

Traffic engineers distinguish between green time and effective green time. The former is the duration a traffic light shows green, the latter is an adjustment of this time to the following two considerations:

- A yellow light following a green light requests approaching vehicles to halt before the stop line - if possible. However, under oversaturated conditions, many drivers still depart during yellow, effectively increasing the time during which discharge occurs. The German HBS (FGSV, 2009) argues that increased driver's frustration amplifies this effect for especially short green phases (≤10 seconds).

- The first few vehicles depart at higher headways, due to the additional time required to react to the initiation of the green phase and to accelerate (TRB, 2010; Pitzinger, 1996). This additional time is known as start-up lost time.

By subtracting the start-up lost time from the green time and adding the utilized time of the yellow phase, a better approximation of the time during which vehicles depart at saturation flow can be determined. According to Pitzinger (1996) however, this so-called *effective green time* still includes not fully saturated times at the beginning and end (see Figure 14 in Appendix A). Based on empirical studies, Pitzinger and the German HBS (FGSV, 2015) further argue that the effective green time is usually *one second longer* than the regular green time [3].

2.2 Influences on Saturation Flow

As discussed in Section 2.1.1 real saturation flow depends on prevailing conditions. This section gives an overview on the various influences on saturation flow as found in reviewed literature. The identified influences are grouped into infrastructural, traffic inherent, operational and environmental aspects[4]. Table 2 provides an according overview, its elements are concisely explained in Section 2.2.1 up to Section 2.2.4. For these sections, the same sources were used as in Table 2. Additional sources are mentioned where relevant.

2.2.1 Infrastructure Factors

Lane width impacts saturation flow in direct proportion. Narrow roads slightly reduce saturation flow rate while wider roads tend to increase it.

The longitudinal *gradient* of an intersection lane impacts saturation flow inversely proportional. A positive inclination will lead to a lower saturation flow than a level one. Moderately negative inclinations may increase saturation flow. The impact of steep negative grades on intersection flows is not commented on in the reviewed literature. The US HCM (TRB, 2010) remarks, however, that trucks may disrupt traffic flow on long highway segments of negative inclination.

[3] The US HCM presents no value for effective green time
[4] Some influences (e.g. on-street parking) could be listed in multiple categories (infrastructure, traffic).

Table 2: Classification of influential factors regarding saturation flow on an intersection lane

Infrastructure	Traffic	Operation	Environment
Lane Width	Share of Heavy Vehicles	Allowed Movements on Lane	Weather
Gradient	Share of Motorcycles	Duration of Green Phase	Readability
Radius of Turn	Share of Bicycles	Transit Priority	Distractions
Upstream Storage Space	Prevalent Trip Purpose	Pedestrian Blockage	Area Type
On-street Parking	Driver Attentiveness	Concurrent Traffic Streams	
Number of Approach Lanes	Cultural Differences	Right Turns on Red	
Proximity of Bus Stop			
Proximity of Side Road			
Lane Position Relative to Curb			

Source: Own representation based on McCoy and Heimann (1990), VSS (1997), FGSV (2009), TRB (2010), Asamer and Van Zuylen (2011), Shao et al. (2011), Jensen (2014), FGSV (2015) and Gao et al. (2016) in order of publication

A narrow *radius of a turn* movement will reduce the saturation flow. The extent of the effect depends on the turn type (i.e. left or right) and the radius.

Insufficient *upstream storage space* may limit saturation flow when a traffic light provides green to an intersection lane while this lane is blocked (e.g. by congestion on a shared lane in the upstream).

On-street parking in vicinity of an intersection reduces saturation flow as a function of the frequency of parking maneuvers during green phases.

The *number of approach lanes* influences saturation flow. If multiple lanes allow the same movements on the intersection, the vehicle mix may differ between these lanes, thereby causing differences in achievable flows.

Bus stops in proximity of an intersection may limit saturation flow if a lane gets blocked by a bus during passenger exchange.

Side roads with low corner clearances may reduce saturation flow in dependency of the frequency of movements onto or from the side road. The geometry of the side junction affects the additional headway time cause by movements onto or from the side road.

On approaches with multiple lanes, a lane's *position relative to the curb* influences saturation flow if other factors such as bus stops, side roads or on-street parking are relevant.

2.2.2 Traffic Factors

Presence of *heavy vehicles* influences the saturation flow twofold. Due to their larger size and lower acceleration, heavy vehicles take longer to pass an intersection. On turning lanes, their reduced maneuverability may further enhance this effect. On positively inclined road infrastructure the limited acceleration has a higher effect (TRB, 2010; VSS, 2010). Additionally, the differences in operation between heavy and regular vehicles create gaps behind and in front of heavy vehicles.

Motorcycles and *bicycles* are shorter and have higher maneuverability capacities than cars. They are therefore given a lower weight than cars in some literature (see Section 2.1.3).

Depending on the *prevalent trip purpose* among road users, saturation flow may vary. This is largely attributed to the drivers' level of familiarity with the intersection. Hence, higher saturation flows are expected for commuting traffic than for leisure traffic.

The drivers' level of *attentiveness* influences their reaction time and thereby their headway. As flow is the inverse of the average headway, driver attentiveness is an important influence on saturation flow.

Due to *cultural differences*, the saturation flow on intersections may vary between different areas of the world. A study found the US HCM methodology to be non-applicable on Chinese intersections (Shao et al., 2011). Among the causes for different flows could be differences in vehicle mix, driver behaviour and climate effects.

2.2.3 Operation Factors

The *allowed movements* on a lane influence the smoothness of traffic on the intersection. Heterogenous movements on an approach lane will disrupt the flow on the intersection.

As established in Section 2.1.5, the *duration of a green phase* affects saturation flow due to lost start-up time and driving during yellow. The shorter the green phase, the higher the impact of both effects. The discussed effective green time accounts for these influences but is only an approximation (see Figure 14 in Appendix A). Therefore, saturation flow may still show dependency on the duration of the effective green time.

Intersections with *transit priority* reduce green time for regular traffic if the public transport gets an exclusive phase. If public transport and regular traffic use the intersection simultaneously, the saturation flow of conflicting traffic may be reduced.

Blockage of a turning traffic stream by a simultaneously active *pedestrian stream* reduces the discharge flow of the blocked stream.

Concurrence among traffic streams will limit the saturation flow of the intersection lane with the lower priority.

Turning right on a red light may increase the capacity of an intersection lane, albeit not its saturation flow. These movements may, however, reduce the saturation flow of rivaling vehicle streams as their discharge flow may be impaired. Regardless, turning right during red is prohibited in Switzerland.

2.2.4 Environment Factors

Weather primarily influences saturation flow by altering road surface conditions or limiting visibility. This effect becomes especially dominant in winter, when icy roads radically hamper vehicle operability.

Readability refers to the road users' ability to navigate safely and smoothly on an intersection. This may be limited when visibility is low (e.g. due to large obstacles) or signalization is bad.

Distractions reduce driver attentiveness, thereby increasing reaction times and headway between vehicles.

Depending on what *type of area* the intersection is located in, different saturation flows may be determined. The German HBS (FGSV, 2009) states that saturation flows in central city areas are usually larger than outside of inhabitated areas. The US HCM (TRB, 2010), on the other hand, claims that saturation flows in central business districts are expected to be lower, due to inefficient intersection design. Either way, the area type effect is likely a result of other factors such as pedestrian blockage, trip purpose or driver attentiveness.

2.3 Reviewed Standards

Internationally, many institutions present methodologies to calculate saturation flow. In this thesis, three of these are analyzed regarding their accuracy in predicting saturation flow on intersection lanes. Section 2.3.2 discusses adjustment factors that are used in the three standards.

2.3.1 Relevant Standards

Since this thesis focuses on conditions in Switzerland, the Swiss standards by the 'Schweizerischer Verband der Strassen- und Verkehrsfachleute' (VSS) are of obvious interest. Additionally, methods proposed by the US Transport Research Board (TRB) are considered as their Highway Capacity Manual (HCM) is internationally known and used. Furthermore, the German Handbuch für die Bemessung von Strassenverkehrsanlagen by the Forschungsgesllschaft für Strassen- und Verkehrswesen (FGSV) is also taken into consideration.

These three standards differ in many aspects such as their understanding of saturation flow, the calculation of specific adjustment factors and what factors are even to be taken into account. Figure 15 in Appendix A shows which standards discuss which influences. The standards' primary goal is to provide calculation methods for the most relevant and also applicable adjustment factors. Therefore, many factors are not discussed in the reviewed standards.

Applicability of a calculation method is mainly dictated by the number of input variables and the effort required to acquire them. Adjustment factors for infrastructural influences like lane width or grade are usually examples of high applicability, as the input variables are often easily acquirable. Factors for traffic-dependent, operational and environmental influences tend to be more complicated to calculate, due to the difficulty to predict or measure the input variables (e.g. driver attentiveness), the complex character of the calculation (e.g. allowed movements on the lane) or interdependence between effects (e.g. heavy vehicle impact may depend on weather).

In Switzerland, standards regarding the field of road and transportation are published by the 'Schweizerische Verband der Strassen- und Verkehrsfachleute' (VSS, 2017). The standards issued by this organisation are split up in individual documents. Multiple standards with varying publication years are relevant to this thesis. The key document regarding adjustment factors for saturation flows on signalized intersections dates back 20 years (VSS, 1997).

The relevant German standards are found in the manual for dimensioning road traffic infrastructure by the 'Forschungsgesellschaft für Strassen- und Verkehrswesen' (FGSV, 2015). Its most recent edition was published in 2015.

The United States' Highway Capacity Manual is issued by the Transportation Research Board (TRB, 2010). Its latest version was published in 2010.

2.3.2 Adjustment Factors in the Reviewed Standards

Of the influential variables discussed in Section 2.2, six are covered in all three analyzed standards[5]. These are *lane width*, *gradient*,*radius of turn*, *share of heavy vehicles*, *pedestrian blockage* and *concurrent traffic streams*. For the former four, all three standards provide adjustment factors for saturation flow, the latter two are sometimes addressed by adjustments of the intersection lane's usable green time.

Table 3: Calculation of the four key adjustment factors according to the reviewed standards

Standard & Factor			Calculation
Lane Width			(where w = *width* in m)
VSS	$f_{w,\text{VSS}}$	=	$1 + \frac{w-3.25}{20}$
HCM	$f_{w,\text{HCM}}$	=	$\left\{\begin{array}{cccc} 0.96 & \text{for} & w < & 3.05\text{ m} \\ 1 & \text{for} & w & \text{otherwise} \\ 1.04 & \text{for} & w > & 3.93\text{ m} \end{array}\right\}$
HBS	$f_{w,\text{HBS}}$	=	$\left\{\begin{array}{cccc} \frac{1}{1+3/8\times(3-w)} & \text{for} & w < & 3\text{ m} \\ 1 & \text{for} & w >= & 3\text{m} \end{array}\right\}$
Radius of Right Turn			(where r = *radius* in m)
VSS	$f_{r,\text{VSS}}$	=	$\frac{1}{1+1.5\times r}$
HCM	$f_{r,\text{HCM}}$	=	$\frac{1}{1.18} = 0.85$
HBS	$f_{r,\text{HBS}}$	=	$\left\{\begin{array}{cccc} \frac{1}{1.3-0.015\times r} & \text{for} & r <= & 20\text{ m} \\ 1 & \text{for} & r > & 20\text{ m} \end{array}\right\}$
Gradient			(where g = *grade* in %)
VSS	$f_{g,\text{VSS}}$	=	$1 - \frac{g}{50}$
HCM	$f_{g,\text{HCM}}$	=	$1 - \frac{g}{200}$
HBS	$f_{g,\text{HBS}}$	=	$\frac{1}{1+g\times 0.03}$
Share of Heavy Vehicles			(where p_{HV} = *share of heavy vehicles* on total vehicles in %)
VSS	$f_{HV,\text{VSS}}$	=	$\frac{100}{100+p_{HV}*(PCE_{HV}-1)}$ with $PCE_{HV} = 2.0$
HCM	$f_{HV,\text{HCM}}$	=	$\frac{100}{100+p_{HV}*(PCE_{HV}-1)}$ with $PCE_{HV} = 2.0$
HBS	$f_{HV,\text{HBS}}$	=	$\frac{100}{100+p_{HV}*(PCE_{HV}-1)}$ with $PCE_{HV} = 1.9$

Source: Own representation based on VSS (1997), TRB (2010) and FGSV (2015) in order of publication

[5]For an overview on which influential variables are considered by which standard, consult Table 15 in Appendix A

As concurrent rivaling traffic streams are not present on any of the analyzed intersections (see Section 3.1.1), this thesis does not further discuss the respective adjustment. Blockage of the discharge by pedestrians is possible on the intersection WH and will be considered for this node only. The other four influential variables are relevant at all chosen intersections. The adjustment factors to these four influences can be calculated with just one input value each (see Table 3) and, with the exclusion of the share of heavy vehicles, these input values can be obtained from technical plans or measured directly. Therefore, these factors are relatively easy to determine and apply.

As visible in Table 3, the calculation methods for the respective adjustment factors vary between standards. The factor for right turns[6], for example, is calculated as a function of the curve radius in the VSS while the HCM presents one constant value for all radii. In contrast, the factor for heavy vehicle presence is calculated very similarly in the three standards. They all use an adjustment over passenger car equivalency-values (PCE) and differ only marginally in the PCE-value applied.

Note that the calculation methods listed in Table 3 are limited to certain domains[7]. For instance, a factor for lane width only make sense if the width is no less than one and no more than two typical car widths.

[6] Since this thesis focuses on right turning lanes, the factor *radius of turn* henceforth refers to right turns only.
[7] For additional information, consider the respective standards.

3 Methodology

This chapter discusses the choice of suitable intersections for the analysis and provides information on the chosen nodes. It also documents the methodology of the used projection methods as well as how saturation flow is determined from real data.

3.1 Intersection Choice

For the comparison of saturation flows derived from projection methods and empirical data, three intersections within the city of Zürich are chosen. This section presents the set requirements for analyzed intersections as well as information on the chosen nodes.

3.1.1 Intersection Requirements

For the purpose of reliable and useful data analysis, potentially interesting intersections have to meet a number of requirements. The most important aspect to consider is that saturation flow can only be measured during saturated times. As such, traffic demand should exceed the capacity of the intersection lane as often as possible to guarantee saturated upstream conditions.

As this thesis focuses on right-turning lanes, only purely homogenous right-turning lanes are of interest. Additionally, the presence of suitable detectors on the intersection lane is required as the empirical determination of saturation flow is to be based on detector data. For obvious reasons, only intersections with operational traffic lights are considered.

To ensure regular discharge, only approach lanes that are at least 100 m long are considered. This length is expected to provide sufficient storage space. Additionally, intersections must not be subject to ongoing construction works during the data collection. Regular discharge also requires the intersection's immediate downstream to be uncongested. Finally, no rivalling vehicle streams are to be active during green phases of the relevant intersection lane.

[7] In traffic engineering, the term *saturated* conventionally describes conditions where traffic demand is exactly *equal to* capacity. In this thesis, for the sake of conciseness, the term is used to describe conditions where traffic demand is *equal to or greater than* capacity, thereby additionally including over-saturated states.

3.1.2 Chosen Intersections

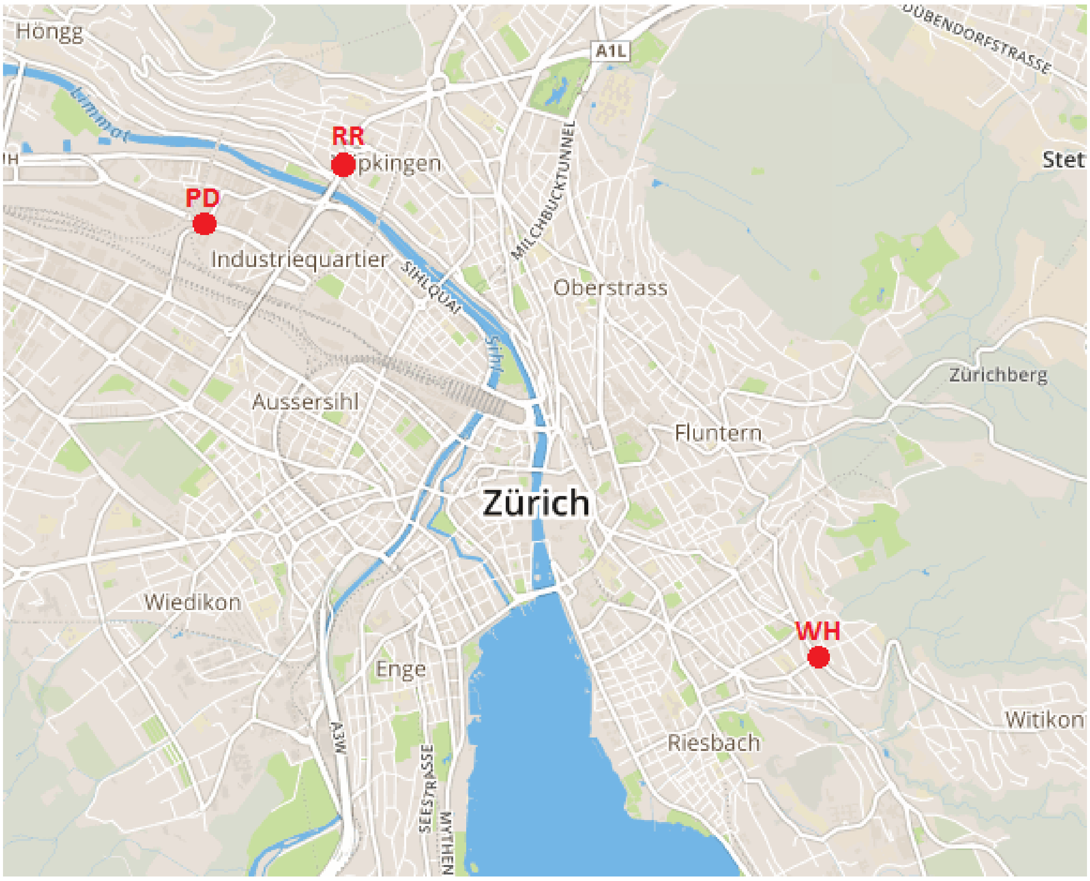
Based on the above defined requirements the below listed intersections are chosen. Their positions in Zürich are shown in Figure 1. Table 4 provides an overview on the geometries of the relevant infrastructure.

Table 4: Chosen intersections and information on the relevant infrastructure

Intersection	RR	PD	WH
Origin-street	Röschibachstrasse	Pfingstweidstrasse	Witikonerstrasse
Destination-street	Rosengartenstrasse	Duttweilerbrücke	Hofackerstrasse
Minimum Lane Width [m]	3.8	3.2	2.8
Minimum Turn Radius [m]	15.0	12.0	6.5
Average Gradient [%]	5.8	3.0	-2.0
Upstream Lane Length [m]	>100	>100	>100

Source: own measurements

Figure 1: Positions of the chosen intersections within the city of Zürich shown by the red markers



Source: Google Maps, modified

3.1.3 Röschibachstrasse - Rosengartenstrasse

The intersection *Röschibachstrasse-Rosengartenstrasse (RR)* is actually more of a merge than an intersection in the traditional sense as there is only one way to go for any vehicle entering the junction (see Figure 15 in B.1.1).

The intersection links the districts *Wipkingen* and *Industriequartier* to the northern part of the city and the regional road network. It is also part of a route connecting motorway tail '1H' with motorway tail '1L' which is increasingly used in times when the regular motorway 'Nordumfahrung' gets more and more congested (DAV, 2017). Since the closing of an alternative route using Röschibachstrasse and some back roads in 2015 (DAV, 2017), demand on the considered intersection lane has further increased. As traffic demand is also high on the rivaling approach from Hardbrücke, the green time that can be given to the right-turn from Röschibachstrasse (northbound) onto Rosengartenstrasse (northbound) is limited. The priority that is given to the busses on the rivaling approach further amplify this situation. Thus, traffic demand on this approach is very often higher than capacity. The resulting congestion spreads to intersections further upstream. To sum up, the right-turn on intersection RR is considered a major bottleneck.

In immediate proximity of the stop line on Röschibachstrasse there is a merge with a sideroad[8]. An estimated share of slightly less than 5 % of vehicles turn onto this side road[9]. A roughly equal amount enters the merge RR from the side road[10]. For additional information and a technical plan of the intersection, see appendix B.1.1.

3.1.4 Pfingstweidstrasse - Duttweilerbrücke

The intersection *Pfingstweidstrasse - Duttweilerbrücke (PD)* is located in a predominantly industrial district of the city. The street 'Pfingstweidstrasse' is one of two roads that directly lead to and from the western motorway tail '1H'. The bridge 'Duttweilerbrücke' is one of few links crossing the railroad tracks in the west of Zürich's main station. As such, the right turn from Pfingstweidstrasse (eastbound) onto Duttweilerbrücke (southbound) is a highly frequented element of the city's road network.

[8] This side road is the north-eastern leg of Röschibachstrasse.

[9] According to own measurements, roughly 2 % of vehicles turn onto the side road during a green phase. This value does not account for vehicles that enter the side road when the traffic light shows red.

[10] Based on own measurements, slightly more than 3 % of vehicles that discharge from Röschibachstrasse onto Rosengartenstrasse (relevant right-turn on intersection RR) come from the side road during a green phase. This value does not account for vehicles that leave the side road during red phases.

Although public transport is present on the intersection, no effect on the distribution of green time is observed. This is due to the intersection being operated in a mostly fixed manner as part of a coordinated axis. For additional information and a technical plan of the intersection, consider appendix B.1.2.

3.1.5 Witikonerstrasse - Hofackerstrasse

The intersection *Witikonerstrasse - Hofackerstrasse (WH)* lies in Zürich's south east. The road Witikonerstrasse connects the district of Witikon with the rest of the city. The right turn from Witikonerstrasse (south-east bound) onto Hofackerstrasse (south-west bound) is part of a route onto Forchstrasse and out of the city.

Discharge from the intersection is sometimes interrupted by the odd pedestrian as the two relevant green phases mostly overlap. The green time allocated to the south-east bound approach of Witikonerstrasse is prolonged when a bus approaches the intersection. Conversely, when a bus approaches on the north-west bound approach, a green phase for the considered right turn may be shortened. For additional information and a technical plan of the intersection, consider appendix B.1.3.

3.2 Projection Methods

On existing intersections, saturation flow can be measured. In the planning of a new intersection or a modification of an existing one, saturation flow has to be estimated. As discussed in Section 2.1.1, the real saturation flow can be calculated by using correction factors to adjust the ideal saturation flow to prevailing conditions. It has further been established that adjustments of saturation flow based on *lane width, right turn movement, gradient* and *the share of heavy vehicles* are the most prevalent (see Section 2.3.2). The reviewed standards differ not only in suggested calculation methodology but also in the determination of the input variable. For instance, the HBS (FGSV, 2015) considers the average grade between two points 30 m before and 30 m behind the stop line while the HCM (TRB, 2010) only looks at the approach's grade. Table 5 gives an overview of the chosen definition of input variables in this research.

In this section, the three analyzed standards projection methods are examined and enhanced with alternative calculation approaches. In total, 14 methods are analyzed; three unchanged 'base' methods (VSS_{base}, HCM_{base}, HBS_{base}) with three alternative modifications each (a1, a2, b) and two submethods ($avg_{VSS,0}$, $avg_{VSS,adj}$) of an approach based on an 'average' value of 1800 PCE/h for saturation flow as presented in the Swiss standards (VSS, 2008).

Table 5: Definition of the input variables used in this thesis for the four most prevalent adjustment factors

Adjustment Factor	Input Variable Specification
Lane Width	Minimum *lane width* 30 m before and behind stop line
Right Turn Movement	Minimum curve *radius* that right-turning vehicles have to pass
Gradient	Average *grade* 30 m before and behind stop line
Share of Heavy Vehicles	Average *proportion of heavy vehicles* on traffic stream

Source: Own specifications based on VSS (1997),TRB (2010) and FGSV (2015) in order of publication

3.2.1 Base Methods and 'Average'-Method

Base Methods

As mentioned in Section 2.1.1, the three reviewed standards present very similar definitions of when saturation flow is ideal. For ideal saturation flows on urban intersections, the HCM(TRB, 2010) presents a value of 1900 PCE/h, the VSS and the HBS assume 2000 PCE/h (VSS, 1997; FGSV, 2015). Based on these values and on the adjustment factors listed in Table 3 in Section 2.3.2, real saturation flow is calculated as presented in Table 6 in accordance with Equation 1 in Section 2.1.1.

Table 6: Methodology of the unchanged 'base' projection methods found in the reviewed standards and the simplistic 'average'-method found in VSS (2008). The adjustment factors are calculated in accordance to Table 3 in Section 2.3.2.

		Base methods			avg_{VSS}	
		VSS_{base}	HCM_{base}	HBS_{base}	0	adj
Ideal Saturation Flow	PCE/hr	2000	1900	2000	-	-
Average Saturation Flow	PCE/hr	-	-	-	1800	1800
Lane Width	-	$f_{w,\text{VSS}}$	$f_{w,\text{HCM}}$	$f_{w,\text{HBS}}$	-	-
Right Turn Movement	-	$f_{r,\text{VSS}}$	$f_{r,\text{HCM}}$	$f_{r,\text{HBS}}$	-	-
Gradient	-	$f_{g,\text{VSS}}$	$f_{g,\text{HCM}}$	$f_{g,\text{HBS}}$	-	-
Share of Heavy Vehicles	-	$f_{HV,\text{VSS}}$	$f_{HV,\text{HCM}}$	$f_{HV,\text{HBS}}$	-	$f_{HV,\text{VSS}}$
Real Saturation Flow	veh/hr	s_{real}	s_{real}	s_{real}	1800	s_{real}

Source: Own representation based on VSS (1997), VSS (2008) TRB (2010) and FGSV (2015) in order of publication

'Average'-Method

For the above mentioned 'average' method found in the Swiss standards (VSS, 2008) two submethods are considered. One submethod ($avg_{VSS,adj}$) contains the adjustment for heavy vehicle presence, the other does not ($avg_{VSS,0}$). The submethod with adjustment shows the correct methodology according to the standard. However, the submethod without adjustment better reflects common practice, as the adjustment for heavy vehicles is often ignored since it requires additional effort and yields very similar results when heavy vehicle shares are very low (i.e. about <2%).

Comparability of Methodologies and Necessary Adjustments

In the case of heavy vehicle presence, the reviewed standards differ in the way they apply the adjustment factor. The HCM (TRB, 2010) and the HBS (FGSV, 2015), on the one hand, apply the adjustment factor for heavy vehicle presence in the calculation of the saturation flow, consequentially stating real saturation flow in vehicles per hour. The Swiss standard (VSS, 1997), on the other hand, applies the heavy vehicle adjustment factor on traffic demand. The real saturation flow is thus stated in passenger car equivalents per hour in the VSS approach. This approach is reasonable when the ratio of demand to saturation flow is considered but is questionable when reached saturation flow itself is of interest. In this analysis, for the purpose of comparability across standards, the VSS base method (VSS_{base}) and its modifications (VSS_{a1}, VSS_{a2}, VSS_{b}) are adjusted to include heavy vehicle adjustments in the calculation of saturation flow. Consequently, all saturation flow estimates used in the comparison are indicated in veh/h.

3.2.2 Modifications

The considered standards present the state of the art in the field of traffic engineering. However, the differences between them (see Table 3 in Section 2.3.2) show that there is no guarantee that any of their methodologies is the most accurate. Therefore, alternative calculations to the above presented base methods are to be considered where reasonable.

The reviewed standards consider the effect on traffic performance of both heavy vehicles and gradient. For uninterrupted road segments (TRB, 2010; VSS, 2010) and unsignalized intersections (VSS, 1999) they also discuss interdependence of these factors. However, they fail to debate on such an interaction for saturation flow at intersections. Based on the observation that heavy vehicles' acceleration rates decrease overproportionally compared to passenger cars on positively sloped roads (Jain et al., 2015), this lack could prove a major oversight. Thus, this thesis examines modifications to take the synergies of the share of heavy vehicles and the road's gradient into account. A total of three modifications is considered, as documented in Table 7.

Table 7: Considered modifications to the three base methods

Modification	Heavy Vehicle Type	Calculation of Heavy Vehicle Impact	Source
a1	Trucks without trailers	see Table 19	SN 640 022 (VSS, 1999)
a2	Trucks with trailers	see Table 19	SN 640 022 (VSS, 1999)
b	Average truck mix	see Table 20	Skabardonis et al. (2014)

Source: Own representation based on VSS (1999) and Skabardonis et al. (2014) in order of publication

Modifications a1 and a2

Modifications a1 and a2 base on a Swiss standard on inclined unsignalized intersections (VSS, 1999). Modification a1 uses the standard's PCE-values for trucks *without* trailers, modification a2 uses the respective PCE-values for trucks *with* trailers (see Table 19 in Appendix B.2.1). As such they differ from the base methods, which use PCE-values for an average mix of heavy vehicles to assess their impact on traffic performance. Both modifications calculate the PCE-values in dependency of the infrastructure gradient.

The effect of the grade on passenger cars is calculated the same way as in the respective base method. Likewise, the adjustment factors for lane width and right turn movement remain unchanged from the base methods[11]. Equation 6 below shows how the modified adjustment for road gradient and heavy vehicle presence is computed.

Modification b

Modification b is based on a 2014 paper by Skabardonis et al. which analyzes the effect of an approach lane's grade on the observed PCE of heavy vehicles. Skabardonis et al. further acknowledge that the PCE of heavy vehicles is a function of their proportion in a traffic stream. This is because the disruptive effect of heavy vehicles decreases when they travel in platoons. Modification b consequently uses PCE-values for an average vehicle mix (see Table 7) that considers the synergetic effects of heavy vehicle presence and gradient and also accounts for the dependency on the share of heavy vehicles. The respective values, as determined by Skabardonis et al. (2014), are listed in Table 20 in Appendix B.2.1.

Gradient effect on passenger cars is computed the same way as in the base methods. The same applies for the adjustment factor for lane width and right turn movement. Equation 6 below shows how the modified adjustment for road gradient and heavy vehicle presence is computed.

[11] Alternatively, grade effect on passenger cars could be calculated using a method proposed in SN 640 022 (VSS, 1999). However, this method is expected to substantially overestimate gradient effect and is thus not considered. See Appendix B.2 for further elaboration.

Simultaneity of Calculation

Although PCE-values for passenger cars and heavy vehicles are determined separately[12] in the modifications (see above), the calculation of the saturation flow adjustment factor requires simultaneous consideration of the two. This is because the two elements can not be decoupled, as evident in Equation 6.

$$f_{\text{grade \& heavy vehicles}} = \frac{100}{100 + s_{HV} \times (PCE_{HV} - 1) + (100 - s_{HV}) \times (\frac{PCE_{PC}}{f_g} - 1)} \tag{6}$$

where

$f_{\text{grade \& heavy vehicles}}$	=	adjustment factor for the effect of gradient and heavy vehicle presence on saturation flow [veh/PCE]
p_{HV}	=	share of heavy vehicles on total vehicles [%]
PCE_{HV}	=	grade-adjusted[13]PCE-value for heavy vehicles [PCE/veh]
PCE_{PC}	=	unadjusted PCE-value for passenger cars (= 1 PCE/veh)
f_g	=	grade adjustment factor for passenger cars [-]

3.2.3 Summary

The above established methodologies of the base methods and the three modifications are summarized in Figure 2. The diagram presents the generalized approach on the calculation of real saturation flows for the analyzed intersections. For the calculation of specific factors, consider sections 3.2.1 and 3.2.2. As mentioned in these sections and as is evident from the diagram, the modifications affect only the last two steps. Potential synergies of heavy vehicle presence and average road gradient are taken into account. Modifications a1 and a2 are based on a Swiss standard on intersections without traffic lights (VSS, 1999), modification b is based on Skabardonis et al. (2014).

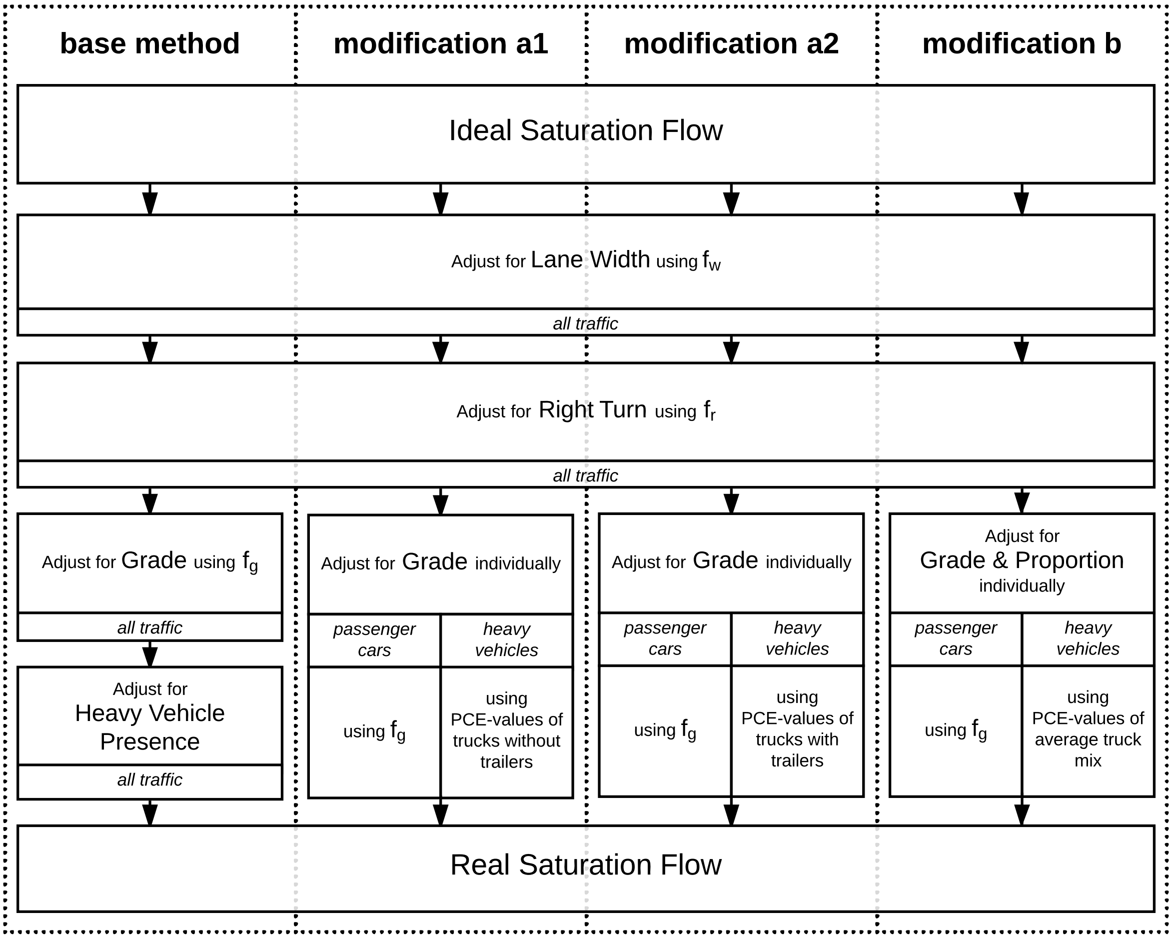
3.2.4 Additional Considerations

The side road at intersection RR and the pedestrian crossing at intersection WH are both worth considering. The respective adjustment factors are not included in the abovementioned methodology and are separately determined and discussed. This preserves the comparability of results between intersections.

[12]The $\frac{PCE_{PC}}{f_g}$ part of equation 6 is essentially a grade-adjustment of the PCE-value for passenger cars.

[13]in the case of modification b, also proportion-adjusted

Figure 2: Generalized methodology of the real saturation flow calculation on analyzed intersections for the base methods and their respective modifications based on Table 6 and Section 3.2.2



Source: own diagram

Side Road at Intersection RR

The merge at intersection RR is an unusual configuration as there is a side road in the immediate vicinity of the stop line (see Figure 15 in B.1.1). This side road is mainly used by residents and vehicles heading for or coming from the car park of a minor shopping mall in the area. For the calculation of saturation flow only vehicle movements during green phases are of importance. Vehicles that leave the side road during red (drives onto the main road with at least the front axis) are treated equally to those approaching the merge from the southern branch of Röschibachstrasse. Vehicles turning onto the side road during a red phase have no impact. However, if side road movements occur during green phases, the following effects may be observed:

- Vehicles that enter the main road from the side road during a green phase[14] must yield to the regular traffic as the southern branch of Röschibachstrasse has the right-of-way. Therefore, vehicles attempting to enter the main road often have to wait a while. When they eventually do depart, they need to accelerate from a standing queue. If the movement occurs in the beginning of a green phase, this is likely to have little to no consequences. However, if the movement occurs in a later part of the green phase, the higher headways of vehicles that accelerate from a standing queue (Pitzinger, 1996) may decrease discharge flow (consider also section 2.1.5). Additionally, vehicles approaching on the main (southern) leg of Röschibachstrasse may be forced to slow down, potentially resulting in a chain reaction with the odd vehicle in the upstream actually coming to a full stop.

- Vehicles leaving the main road by turning onto the side road during a green phase[15] causes gaps in the discharge flow on the intersection. Due to the small distance between side road and stop line, subsequent vehicles are unlikely to be able to close the gap in spite of it allowing them to drive at higher speed. Hence, vehicles turning onto the side during green are expected to decrease saturation flow.

- The gaps in the discharge flow caused by vehicles turning onto the side road may be utilized by vehicles aiming to leave the side road. This reduces the negative impact of both movements as the gap is filled again and the entering vehicle is less likely to hamper traffic flow on the main road.

To summarise, vehicle movements onto/from the side road have a mostly negative impact on the discharge flow of the intersection. There is no mention of these side roads effects on saturation flow in any of the reviewed standards (see Table 15 in Appendix A). However, based on a 1990 study by McCoy and Heimann, an adjustment factor for saturation flow of 0.96 is determined. The calculation of this adjustment factor as well as further information on the McCoy and Heimann paper are found in Appendix B.2.3. Its effect on saturation flow estimates is discussed in Section 4.3.4.

Pedestrians at Intersection WH

The effect of concurrent pedestrian traffic on saturation flow of a turning lane is addressed in all three reviewed standards. As concurrent pedestrian traffic is observable at intersection WH, it would be interesting to include the respective factors in the comparison of calculated and measured flows. However, due to limited available data on saturated flows at intersection WH (see Section 3.3.2), no saturated green phases with pedestrian blockage are observed. Therefore,

[14] About 2% of vehicles crossing the relevant stop line, according to own measurements

[15] About 3% of vehicles approaching from the southern leg of Röschibachstrasse, according to own measurements

the application of a pedestrian adjustment factor for the calculated saturation flows is not expedient. As such, the analysis on this topic is only done for the sake of completeness, conducted at the example of the HCM method[16] as documented in Equation 7.

$$f_{\text{ped,HCM}} = 1 - \frac{q_{\text{ped}}}{2000} \tag{7}$$

where

$f_{\text{ped,HCM}}$	=	adjustment factor for saturation flow according to the HCM (TRB, 2010)
q_{ped}	=	pedestrian flow per hour

To estimate the adjustment value, pedestrian traffic across the street Hofackerstrasse is measured. In the analyzed time frame (see Table 22 in B.3), pedestrian traffic is low with a maximum count of 20 pedestrians per hour. According to the HCM methodology, this results in an insubstantial adjustment factor of 0.99. Regardless of its insignificance, the pedestrian adjustment is not further discussed, due to the above-mentioned aspects.

3.3 Real Saturation Flows

In order to evaluate the accuracy of the projection methods, the results they compute are to be compared to the real saturation flow values. However, these empirical values need to be determined first. This section addresses this process and discusses encountered difficulties.

3.3.1 Available Data

According to reviewed literature (Pitzinger, 1996; TRB, 2010; Shao et al., 2011) the most common approach to determine saturation flow is to manually measure the headways of discharging vehicles (excluding the first four or five vehicles each phase[17]). As headway is the inverse of flow, during saturated green phases, this method yields a value for saturation flow. A key advantage of this measurement method is its simplicity. However, it comes at the cost of either small sample size or high expenditure of time for measurements. Furthermore, manual measurements will only yield results for the analyzed time. As discussed in Section 2.2, saturation flow depends on influences that may change over time. As such, short measurements may suggest a biased saturation flow value. With the ultimate goal of finding a methodology that takes such factors

[16] The VSS (1997) method also presents an adjustment factor for saturation flow, the HBS adjusts usable green time instead (FGSV, 2015).

[17] The first few vehicles that discharge during a green phase usually have to accelerate from a standing queue. According to numerous sources, these vehicles depart at above-average headways (see Section 2.1.5)

into account, in this thesis, the determination of saturation flow is approached using detector data.

Table 8: Position and inaccuracy of the available detectors. Inaccuracy is based on verification measurements conducted by the DAV which lasted 30 minutes per detector.

Intersection	Designation	Inaccuracy [%]	Average Distance to Stop Line [m]	Length [m]
RR	D15	4.3	upstream, 20	1.5
	D16	3.7	upstream, 1	0.5
PD	D14	1.6	downstream, 11	1.5
WH	D11	2.9	upstream, 6	1.5

Source: DAV (2017), own measurements

In cooperation with the DAV, data on traffic counts[18] and the detector occupancy is collected. This data is aggregated in 3-minute-intervals, the highest temporal resolution available. Additionally, information on the number of green phases and the green time during the same intervals is gathered. With this data, effective green time during the 3-minute-intervals is calculated by adding one second to the green time for each green phase that ended within the interval (see Section 2.1.5 and Pitzinger (1996)). Each interval's average flow rate is then determined by dividing the observed traffic count by the calculated effective green time.

Overall, data from 16 workdays[19] between 6 am and 9 pm is analyzed. Some minor data cleaning is applied to remove intervals where data seems unreliable[20]. Intervals shortly before and after are also removed. Information on the relevant detectors is shown in Table 8.

3.3.2 Determining Saturation

Since this thesis focuses on saturation flow, only flow rates from saturated phases are of interest. As the data is aggregated in 3-minute-intervals, only intervals that are saturated over their entire green time are of interest. Thus, a method must be devised to determine saturation of intervals.

Video Analysis

A method without any field measurements is tested but is rejected as it yields no reliable results (consider Appendix B.3 for further information). Therefore, saturation of the relevant approach

[18] At intersection RR, where two detectors are available, traffic count is used from D16, which is closer to the stop line (see Table 8).

[19] From 8th of may to 30th of may, excluding weekends and holidays

[20] e.g. an occupancy of 100% is reached while traffic count is not equal to one vehicle

lanes is determined by analyzing it in the field. The same 3-minute-intervals are used as the above mentioned detectors and traffic lights use. The intersections are filmed and the videos are then analyzed. Consider Table 22 in Appendix B.3 for information on the video analysis. The following aspects are considered in the analysis:

- *Saturation*
  The primary objective of the analysis is to determine saturation for each 3-minute-interval. Saturation is confirmed if and only if all green phases of the interval are saturated. A green phase counts as unsaturated if demand is satisified before the light turns yellow or there are gaps in the traffic stream that are not explainable by other factors such as heavy vehicle presence.

- *Heavy vehicle traffic*
  For saturated intervals, the amount of heavy vehicles[21] is counted in order to calculate their average share on the total traffic volume in saturated times.

- *Side road movements*
  Movements onto or from the side road at intersection RR during green phases of the relevant approach are counted for each interval.

- *Pedestrian movements*
  At intersection WH the amount of pedestrians that use the relevant pedestrian crossing in each interval is counted.

On intersections RR and PD, a total of 52 (RR) respectively 29 (PD) saturated 3-min-intervals is observed in the field analysis. Intersection WH, however, was never saturated for an entire 3-minute-interval during the field analysis. Therefore, at intersection WH, saturation flow can not be derived from detector data and is instead determined by manually measuring flow rate during saturated green phases that occur in the videos[22].

In order to improve representation of the impact of changing conditions, flows of 3-min-intervals where no videos exist are to be taken into account. At intersection PD, no such method could be devised, as the relevant lane lacks an upstream detector.

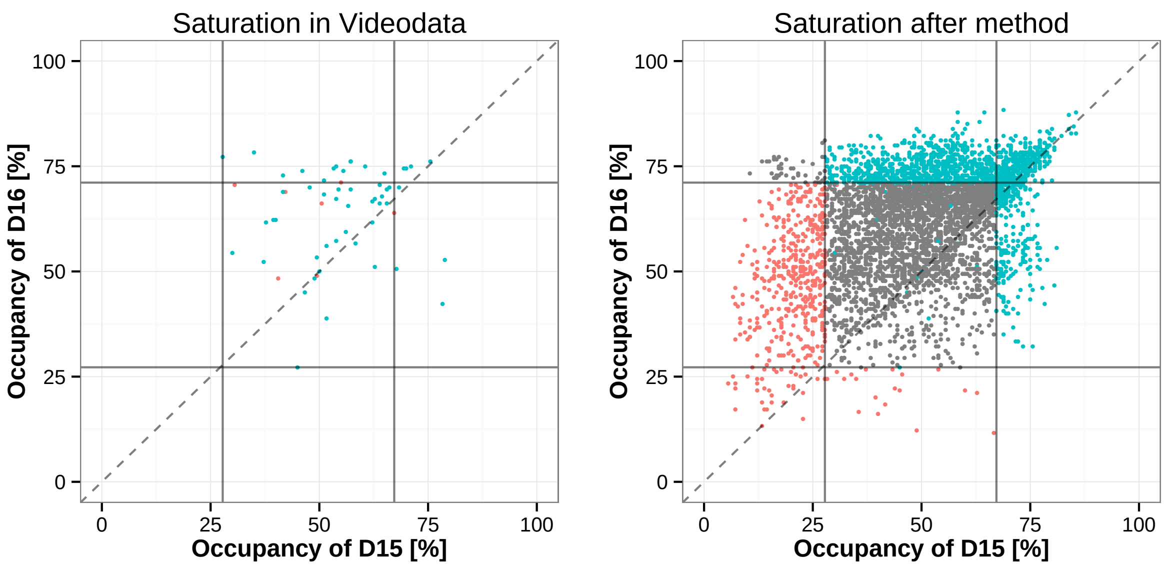
Increasing Sample Size for Intersection RR Using Detector Occupancy

For intersection RR, a method is developed that identifies additional saturated intervals. The

[21] Defined as any vehicle with more than four tires touching the ground (see Section 2.1.4)
[22] Only green phases without pedestrian traffic are considered.

method is based on detector occupancy and requires two detectors in the relevant lane's upstream. It further requires a reference sample of saturated and unsaturated intervals (here acquired in the field analysis). It compares the occupancy values of all intervals to the occupancy values of the reference sample. It then determines saturation using the following threshold criteria: an interval counts as saturated if *its occupancy value on at least one detector is higher than the reference sample's maximum occupancy observed for unsaturated intervals on the respective detector* while *its occupancy value at the other detector is not below the reference sample's minimum occupancy observed for saturated intervals on the respective detector* (see Figure 3). A more comprehensive explanation as well as additional information on the methodology can be found in Appendix B.3.3.

Figure 3: Method using detector occupancy to increase sample size at intersection RR. Excerpt of Figure 19 in Appendix B.3.3.
Blue: saturated 3-min-intervals
Red: unsaturated 3-min-intervals
Gray: unspecified 3-min-intervals



Source: own plots

The method increases the sample size of saturated intervals at intersection RR from 52 (analyzed in the video) to 1574 intervals (data from 16 working days[23]). This massive increase in sample size improves the reliability of the subsequently determined saturation flow value. It also better represents the changing conditions at the intersection as intervals of different times of day are included (see Section 5.4).

[23]Would lead to a bigger increase in sample size if more days were analyzed.

Intermediate Summary

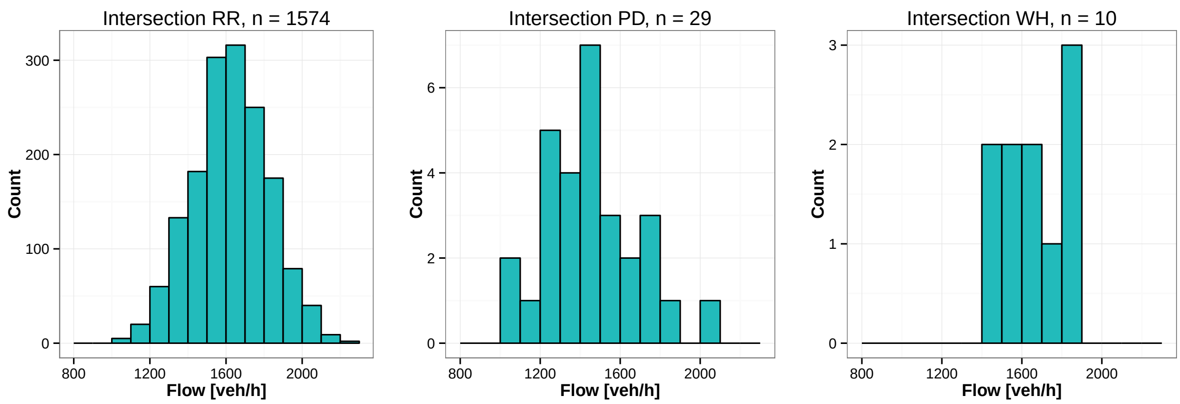
Measured saturation flow values base on 10 saturated green phases at intersection WH, on 29 saturated 3-min-intervals at intersection PD and on 1574 saturated 3-min-intervals at intersection RR.

3.3.3 From Saturated Flows to *Saturation Flow*

Figure 4 shows the frequency of flows during saturated intervals[24] as determined by the methodologies presented in the previous chapter. At this point the question arises how saturation flow is now to be determined based on these distributions. For this purpose, let us review the definitions of saturation flow that are found in the considered standards:

VSS	(VSS, 1997)	The highest discharge flow from an approach lane that is observable during the lane's effective green time.
HCM	(TRB, 2010)	The expected average discharge flow for a saturated approach lane during the lane's green time.
HBS	(FGSV, 2015)	The amount of vehicles that could discharge from an approach lane in an uninterrupted green phase of one hour.

Figure 4: Histograms of flows during saturated (blue) intervals according to the methodologies discussed in Section 3.3.2



Source: own plots

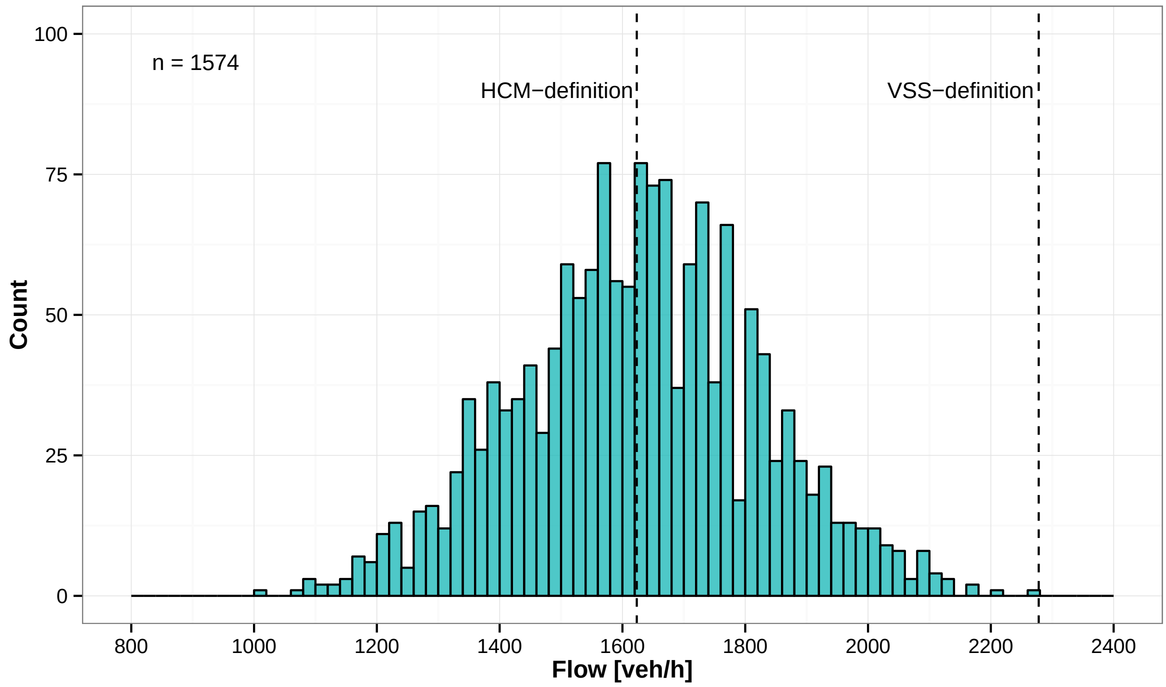
Literal interpretation of the definitions shows the differences in the understanding of saturation flow. The Swiss VSS standard defines it as the highest observed flow rate, the US HCM as the average expected value, and the German HBS approach, albeit dependent on interpretation[25],

[24] green phases in the case of intersection WH

[25] The described scenario of a consecutive hour of green time is most likely not meant to be taken literally as according results would not describe ordinary conditions. As a result, the HBS definition is subject to interpretation.

would most likely suggest something in between. Figure 5 shows just how big the difference is between the VSS- and the HCM-definitions[26] at the example of intersection RR.

Figure 5: Distribution of saturated (blue) discharge flow rates at intersection RR and the resulting *saturation flow* values according to VSS- and HCM-definitions



Source: own plot

For the purpose of smooth traffic operation at an intersection, it is important that green time is allocated in a way that minimizes the occurrence of inefficiencies. Underestimation of an approach lane's saturation flow induces too much green time for that approach. This results in lost green time and may cause congestion problems at other approaches. Conversely, overestimation of saturation flow induces too little green time thereby causing congestion in the immediate upstream and possibly spill-back in the extended upstream of the relevant approach.

It is thus evident that in general the value for saturation flow is to be determined so that both types of inefficiencies are minimized. Therefore, saturation flow should be defined as the average flow during saturated intervals, which fully corresponds to the HCM-definition.

In some occasions one inefficiency may be less adverse than the other. For example, it may prove reasonable to accept the risk of wasted green time to prevent spill-back in an upstream intersection. In such cases, a lower (or higher, depending on the scenario) saturation flow value could be used. The value determined with the VSS-definition as well as the minimum observed

[26]The HBS definition is no further discussed, due to its ambiguity.

flow during saturated phases may further be useful when extreme values are of interest. For instance, the VSS-definition of saturation flow could be used to determine the minimum reached service level for given green durations.

However, using the minima and maxima of measured saturated flows has the disadvantage that their values strongly depend on the analyzed time frame. Short measurements are expected to yield restrained values, thereby underestimating the extremes' deviation from the mean. Conversely, long measurements are prone to overestimating this deviation as a longer duration increases the risk of including measurements where irregularities such as detector malfunctions skew the measured flow[27]. For long measurements, it may thus be more reasonable to calculate boundary values (e.g. minimum reached service level) by using the mean and a desired multiple of the distribution's standard deviation[28].

To summarize, as explained above, the VSS-definition is not suitable for the general dimensioning and operation of intersections[29]. Instead, the average flow during saturated intervals should be used, as defined in the HCM.

[27] While the probability of observing irregularities per time interval remains constant, the probability to include time intervals with irregularities increases with measurement duration.

[28] This is further supported by the observation that the measured saturated flows follow a normal distribution (see Section 5.1).

[29] Although the VSS definition of saturation flow was shown to be unsuitable, the actual results of the VSS-projection method are fairly accurate, as is shown in Chapter 4.

4 Comparison of Projected and Measured Saturation Flows

In this chapter, the accuracy of the considered projection methods (see Section 3.2) is determined by comparing their results to the measured saturation flows. Sections 4.1 and 4.2 give an overview on the results of calculated and measured flows. The accuracies of the analyzed projection methods relative to the measured flows are discussed individually in Section 4.3 and drawn together in Section 4.4 with consideration of each method's average absolute deviation.

4.1 Projected Saturation Flows

Table 9 shows the calculated saturation flow values of the projection methods as well as the respective total adjustment factors applied. For explanation of the calculation methodologies, consider the preceding sections. For information on individual adjustment factors, consider Table 23 in Appendix C. The shown results are discussed in the following sections.

Table 9: Adjusted saturation flow values as calculated by the analyzed projection methods as well as used initial flow value and total adjustment factor.

	avg_{VSS}		VSS				HCM				HBS			
Intersection **RR**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
Total Adjustment [-]		0.97	0.80	0.79	0.74	0.78	0.80	0.78	0.73	0.77	0.77	0.76	0.72	0.75
Adjusted Saturation Flow [veh/h]	1800	1760	1610	1580	1490	1570	1520	1490	1390	1470	1550	1520	1430	1510

	avg_{VSS}		VSS				HCM				HBS			
Intersection **PD**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
Total Adjustment [-]		0.92	0.77	0.79	0.72	0.72	0.77	0.77	0.71	0.71	0.76	0.76	0.71	0.71
Adjusted Saturation Flow [veh/h]	1800	1650	1530	1550	1430	1430	1460	1460	1350	1350	1520	1530	1420	1420

	avg_{VSS}		VSS				HCM				HBS			
Intersection **WH**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
Total Adjustment [-]		0.99	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.81	0.82	0.82	0.82	0.81
Adjusted Saturation Flow [veh/h]	1800	1780	1640	1650	1650	1630	1550	1560	1560	1550	1640	1640	1640	1630

Source: own calculations based on VSS (1997), VSS (1999), VSS (2008), TRB (2010), Skabardonis et al. (2014) and FGSV (2015) in order of publication

4.2 Overview

As discussed in Section 3.3.3, the most suitable empirical value to use for saturation flow in the designing and operation of intersections is the mean of measured saturated flows. Therefore, Table 10 shows the resulting projected saturation flow for each projection method as well as each value's deviation from the mean of measured saturated flows. For the sake of completeness, the table also contains information on the median and the maximum observed flows during saturated intervals[30]. To put the deviations of the obtained results in perspective, detector inaccuracy is also included for each intersection[31].

Table 10: Comparison of projected and measured saturation flows. Shows deviation from the average measured saturated flow. Flow values are rounded to the nearest whole multiple of 10, deviation values to the nearest whole number.

	Projected Saturation Flow														Measured Saturation Flow		
	avg_{VSS}		VSS				HCM				HBS						
	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b	mean	median	max
Intersection RR	1800	1760	1610	1580	1490	1570	1520	1490	1390	1470	1550	1520	1430	1510	1620	1630	2280
Deviation from mean [%]	11	8	-1	-2	-8	-3	-6	-8	-14	-9	-5	-6	-12	-7	-	0	40
Detector inaccuracy = 3.7%																	
Intersection PD	1800	1650	1530	1550	1430	1430	1460	1460	1350	1350	1520	1530	1420	1420	1450	1410	2000
Deviation from mean [%]	25	14	6	7	-1	-1	1	1	-7	-7	5	6	-2	-2	-	-2	38
Detector inaccuracy = 1.6%																	
Intersection WH	1800	1790	1640	1650	1650	1630	1550	1560	1560	1550	1640	1640	1640	1630	1630	1670	1800
Deviation from mean [%]	10	10	1	1	1	0	-5	-4	-5	-5	0	1	1	0	-	2	10
Detector inaccuracy = 2.9%																	

Source: own calculations based on VSS (1997), VSS (1999),VSS (2008), TRB (2010), Skabardonis et al. (2014) and FGSV (2015) in order of publication

4.3 Evaluation

4.3.1 Method avg_{VSS}

Submethod $avg_{VSS,0}$ uses an average of 1800 veh/h for saturation flow on any and all intersection lanes, regardless of prevailing conditions. Submethod $avg_{VSS,adj}$ use the same value but adjusts for heavy vehicle presence, thereby consistently yielding lower values for projected saturation flow than $avg_{VSS,0}$. As Table 23 in C shows, the difference between the two submethods is only relevant for substantial heavy vehicle shares.

[30] green phases in the case of intersection WH

[31] At intersection WH, the inaccuracy of the detector is in principle irrelevant as saturation flow was not determined using detector data. However, it may still serve as an indicator.

The two methods have the obvious advantage of their simplicity, especially submethod $\mathrm{avg}_{\mathrm{VSS,0}}$, where no adjustment is made at all. However, as evident in Table 10, at the analyzed intersections the methods fail to accurately estimate saturation flow. The projected results overestimate real saturation flow by 8 to 14% ($\mathrm{avg}_{\mathrm{VSS,adj}}$) and 10 to 25% ($\mathrm{avg}_{\mathrm{VSS,0}}$). As such, the two submethods consistently provide worse estimates than the considered base methods. The best estimate provided by submethod $\mathrm{avg}_{\mathrm{VSS,adj}}$ (8% overestimation) is still off by about 140 veh/h which is a substantial amount. Submethod $\mathrm{avg}_{\mathrm{VSS,0}}$ is even less accurate.

The failure of these submethods to accurately estimate saturation flow at even just one of the analyzed intersections questions their applicability. The lack of adjustment for lane width, grade and right turn movement seems to come at the cost of substantial and consistent overestimation. This is supported by the fact that adjustment for heavy vehicle presence in $avg_{VSS,adj}$ leads to better estimates in comparison to $\mathrm{avg}_{\mathrm{VSS,0}}$.

In light of the described overestimation, the usage of the two submethods for the estimation of saturation flows on right turning lanes is not advised[32]. Considering that submethod $\mathrm{avg}_{\mathrm{VSS,0}}$ is in fact the more commonly used method (DAV, 2017), a change in methodology seems all the more appropriate.

4.3.2 Base Methods

Overall, the projected saturation flow rates calculated with the base methods are fairly accurate. The absolute deviations from the mean of measured saturation flows are never higher than about 6% (see Table 11), which corresponds to 80-100 veh/h or to a maximum of about two to three times the respective detector inaccuracy.

Table 11: Deviation of projected saturation flows calculated according to the base projection methods at the analyzed intersections. Based on Table 10.

Intersection	Projection Methods $\mathrm{VSS}_{\mathrm{base}}$	$\mathrm{HCM}_{\mathrm{base}}$	$\mathrm{HBS}_{\mathrm{base}}$
RR	-1%, within detector inaccuracy	-6%, underestimation	-5%, underestimation
PD	+6%, overestimation	+1%, within detector inaccuracy	+5%, overestimation
WH	+1%, within detector inaccuracy	-5%, underestimation	+0%, within detector inaccuracy

Source: Table 10 in Section 4.2

[32]The results indicate that the two submethods may fail to predict saturation flow for other types of intersection lanes as well (e.g. left-turning lanes). As this can not be confirmed with the presented analysis, this research refrains from an according statement.

As evident in Table 11, the VSS_{base} method predicts saturation flow within detector inaccuracy at intersections RR and WH. At intersection PD it overestimates saturation flow. The HBS_{base} method, like the VSS_{base} approach, is highly accurate at intersection WH and overestimates flow at PD. At intersection RR, however, it underestimates flow rate. The HCM_{base} method underestimates saturation flow at both RR and WH but yields a highly accurate result at intersection PD. It is thus apparent that no base method is consistently better than the other two. However, the following observations are noteworthy, based on Table 11:

1. The HCM_{base} method consistently yields the lowest estimates of saturation flow at all intersections.
2. The VSS_{base} method consistently provides the highest estimates of saturation flow at all intersections.
3. At intersection RR, each base method provides its most underestimating value for saturation flow.
4. At intersection PD, each base method yields its most overestimating value for saturation flow.

Observation (1) is most likely a direct result of the difference in ideal saturation flow (2000 PCE/h in VSS and HBS, 1900 PCE/h in HCM). The calculation of the adjusted saturation flow estimate is based on these initial values as described in Table 9 in Section 4.1. Table 9 further shows that the total adjustment factor used in the HCM_{base} method is almost equal to those of the other methods, thereby reinforcing the assumption that the lower initial value used in the HCM_{base} method is the cause of the consistently lower estimates.

Observation (2) is partially (difference of the VSS_{base} and the HCM_{base} methods) explained by the same argument as observation (1). The difference of the VSS_{base} and the HBS_{base} methods is only substantial at intersection RR. A look at the used adjustment factors in Table 23 in Appendix C shows that the difference could be due to different weighting of lane width or gradient effects. The data does not allow for a definite statement on this issue, however.

Observation (3) has no obvious explanation. It is noteworthy that intersection RR not only has the highest grade but also the highest lane width and the biggest turn radius (see Table 4 in Section 3.1.2). Any of these factors could be the cause of the low estimates and the cause may differ between standards. For instance, the HCM_{base}'s constant adjustment factor for all right turn regardless of respective radii is likely to overestimate the turn effect at this intersection. Conversely, the HBS_{base}'s strong weighting of gradient effect could explain why it yields an underestimating value for flow at this intersection. The same argument might explain the relatively low estimate of the VSS_{base} method.

Observation (4) is not explicable with certainty either. However, intersection PD has the biggest proportion of heavy vehicles by a large margin while neither lane width, nor gradient, nor turn radius particularly stand out. Therefore the high estimates for saturation flow at the intersection could be caused by underweighting of heavy vehicle effects.

4.3.3 Modifications

The in Section 3.2.2 introduced modifications to the base methods aim to take the synergies of heavy vehicle presence and road gradient into account. The following paragraphs discuss the effect of these modifications on the accuracy of the projected saturation flow values.

Table 12: Deviation of projected saturation flows calculated according to the analyzed modifications. Based on Table 10.

	VSS				HCM				HBS			
Intersection	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
RR	-1%	-2%	-8%	-3%	-6%	-8%	-14%	-9%	-5%	-6%	-12%	-7%
PD	+6%	+7%	-1%	-1%	+1%	+1%	-7%	-7%	+5%	+6%	-2%	-2%
WH	+1%	+1%	+1%	+0%	-5%	-4%	-5%	-5%	+0%	+1%	+1%	+0%

Source: Table 10 in Section 4.2

Special Case of Intersection WH

At intersection WH, the difference in accuracy between the base methods and the modifications is never higher than 1 %-point (see Table 12). This is due to the very low proportion of heavy vehicles on the total traffic stream (0.7%). The lack of change from the base methods to the modifications shows that the modifications perform similarly to the base methods for low heavy vehicle shares. As the data on intersection WH does not allow for further conclusions, it is henceforth excluded from analysis.

Modification a1

Modification a1 calculates the effect of heavy vehicles as described by the Swiss standards (VSS, 1999) with grade-adjusted PCE-values for trucks *without* trailers. For further information on the methodology, consider Section 3.2.2 and Appendix B.2.

Table 12 shows that the deviations of projected saturation flows calculated in accordance with modification a1 differ only slightly (0-2 %-points) from the results obtained with the base methods. In most cases, the modified method yields a less accurate result. Saturation flow values that were already overestimated in the base method are estimated to be even higher in the modified method and vice versa for underestimated flows. As such, modification a1 usually provides worse estimates of saturation flow than the base methods.

Modification a2

Modification a2 calculates the effect of heavy vehicles as described by the Swiss standards (VSS, 1999) with grade-adjusted PCE-values for trucks *with* trailers. For further information on the methodology, consider Section 3.2.2 and Appendix B.2.

As evident in Table 12, modification a2 consistently yields lower estimates for saturation flow than its counterpart a1. This is due to the higher PCE-values that are used in modification a2 (see Table 19 in Section 3.2.2). Compared to the base methods, modification a2 consistently provides estimates that are lower by about 7-8 %-points. This could be caused by overly high PCE-values for heavy vehicles. The method improves estimates in cases where the base methods substantially overestimate saturation. However, in cases where the base method is highly accurate or even underestimating, modification a2 leads to less accurate results than the base methods.

Modification b

Modification b calculates the effect of heavy vehicles as described by Skabardonis et al. (2014) with grade-and-proportion-adjusted PCE-values for an average truck mix. For further information on the methodology, consider Section 3.2.2 and Appendix B.2.

The projected saturation flows of modification b are 2-3 %-points lower than the estimates of the base methods at intersection RR and 7-8 %-points lower at intersection PD (see Table 12). The difference in impact is a result of the different proportions of heavy vehicles (2.6% at RR, 9.0% at PD). At both intersections, the reduction in resulting saturation flow value is consistent. The lower estimates lead to higher accuracy where the base methods substantially overestimate saturation. Much like modification a2, however, modification b leads to less accurate results in cases where the base method is highly accurate or already underestimated.

4.3.4 Additional Considerations

The following paragraphs refer to the additionally considered side road effects at intersection RR and the pedestrian traffic at intersection WH as discussed in Section 3.2.4.

As mentioned in Section 3.2.4, the calculated adjustment factor for side road presence based on (McCoy and Heimann, 1990) is 0.96. This corresponds to a reduction of 4%. The applicability of this factor is questionable (see Appendix B.2.3). A look at Table 12 shows that flows are consistently underestimated by base methods and modifications (albeit insubstantially in some cases), even without further reduction. Inclusion of the side road factor would thus only lead to further underestimation.

As described in Section 3.3.2, no saturated 3-min-intervals could be determined at intersection WH. The average measured value of saturation flow is thus based on saturated green phases. The considered green phases experienced no blockage by pedestrians, which is why an adjustment of the projection methods' estimates for pedestrian traffic is no further discussed.

4.4 Synthesis

In this section, found results, reasons and learnings are summarized. The argumentation is supported by Table 13, which shows the average absolute deviation between the calculated saturation flows in the 14 analyzed projection methods and the mean of measured saturated flows.

Table 13: Average absolute deviation of the projected saturation flows from the mean of measured saturation flows at all considered intersections. Based on Table 10.

Projection Methods	avg_{VSS}		VSS				HCM				HBS			
	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Average Absolute Deviation [%]	15	11	2	4	3	1	4	5	9	7	3	4	5	3

Source: own calculations based on Table 10 in Section 4.2

Both submethods $avg_{VSS,0}$ and $avg_{VSS,adj}$ consistently yield inaccurate estimates of saturation flow at all intersections. They overestimate saturation flow by a substantial amount. Their projected values are consistently worse than the estimates of all base methods which is also reflected in the estimates' average absolute deviation from the mean measured value (see Table 13).

The three base methods generally deliver good estimates of saturation flow. Four of the nine estimates are actually within detector inaccuracy. The high accuracy is also apparent in Table 13, which shows that, on average, the methods deviate only 2-4% from the mean of measured saturated flows. The differences in these average absolute deviations are insubstantial. No base method consistently provides better estimates than the other two. In fact, each of the three base methods yields the most accurate result at one of the three intersections.

The modifications to better take synergies of gradient and heavy vehicle effects into account are shown to have no substantial effect on the estimate of saturation flow when heavy vehicle proportions are low. Modification a1 generally provides estimates that are slightly worse than the results of the base methods. This is reflected in Table 13, which shows that modification a1's average absolute deviation is consistently higher than the respective base method. The same applies to modification a2, albeit with higher variability. The modification's estimates are consistently lower than the base methods, thereby decreasing accuracy of already accurate or underestimating base methods. Modification b yields lower estimates than the base methods as well. However, its estimates are consistently more accurate than those of modification a2. In the case of the VSS approach, modification b even has a lower average absolute deviation than the base method. Additional adjustment for side road effects at intersection RR reduces accuracy of the base methods and modifications alike.

In conclusion, the base methods, as described by the considered standards, are a valid tool for estimating saturation flow on right turning lanes[33]. The tested modifications generally decrease accuracy of the estimates with the exception of modification b. While this modification's results improve average performance of some base methods, further tests and possibly improvements in the methodology are necessary before its application is justifiable. The projection method using a fixed average value is shown to substantially and consistently overestimate saturation flow, regardless of the consideration of an adjustment for heavy vehicle presence. As such, the usage of this methodology on right-turning lanes is strongly advised against and an according adjustment or removal of the relevant entries from the Swiss standards is recommended.

[33] On right turning lanes of higher complexity than the analyzed intersections (e.g. high pedestrian volumes, limited upstream storage space, ...), additional factors may require consideration.

5 Further Results and Discussion

In Chapter 4, with the comparison of projected and measured saturation flows, the main subject of this thesis was addressed. This chapter discusses additional topics that are relevant to the field of saturation flow estimation. In Section 5.1, the distribution of measured saturated flows is tested for normality. Section 5.2 discusses the impact of heavy vehicle presence on measured saturated flows. In Section 5.3 the effect of the green phase length on saturation flow and the adjustment to effective green are discussed. Section 5.4 analyzes changes in saturation flow over the course of the day and considers potential causes. As a supplement to the analysis of commonly used projection methods, in Section 5.5 a simplistic method to empirically estimate saturation flow is considered. Finally, the methodology and the findings of this research are challenged by analyzing the maximum flow rate that can be sustained over a consecutive hour.

For the analyses in sections 5.1 to 5.4, relatively large sample sizes are required. Therefore, in these sections, only intersection RR is considered where sample size was increased with the methodology described in Section 3.3.2.

5.1 Normal Distribution of Saturated Flows

The measured saturated flows are tested for normal distribution. A reliable analysis requires a relatively large sample size, which is why only flows at intersection RR are tested, where sample size was increased as described in Section 3.3.2. Normality of the flow distribution is tested with a QQ-plot and a Shapiro-Wilk test. The tested data is shown in plot a of Figure 6, which also shows the curve of a normally distributed reference sample with the same mean (1620 veh/h) and the same standard deviation (200 veh/h).

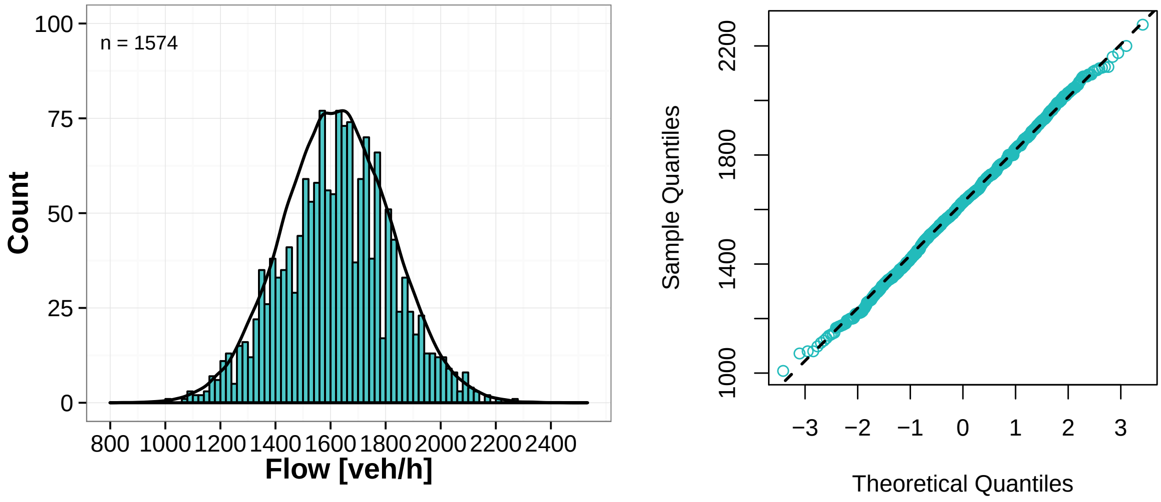
The QQ-plot in plot b of Figure 6 shows that the relation of the sample quantiles to the theoretical quantiles is mostly linear[34]. Albeit this is not definite proof, it indicates normal distribution. The Shapiro-Wilk normality test yields a test-statistic W = 0.999 with a p-value of 0.458. Since the p-value is larger than an α-level[35] of 0.05, the null hypothesis of normal distribution is not rejected. Based on the results of this test and the QQ-plot, it appears reasonable to assume normal distribution of saturated flows.

[34] excluding measurements at the fringes of the distribution

[35] An α-level of 0.05 corresponds to a 5% risk of incorrectly rejecting the null hypothesis.

Figure 6: Comparison of measured saturated flows at intersection RR to normally distributed data. Plot (a) shows a histogram of measured saturated flows (blue) in relation to normally distributed data (curve) with the same mean and the same standard deviation. Plot (b) shows a QQ-plot of the same flow data.

(a) Histogram & normal distribution (b) QQ-plot, n = 1574



Source: own plots

5.2 Heavy Vehicle Effect on Saturation Flow

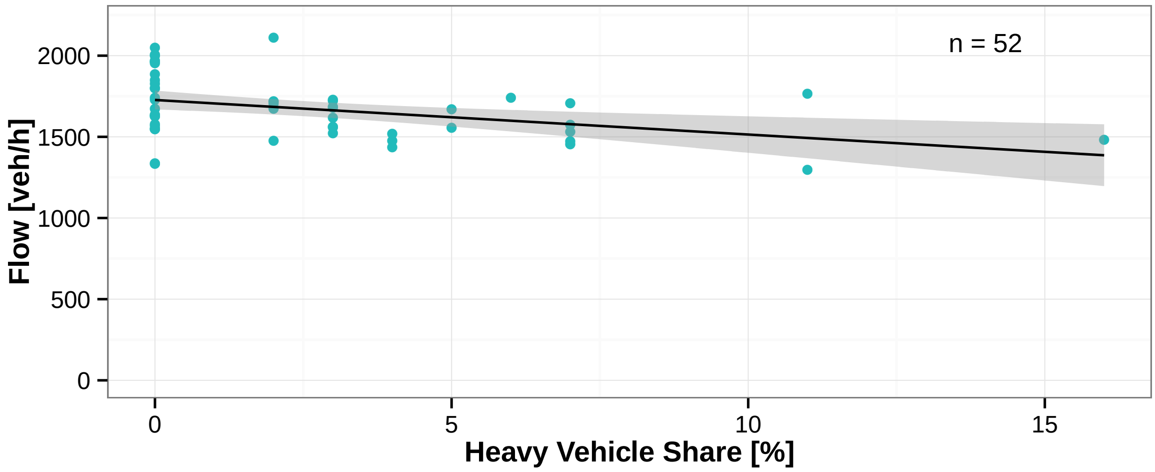
Detectors measure traffic volume in vehicles not in PCE[36]. Accordingly, the measured saturation flows are indicated in veh/h as well[37]. As heavy vehicles have PCE-values higher than 1.0 PCE/veh, measured intervals with relatively high heavy vehicle shares are expected to show correspondingly low saturated flows. Figure 7 shows that this effect is indeed observable at intersection RR. This supports that the assumption that saturated intervals with high heavy vehicle shares generally experience lower flows.

The relatively low sample size at intersection RR (52 saturated intervals) is a result of heavy vehicle shares only being detectable by manually counting in the field (see Appendix B.3.2). At intersections PD and WH, where the number of observed saturated intervals is even smaller, the above-mentioned effect cannot be confirmed.

[36]Note that the detectors may count some trucks with trailers as two vehicles. This phenomenon is not analyzed in detail but is considered a potential source of errors.

[37]The conversion from PCE/h to veh/h is also a fundamental element of projection methods that consider synergetic effects of heavy vehicles and road gradient. When not considering these synergies, saturation flow can be indicated in PCE/h. However, traffic demand must then be adjusted from veh/h to PCE/h for correct comparison.

Figure 7: Flows at intersection RR in saturated (blue) 3-min-intervals in relation to the correspondent proportion of heavy vehicles on total traffic. Includes a smoother with GAM (general additive model) configuration.



Source: own plot

5.3 Phase Length Effect on Saturation Flow

As discussed in sections 2.1.5 and 2.2.3, a saturated green phase's discharge rate depends on its duration. On the one hand, the first few vehicles depart at higher headways (Pitzinger, 1996; TRB, 2010), on the other hand, short green phases induce increased utilization of the yellow phase after the green phase (FGSV, 2009). Figure 8 shows the relation of measured saturated flows to the average phase length for each 3-min-interval at intersection RR. The plot shows that on average, there is a slight decrease in saturated flow rate with increasing average phase length. This indicates that the increased utilization of the yellow phase has a bigger impact on flow rate than the increased headway of the first few vehicles[38].

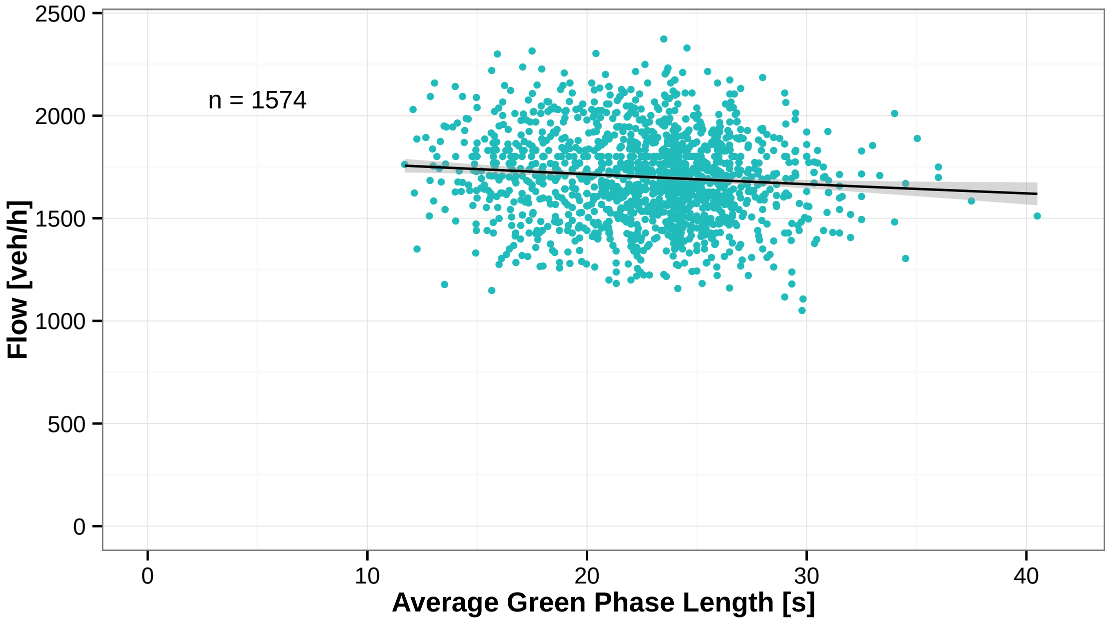
To account for the above-mentioned effects, the green time is adjusted by adding one second to each green phase (Pitzinger, 1996; FGSV, 2009). This adjusted green phase length is called the effective green time. Since flow is a function of the green time[39], the adjustment for effective green time also adjusts flow rate. Figure 9 shows the adjusted flows in relation to the effective green time for each saturated interval at intersection RR. The smoothing function in the plot shows that there is no observable correlation between the effective green time and the flow rate. This suggests that the commonly used methodology[40] to adjust green time is indeed effective.

[38] In Zürich, the minimum valid green phase length is 6 seconds (DAV, 2017). Due to the smoothing effect of the interval data, the lowest observed average green time is 12 seconds. The impacts of effects may differ for green phase lengths of 6-12 seconds.

[39] Flow is calculated by dividing the detector's vehicle counts by the respective green phase length.

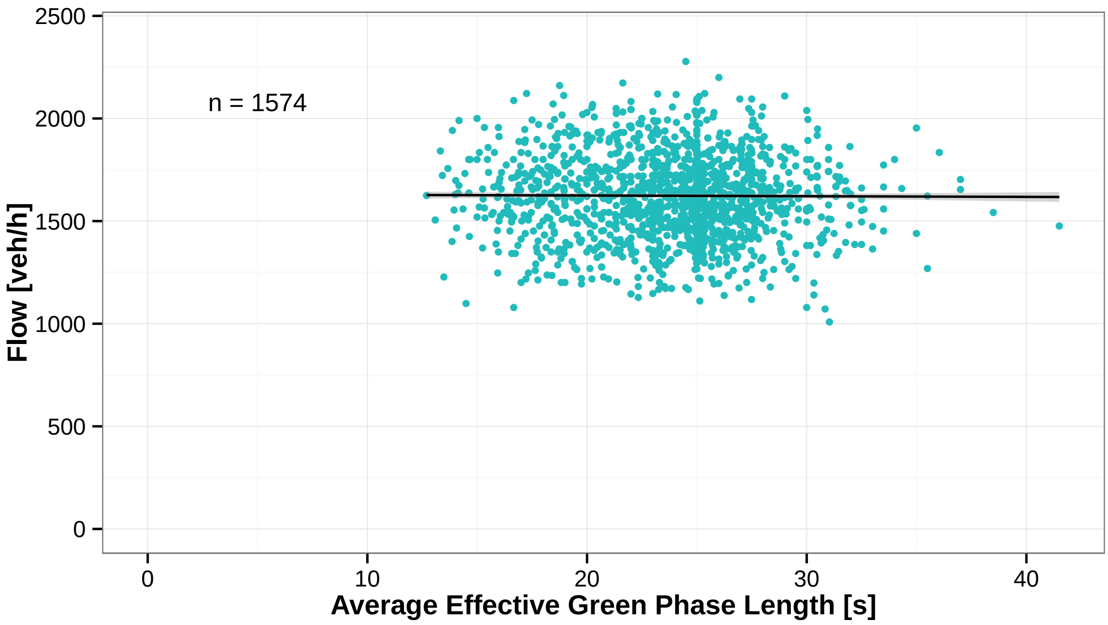
[40] As introduced in Section 2.1.5 and as used in this thesis for all flows.

Figure 8: Flow rates (unadjusted for effective green) at intersection RR during saturated (blue) 3-min-intervals in relation to the correspondent average green phase length (unadjusted for effective green). Includes a smoother with GAM configuration.



Source: own plot

Figure 9: Flow rates at intersection RR during saturated (blue) 3-min-intervals in relation to the correspondent average effective green phase length. Includes a smoother with GAM configuration.



Source: own plot

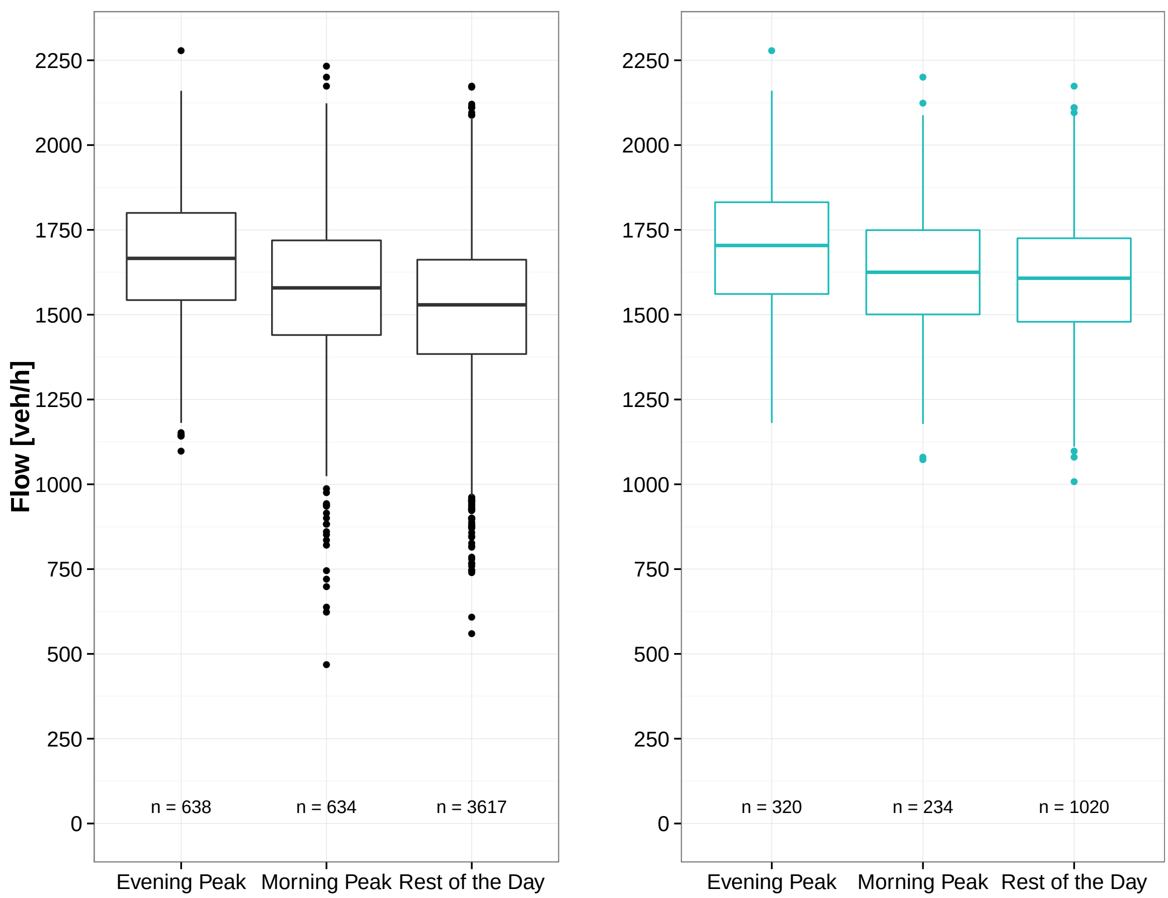
5.4 Peak and Off-peak Observations

It is a common observation in traffic management that mobility demand is highest during morning and evening commuting hours. Accordingly, on an average intersection, flows can be expected to be higher during these demand peaks as well, as more green phases are likely to be saturated. Plot (a) in Figure 10 shows that this pattern is observable at intersection RR for both morning peaks (6.30 - 8.30 am) and evening peaks (5 - 7 pm). Average flow rate over all intervals is higher during peak hours than during the rest of the day. Interestingly, evening peaks are substantially more pronounced than morning peaks.

Figure 10: Boxplots of total flows (a) and saturated flows (b) in peak hours and during the rest of the day at intersection RR

(a) Boxplot of all flows over the day

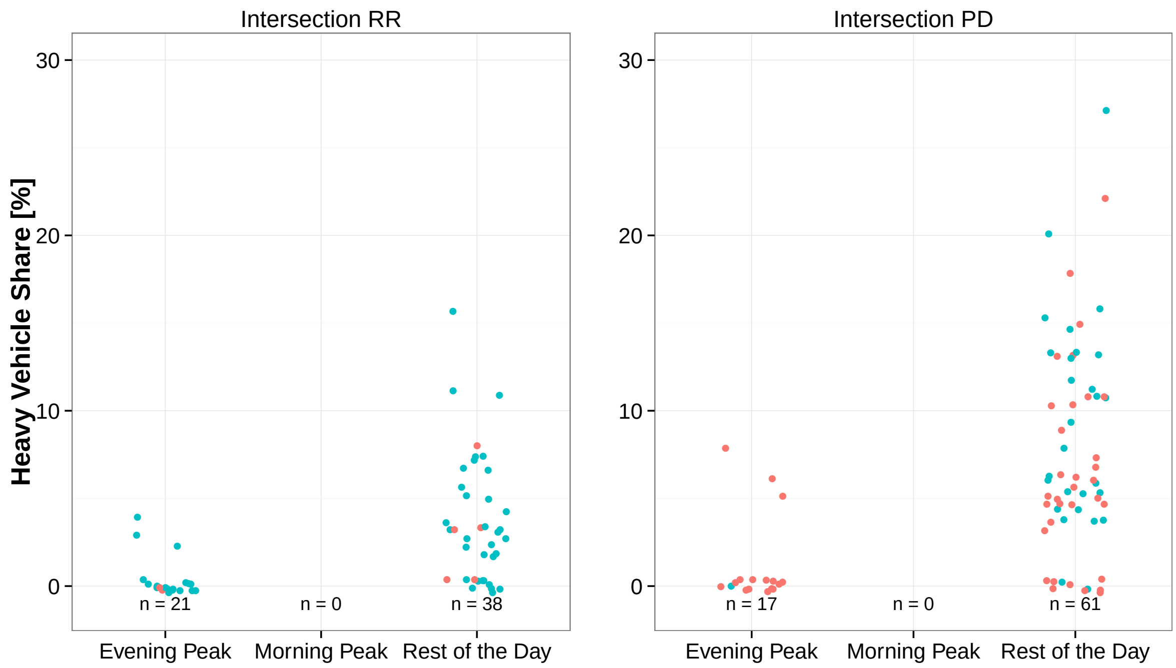
(b) Boxplot of saturated (blue) flows over the day



Source: own plots

Plot (b) of Figure 10 shows similar information as plot (a) but only takes saturated phases into account. As evident from the plot, the median saturation flow in the evening peak is substantially higher than in the morning peak or during the rest of the day[41]. This is obviously not explicable with the above-mentioned frequency of saturated flows, which is why other potential causes are examined.

Figure 11: Heavy vehicle shares at intersections RR and PD during peak hours and the rest of the day. Based on field analysis (see 22).
Blue: saturated 3-min-intervals
Red: unsaturated 3-min-intervals



Source: own plots

As discussed in Section 5.2, heavy vehicle presence reduces the measured saturation flow. Therefore, if the evening peak hours were shown to experience significantly less heavy vehicle traffic, that could be an explanation for the increased saturation flows in the evening. As the plots in Figure 11 show, this is indeed the case for both intersections where average heavy vehicle shares are above 1% [42]. At intersection RR, the average heavy vehicle share during saturated intervals in the evening peak is below 0.5%, the respective share in the rest of the day's saturated intervals is 3.7%. The observation that heavy vehicles are less frequent during the evening peaks could be attributed to the fact that processes like delivery of goods typically occur earlier during the day.

[41] The difference between the rest of the day and the morning peak is negligible.
[42] No heavy vehicle data on morning peaks available, insufficient heavy vehicles at intersection PD.

Additional explanations for higher saturation flow rates in the evening peak may be found by considering the overview in Table 2 (see Section 2.2). The most relevant factors being *prevalent trip purpose* and *driver attentiveness*, which among themselves include factors like driver experience, driver aggressiveness and familiarity with the intersection. Commuting traffic may reasonably be expected to largely consist of drivers that drive the same routes every workday for several months or years. Therefore, average driver's experience and familiarity is expected to be higher during commuting hours. Albeit not provable by this research, it also seems likely that the average commuter's desire to reach their destination is higher when that destination is their home as opposed to their workplace. As a consequence, average drivers in commuting hours are expected to drive more aggressively (higher willingness to utilize yellow or even early red phases for their departure, lower headway to preceding vehicle, . . .) than average drivers during the rest of the day.

To summarize, the difference in typical saturation flow between the evening peak and the rest of the day is likely a cause of vehicle-related (heavy vehicle presence) and driver-related (experience, aggressiveness, familiarity with the intersection) characteristics.

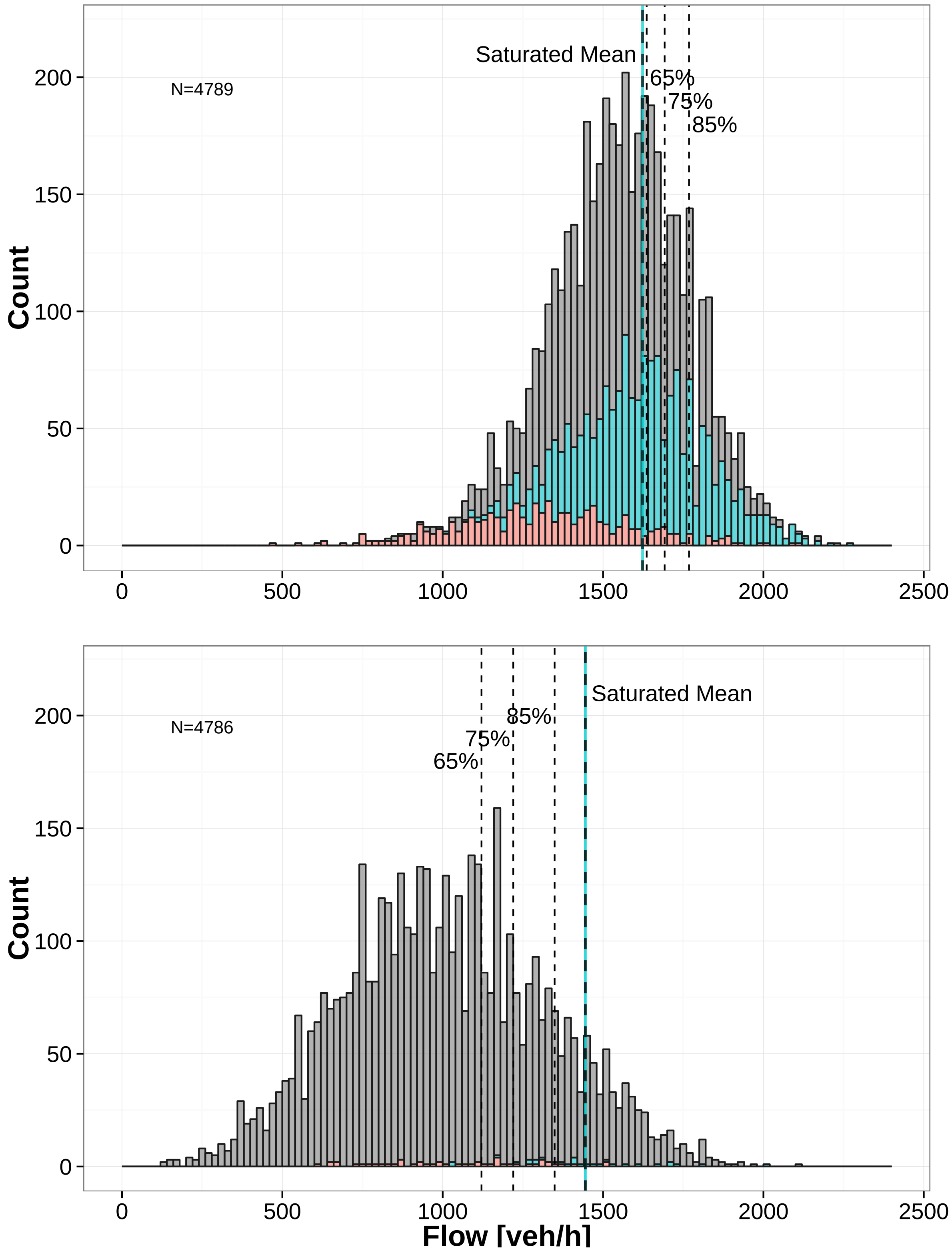
5.5 Alternative Empirical Approach - Percentile Method

In Section 3.3.3 it was shown that the average of measured saturated flows is the best value for the saturation flow used in the dimensioning and operation of intersections as it minimizes inefficiencies. This method has the disadvantage that it requires field measurements to acquire a sample of saturated intervals. If the time and money for such measurements is not available, traffic planners may resort to the discussed projection methods. Alternatively, if empirical flow data is available but the respective saturation states are unknown, planners may consider using simple empirical methods to determine saturation flow.

Among the simplest empirical methodologies are those that estimate saturation flow based on a certain percentile of all measured flows. For example, a traffic planner might assume that a flow that is only surpassed by 15% of all flows is a good dimensioning value. One would thus use the 85-percentile of all measured flows as his saturation flow estimate[43] This straightforward method has the major advantage that they require no information on the saturation of considered flows. However, their applicability for more than provisional estimates is questionable, as is shown below.

[43] Considering the extreme case of the 100-percentile, the resulting estimate corresponds to the in Section 3.3.3 rebutted saturation flow definition in the Swiss standards (VSS, 1997).

Figure 12: Comparison of percentiles of all measured flows (excl. nightly flows between 9pm and 6am) and the determined mean of saturated flows at the example of stacked flow histograms (per interval) for intersections RR (upper plot) and PD (lower plot)
Blue: saturated 3-min-intervals
Red: unsaturated 3-min-intervals
Gray: unspecified 3-min-intervals



Source: own plots

Figure 12 shows stacked histograms of all measured flows at intersections RR (upper plot) and PD (lower plot), colored by their saturation state. Also shown are the values of three arbitrary percentile-levels on the one hand and the mean of saturated flows on the other hand. As evident from the figure, the considered percentiles overestimate saturation flow at intersection RR (albeit slightly in the case of the 65-percentile) and underestimate it at intersection PD. At RR the 65-percentile provides the best estimate, at PD the 85-percentile performs better than the other two. This shows that the accuracy of percentile methods varies between intersections, regardless of the chosen percentile level.

This variability is a consequence of different saturation patterns. Intersection RR is more often saturated than PD[44]. As a result, intersection RR requires a smaller percentile level for accurate saturation flow estimates. This dependency of the methods on the intersection pattern nullifies the presumed advantage of not requiring information on the saturation pattern. Percentile methods are therefore not recommended for detailed planning purposes.

5.6 Challenging the Used Methodology

A further method to empirically estimate saturation flow from data of unknown saturation state is to calculate the highest flow rate observable during a consecutive hour. This is done by calculating the average discharge flow in the 30 minutes before and after (a desired number of) specified points in time[45]. The idea behind this approach is that a flow rate that can be measured over a full hour should be considered sustainable. Whether or not the method is suitable for traffic planners is discussed in the following.

5.6.1 Usage as Lower Boundary

A relatively obvious problem of the method is its inability to distinguish between saturated and unsaturated flows. Not unlike the above discussed percentile-method, this method will thus underestimate saturation flow at intersections that are only sporadically saturated. Consequently, if the intersection's saturation pattern is unknown, the method should not be used to estimate mean saturation flow. Nevertheless, the method may still be of use. Due to its apparent tendency to underestimate saturation flow, one might argue that the method could be used as a lower boundary for valid saturation flow estimates.

[44] Statement based on DAV (2017) and observations in the video analysis. See Table 22 in Appendix B.3.

[45] Note that with increasing temporal density of specified points the maximum measured flow is likely to increase as well. This is due to better approximating the real peak. In this thesis, the average hourly discharge flow (30 minutes before and after) is calculated every three minutes (as it is based on the 3-min-interval data). In contrast, typically available temporal resolutions to planners are limited to 15 or even 60 minutes.

Table 14: Comparison of the determined mean of saturated flows and the (average daily) maximum flow over a consecutive hour

		Intersection		
Supposed usage	Method	RR	PD	WH
Planning value	Mean saturation flow [veh/h]	1620	1450	1630
Lower boundary	All time maximum hourly flow [veh/h]	1800	1500	1110
	Average daily maximum hourly flow [veh/h]	1730	1320	1050

Source: own calculations

However, as shown in Table 14, the resulting lower boundaries are higher than the determined mean saturation flow values at intersections RR and PD. This inconsistency questions the findings of this thesis and challenges the used methodology. Therefore, the used methodology is reconsidered in light of these observations.

5.6.2 From All Time Maximum to Average Daily Maximum

The most important advantage of using the mean of saturated flows for the allocation of green time is that doing so minimizes inefficiencies at intersections (see Section 3.3.3). A further advantage is that the accuracy of mean based methods increases with longer time frames. In contrast, the presented method to determine the lower boundaries of saturation flow uses the *all time maximum* flow during a consecutive hour. Thus, the method will only re-adjust its result when a higher flow level is reached. Consequently, the method yields a one-sided result in the long-term. To address this issue, the maximum hourly flows on each day are calculated and the average of these values is taken as the new estimate for the lower boundary of valid saturation flow estimates. As evident from Table 14, the resulting lower boundary at intersection RR is still higher than the mean of saturated flows. This contradiction is discussed in the following.

5.6.3 Validity Discussion

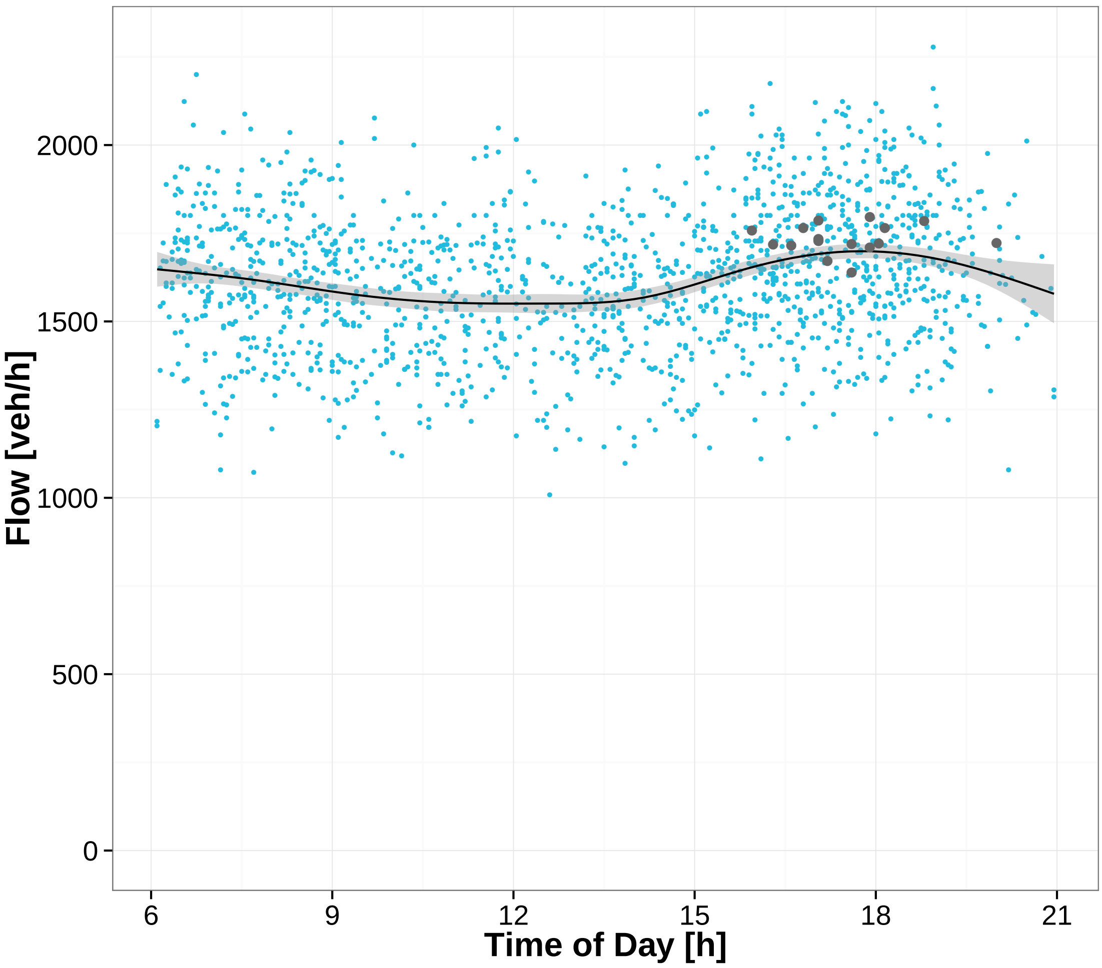
From the abovementioned contradiction it is obvious that (at least) one of the following statements is true.

1. The methodology to enhance sample size of saturated intervals, as presented in Section 3.3.2, includes too many falsely classified intervals.
2. The saturated mean is (sometimes) not a valid planning tool.

3. The average daily maximum hourly flow is (sometimes) not a suitable estimate for the lower boundary of saturation flows.

While the presented methodology to enhance sample size of saturated intervals is not unlikely to wrongly classify a few unsaturated intervals as saturated, the share of such false positives is deemed to be low (see Appendix B.3). Additionally, the mean of saturated flows as determined in the video analysis (=1680 veh/h) is also below the respective lower boundary of 1730 veh/h. Thus, a significant error in this sample-size-enhancing methodology is not only unlikely but would also not suffice to explain the discrepancies. As the mean of saturated flows was determined to be the most suitable planning value, confirmation of the second statement would void most findings of this research. Fortunately, the discrepancies can be explained by a detailed analysis of the average daily maximum hourly flow.

Figure 13: Saturated flows (blue, n = 1574) of all days at intersection RR and daily maximum hourly flows (gray, N = 16) plotted against the time of day they occur at. Includes a smoother (for saturated flows) with GAM configuration.



Source: own plot

There are two main reasons why a flow that can be sustained over a consecutive hour might be considered a good estimate for the lower boundary of valid saturation flow values. Firstly, the yielded flow rate potentially includes unsaturated intervals in its calculation and secondly, the variability of saturation flows is smoothed over an hour. While reasonable, these are not the only necessary considerations. Figure 13 shows that on every measured day, the maximum hourly flow is reached in the evening hours. This is consistent with the obvious assumption that, due to high demand, saturated intervals are more frequent during commuting hours. However, it also explains how the maximum flow over a consecutive hour can be higher than the mean saturated flow.

The discourse can be broken down into two opposing effects. On the one hand, potentially including unsaturated flows is expected to yield conservative estimates[46]. On the other hand, Figure 13 shows that saturated flows are higher during commuting hours, especially so in the evening peak. As discussed in Section 5.4, the higher flow rates are deemed to be a product of lower heavy vehicle traffic and increased driver skill and aggressiveness. The discrepancies between the mean of measured saturation flows and the supposed lower boundary for valid saturation flows can thus be explained by these vehicle-related and driver-related peak effects having a higher impact on the maximum reached flow than the mentioned potential to include unsaturated flows.

Hence, the method for determining the lower boundary of valid saturation flows fails due to its inability to sufficiently consider changes of saturation flow over the course of the day. As such, and due its dependency on the intersection's saturation pattern, its usage is not recommended for detailed planning processes.

5.6.4 Adjustment Factor for Commuting Hours

Based on the finding that the time of day influences saturation flow, an inclusion of such a factor in relevant standards seems appropriate. However, as the effect of commuting hours is but a product of other influences (see above), its impact may prove variable between locations. Ultimately, this research lacks the data to reliably determine an according adjustment factor and not enough is currently known about the variability of the impact. Therefore, no such adjustment factor is presented in this research.

[46] Due to the lower frequency of unsaturated intervals during peak hours and considering that unsaturated intervals during times of high demand (at intersection RR typically caused by suboptimal operation of upstream intersections) are likely to be less strongly undersaturated than those in times of low demand, this effect is expected to be relatively minor.

6 Conclusion and Outlook

The target of this research was to provide a scientific foundation for discussions on what methods to predict saturation flow are reasonable to use in the planning and operation of intersections. This target is reached as the conducted analyses allow for statements on the accuracy of various projection methods. Although, the research was limited to right-turning lanes, the findings may be indicatory of other lane types as well. In this section, the drawn conclusions are collated and topics with further research potential are discussed.

Methodology

In Chapter 3, a methodology was devised to increase the sample size of saturated (and unsaturated) intervals based on the detectors' occupancy levels. The method not only induced a broader-based value of saturation flow but also allowed for various otherwise impossible or unreliable analyses (see Chapter 5). This method requires two detectors in the upstream intersection lane and field observations as reference points. Its benefits are therefore limited to such intersection lanes. An analysis of how this methodology could be improved with the consideration of other intersections could be an interesting topic for further research. Ultimately, a modified methodology which is valid at intersections with just one upstream detector could be of grat use for traffic planners.

It was further shown in Chapter 3 that for the planning and operation of intersections, more specifically the allocation of green time, the mean of measured saturated flows is the most reasonable dimensioning value. This stands in contrast to the definition of saturation flow given by the current Swiss standards. As the relevant standard (VSS, 1997) is aimed at traffic planners for the precise reason of dimensioning intersections, an adjustment of the respective entry from 'highest discharge flow during green phases' to 'expected average discharge flow during saturated green phases' seems appropriate.

Comparison of Projected and Measured Saturation Flows

The comparison of 14 projection methods to the empirically determined saturation flows in Chapter 4 is the core of this research. It was shown that the three base methods, as presented by Swiss, US and German standards (VSS, 1997; TRB, 2010; FGSV, 2015), are all fairly accurate. No base method could be shown to consistently provide better (or worse) estimates than the other two. For these reasons, all base methods are considered valid. However, further research could provide information on which factors the standards overestimate and which they underestimate, thereby enabling improvements on the methods.

The analyzed modifications of the base methods to account for presumed synergies between the road's gradient and heavy vehicle presence generally decreased average accuracy of the estimates. An exception to that is a modification based on Skabardonis et al. (2014), which slightly increased average accuracy when applied to the Swiss base method. Additional research and possibly improvements on the method are necessary, however, before its inclusion in commonly used methods is justifiable.

The projection methods based on a supposed average value, as presented by VSS (2008), were shown to substantially and consistently overestimate saturation flow, regardless of the consideration of an adjustment for heavy vehicle presence. Therefore, the usage of these methods on right-turning lanes is strongly advised against and an according adjustment or removal of the relevant entries from the Swiss standards is recommended.

Further Results & Discussion

The flows measured in saturated intervals, as determined by the sample-size-enhancing methodology, were shown to follow normal distribution. This may increase the practicability of methods for calculating extreme values (e.g. minimal green time for a throughput of x vehicles) by using the mean and a desired multiple of the standard deviation of the measured saturated flows.

The commonly used adjustment for effective green time by adding one second to each green phase was shown to effectively remove dependency of saturation flow on green phase length.

The simplistic percentile-method, used to empirically estimate saturation flow, was shown to depend on the relevant lane's saturation pattern. Its accuracy is thus highly variable. Usage of this method is advised against for more than first estimates is advised against, due to its low reliability.

Finally, the flow rate in saturated intervals was shown to change over the course of the day. Specifically, average saturation flow is higher during evening commuting hours than during the rest of the day. These changes are expected to be due to differences in the vehicle mix (less heavy vehicles present in the evening) as well as driver-related factors like driver skill, attentiveness and aggressiveness.

Additional Research Topics

A key shortcoming of this research is its limitation to right-turning lanes. An analysis of the prediction methods' accuracies for other lane types (straight, left-turning, mixed) is likely to yield valuable additional information. This might be done in connection with an analysis of the accuracy of common simulation methods, which are increasingly used in traffic planning.

7 References

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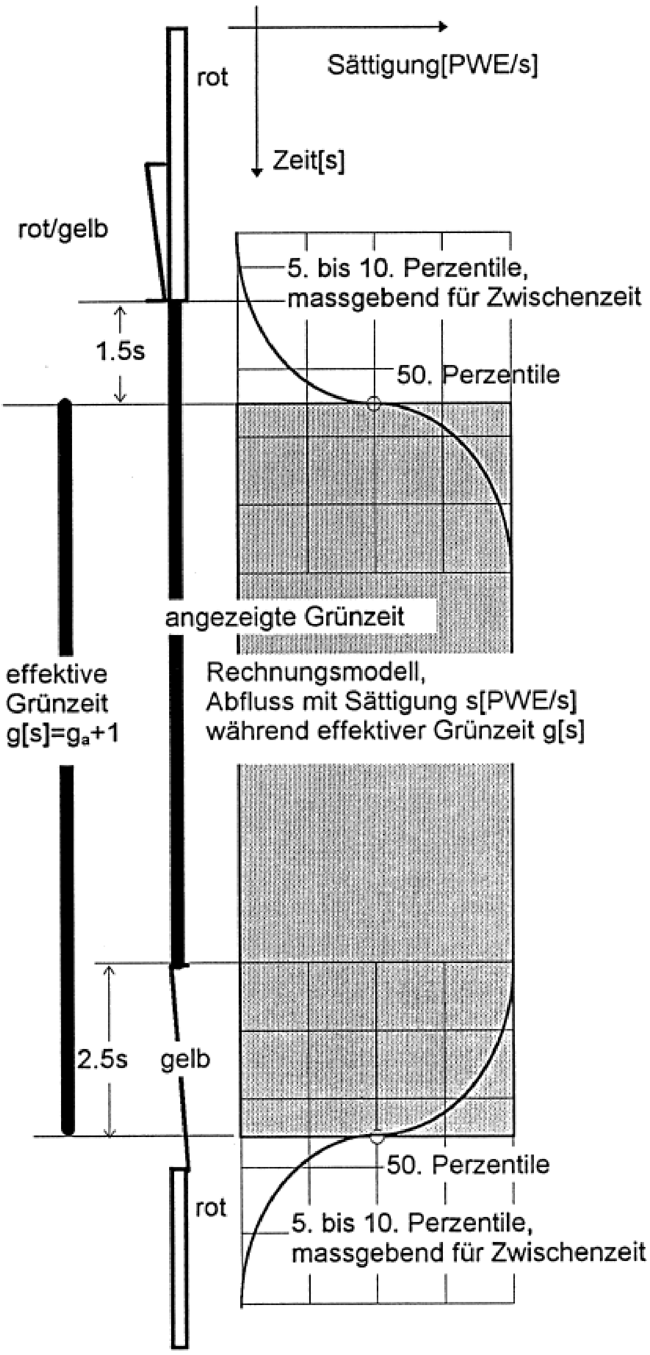
A Appendix – Literature Review

Effective Green Time

Figure 14: Calculation of the effective green time adjustment of one second based on the distribution of departures over a green phase.



Source: Pitzinger (1996)

Adjustment Factors in Reviewed Standards

Table 15: List of the identified influential variables and whether the reviewed standards mention them or even offer adjustment values. Influences without mention in any of the three standards are taken from the following sources: McCoy and Heimann (1990), European Parliament (2008), European Parliament (2008) and Brumfield and Pulugurtha (2012).

Category	Influential Variable	VSS	HCM	HBS
Infrastructure	Lane Width	Value	Value	Value
	Gradient	Value	Value	Value
	Radius of Turn	Value	Value	Value
	Upstream Storage Space	Value	Mention	Value
	On-street Parking	Value	Value	-
	Number of Approach Lanes	-	Mention	Mention
	Proximity of Bus Stop	Value	Value	-
	Proximity of Side Road	-	-	-
	Lane Position Relative to Curb	-	-	Mention
Traffic	Share of Heavy Vehicles	Value	Value	Value
	Share of Motorcycles	Value	-	-
	Share of Bicycles	Value	-	Mention
	Prevalent Trip Purpose	-	Mention	-
	Driver Attentiveness	-	-	-
	Cultural Differences	-	-	-
Operation	Allowed Movements on Lane	Value	Value	-
	Duration of Green Phase	Value	-	Value
	Transit Priority	Value	-	-
	Pedestrian Blockage	Value	Value	Value
	Concurrent Traffic Streams	Value	Value	Value
	Right Turns on Red	-	Mention	-
Environment	Weather	-	-	Mention
	Readability	-	-	-
	Distractions	-	-	-
	Area Type	-	Value	Mention

Source: VSS (1997), FGSV (2009), TRB (2010) and FGSV (2015) in order of publication

B Appendix – Methodology

B.1 Intersections

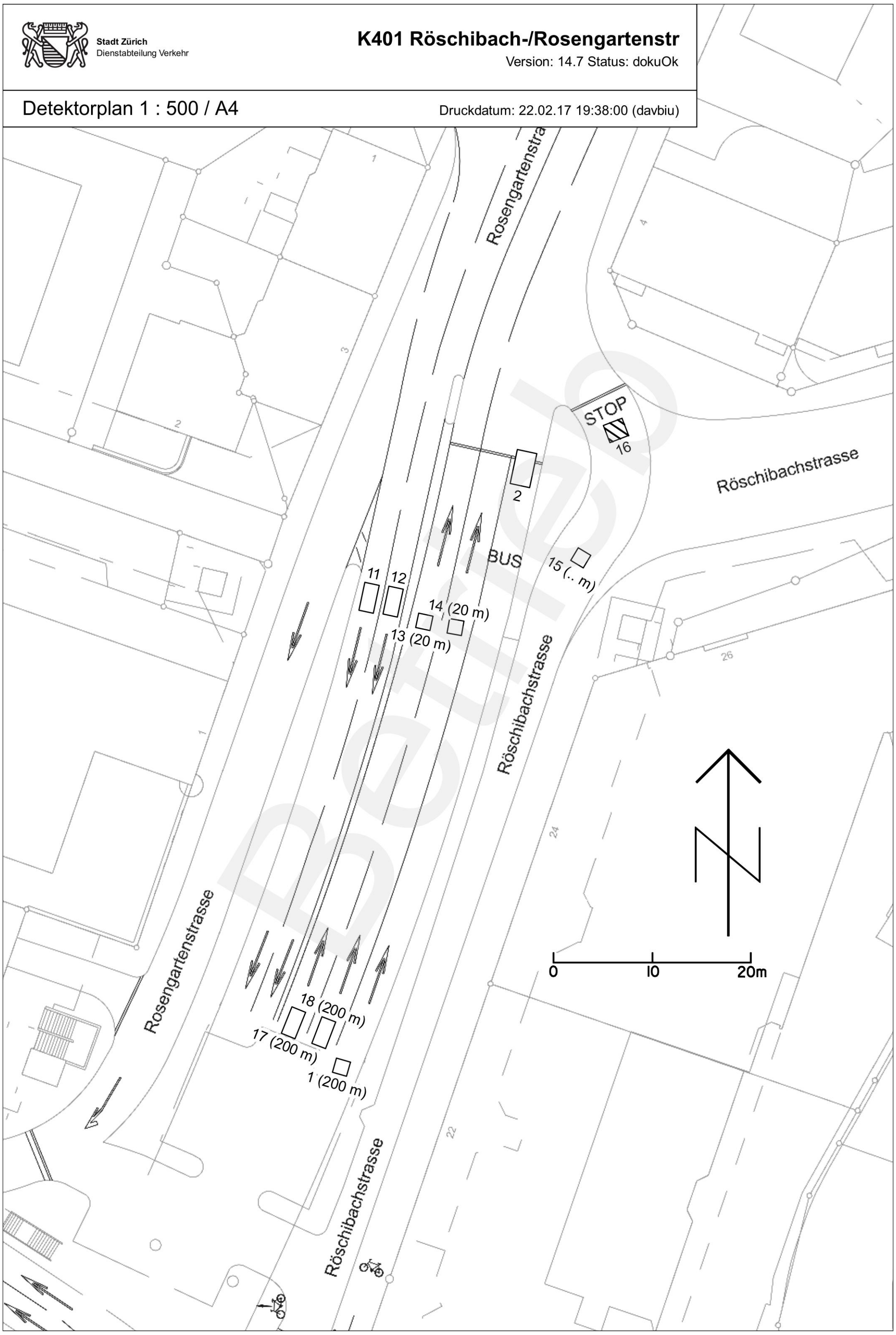
B.1.1 Intersection Röschibachstrasse-Rosengartenstrasse

Table 16: Overview over intersection ’Röschibachstrasse-Rosengartenstrasse’

Abbreviation	RR		
From	Röschibachstrasse		
Onto	Rosengartenstrasse		
Traffic Light	Actuated with bus priority		
Infrastructure	Minimum lane width	3.8	m
	Minimum Radius of Turn	15.0	m
	Grade (average)	5.8	%
	Grade (maximum)	8.7	%
Side Road	Immediately before stop line		
Pedestrians	None, overpass available		
Cyclists	Extremely rare, own bicycle path available		
Public Transport	On intersection?		Yes
	On upstream lane?		No
	On downstream lane?		Yes
	In rivaling traffic stream?		Yes
	Conflict during green?		No
Detector ’D16’	Average distance from stop line	0.5	m, upstream
	Inaccuracy	± 3.7	%
	Length	0.5	m
Detector ’D15’	Average distance from stop line	20	m, upstream
	Inaccuracy	± 4.3	%
	Length	1.5	m

Source: Own measurements and analysis

Figure 15: Intersection RR - Röschibachstrasse-Rosengartenstrasse, technical plan



Source: DAV Zürich

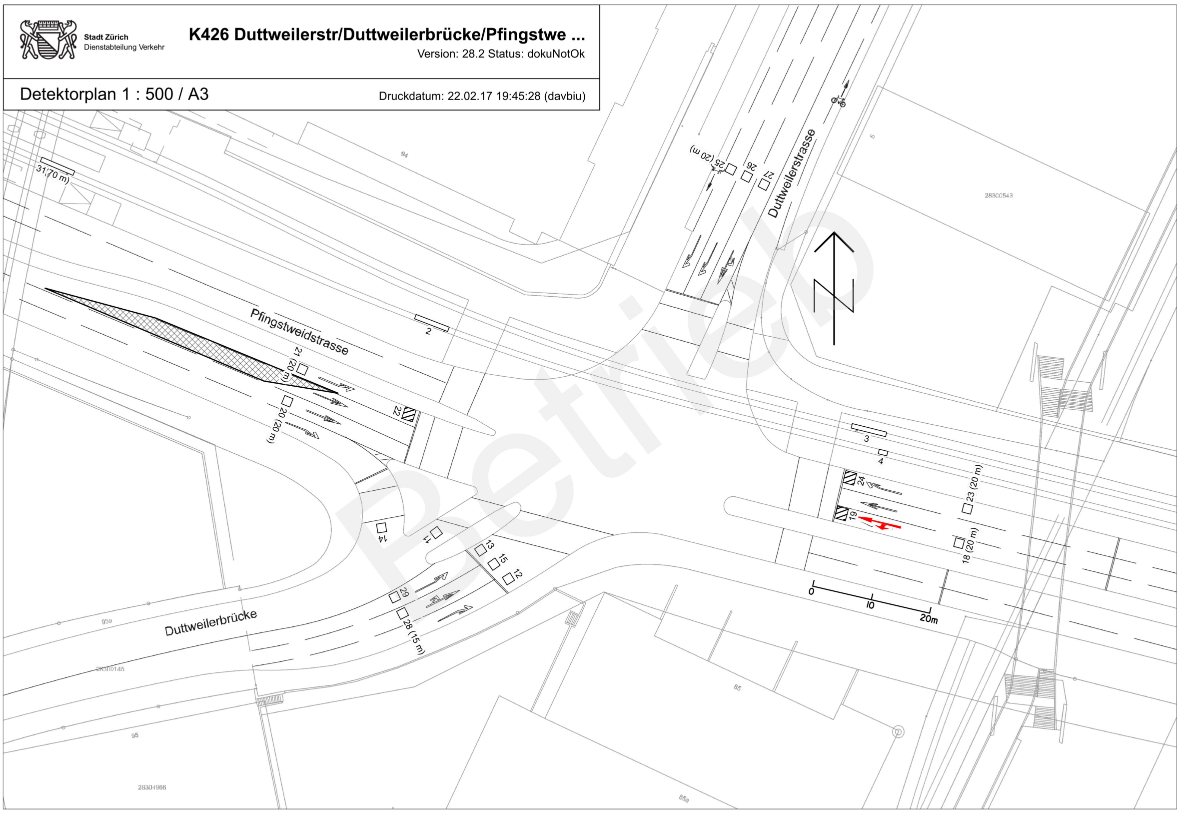
B.1.2 Intersection Pfingstweidstrasse - Duttweilerbrücke

Table 17: Overview over Intersection 'Pfingstweidstrasse - Duttweilerbrücke'

Abbreviation	PD		
From	Pfingstweidstrasse		
Onto	Duttweilerbrücke		
Traffic Light	Fixed in coordinated axis		
Infrastructure	Minimum lane width	3.2	m
	Minimum Radius of Turn	12.0	m
	Grade (average)	3.0	%
	Grade (maximum)	7.0	%
Pedestrians	Not simultaneously, no conflicts possible		
Cyclists	Extremely rare, own bicycle path available		
Public Transport	On intersection?		Yes
	On upstream lane?		No
	On downstream lane?		No
	In rivaling traffic stream?		No
	Conflict during green?		No
Detector	Average distance from stop line	11	m, upstream
	Inaccuracy	± 1.6	%
	Length	1.5	m
	Designation	D14	

Source: DAV (2017) and own measurements

Figure 16: Intersection PD - Pfingstweidstrasse-Duttweilerbrücke, technical plan



Source: DAV Zürich

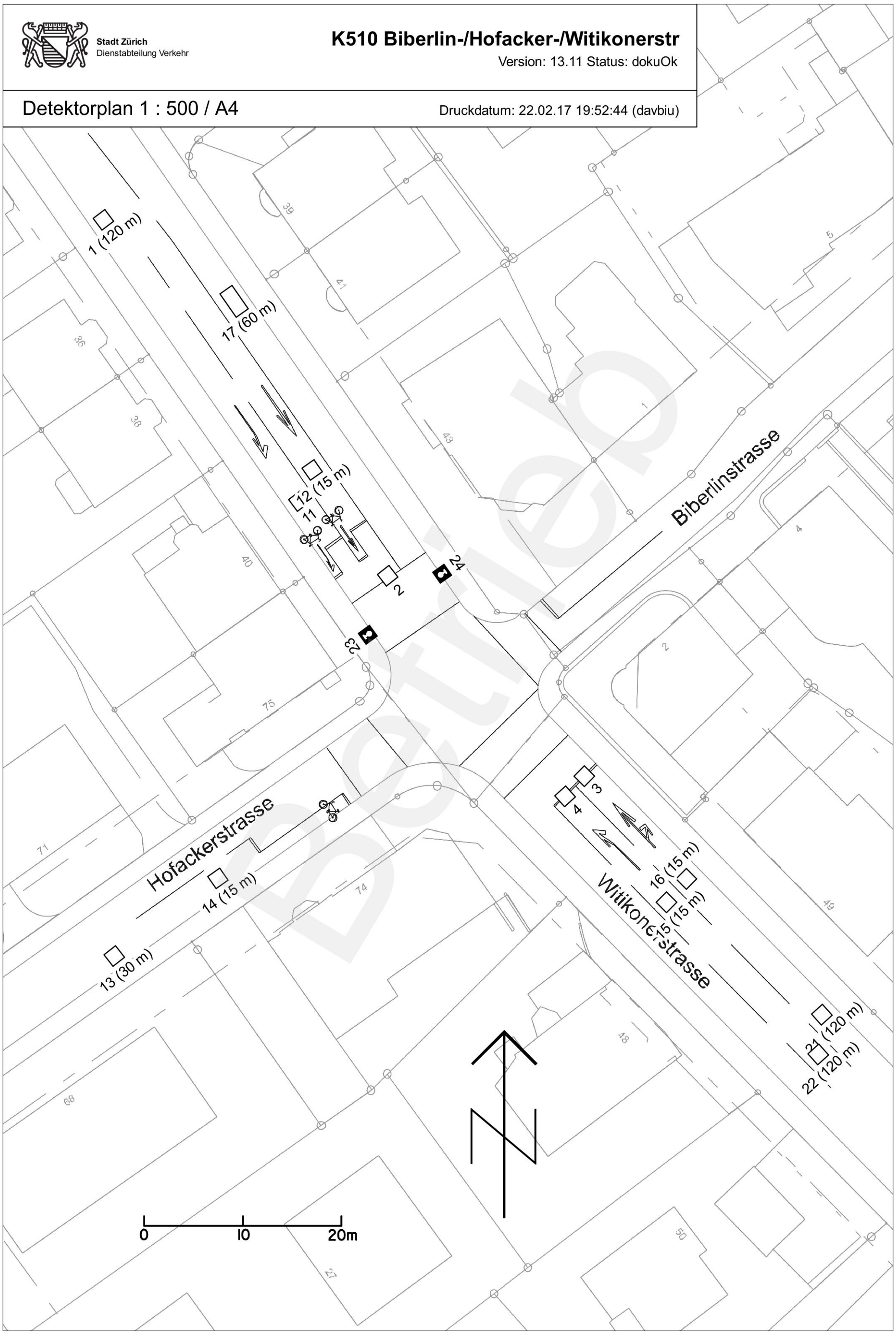
B.1.3 Intersection Witikonerstrasse - Hofackerstrasse

Table 18: Overview over intersection *Witikonerstrasse - Hofackerstrasse*

Abbreviation	WH		
From	Witikonerstrasse		
Onto	Hofackerstrasse		
Traffic Light	Actuated with bus priority		
Infrastructure	Minimum lane width	2.8	m
	Minimum Radius of Turn	6.5	m
	Grade (average)	-2.0	%
	Grade (minimum)	-12.0	%
Pedestrians	Simultaneous green phase, conflicts possible		
	Pedestrian lead time	2	s
Cyclists	Some, often bypass detector on side		
Public Transport	On intersection?		Yes
	On upstream lane?		No
	On downstream lane?		Yes
	In rivaling traffic stream?		Yes
	Conflict during green?		No
Detector	Average distance to stop line	5.7	m, upstream
	Inaccuracy	± 2.9	%
	Length	1.5	m
	Designation	D11	

Source: DAV (2017) and own measurements

Figure 17: Intersection WH - Witikonerstrasse-Hofackerstrasse, technical plan



Source: DAV Zürich

B.2 Projection Methods

B.2.1 Heavy Vehicle Impact on Inclined Intersections

Table 19: PCE values for vehicle types on inclined unsignalized intersections according to the Swiss standard SN 640 022

used in	Vehicle Type	Grade [%]	-4	-2	0	+2	+4
none	Passenger car	[PCE/veh]	0.8	0.9	1.0	1.2	1.4
Modification a1	Trucks without trailers	[PCE/veh]	1.0	1.2	1.5	2.0	3.0
Modification a2	Trucks with trailers	[PCE/veh]	1.2	1.5	2.0	3.0	6.0

Source: VSS (1999)

Table 20: PCE-values used in modification b as presented by Skabardonis et al. (2014) for heavy vehicles at signalized intersections as a function of the approach lane's inclination and the proportion of heavy vehicles in the traffic stream

Grade [%]	Truck Proportion [%]	5	10	20	30	≥ 40
< 0	[PCE/veh]	1.5	1.5	2.0	2.0	2.0
≤ 2	[PCE/veh]	2.0	2.0	2.0	2.0	2.0
4	[PCE/veh]	2.5	2.5	2.5	2.5	2.5
6	[PCE/veh]	3.0	3.0	3.0	3.0	3.0
8	[PCE/veh]	4.0	4.0	4.0	3.5	3.5
10	[PCE/veh]	8.0	7.0	6.0	6.0	6.0

Source: Skabardonis et al. (2014)

B.2.2 Passenger Car Impact on Inclined Intersections

The values in Table 21 show that the adjustment of saturation flow for gradient using passenger car equivalents[47], differs heavily from the other presented factors. Relatively modest inclinations of plus or minus four degrees result in a saturation flow change of -29 % or +25 % respectively. As this seems to be an overestimation of effects, this approach is not further pursued in this analysis.

[47] as proposed by the Swiss standard *SN 640 022* (VSS, 1999)

Table 21: Effect of gradient (inclination in direction of travel) on saturation flow of vehicle streams with 100% passenger cars as presented by various standards.

Standard	Calculation	Grade [%]	-6	-4	-2	0	+2	+4	+6
SN 640 835	$f_g = 1 - \frac{g}{50}$		-	-	-	1.00	0.96	0.92	0.88
HCM 2010	$f_g = 1 - \frac{g}{200}$		1.03	1.02	1.01	1.00	0.99	0.98	0.97
HBS 2015	$f_g = \frac{1}{1+g \times 0.03}$		-	1.14	1.06	1.00	0.94	0.89	-
SN 640 022	PCE adjustment (see Table 19)		-	1.25	1.11	1.00	0.83	0.71	-

Source: own representation based on VSS (1997), VSS (1999), TRB (2010) and FGSV (2015) in order of publication

B.2.3 Adjustment for Side Road at Intersection RR

The adjustment factor for saturation flow due to side road presence is calculated according to Equation 8 as proposed by McCoy and Heimann (1990). The research of McCoy and Heimann is limited to only two test sites. The reliability of their findings is therefore contestable. Furthermore, both test sites involve driveways located at greater distance from the respective stop line than the side road at intersection RR. The minimum corner clearance considered by McCoy and Heimann amounts to roughly 30 m. The corner clearance at intersection RR is below 10 m. Finally, there might be a difference between driveway behaviour and side road behaviour. Thus, applicability of the McCoy and Heimann findings on intersection RR is questionable.

$$f_{\text{side road}} = \frac{h}{h + p_{\text{enter}} \times t_{\text{enter}} + p_{\text{leave}} + t_{\text{leave}}} = 0.96 \tag{8}$$

where

h	= 2.0	s/veh	assumed headway without interference from side road traffic
p_{enter}	= 3.4	%	share of vehicles that turn onto the side road (own measurements)
t_{enter}	= 1.9	s/veh	increase in departure headway caused by vehicles turning onto the side road (value provided by McCoy and Heimann (1990) for minimum corner clearance)
p_{leave}	= 2.2	%	share of vehicles that leave the side road (own measurements)
t_{leave}	= 1.1	s/veh	increase in departure headway caused by vehicles leaving the side road (value provided by McCoy and Heimann (1990) for minimum corner clearance)

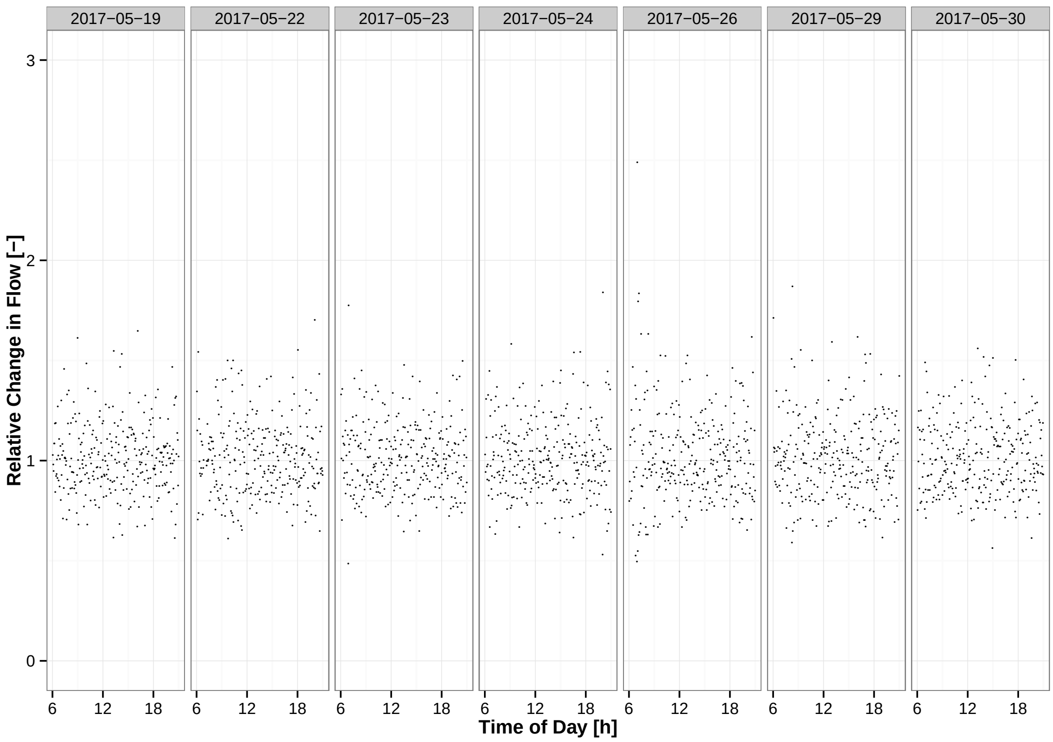
B.3 Real Saturation Flows - Determining Saturation

B.3.1 Unsuccessful Attempt - Variability of Flow

The discharge flow from an intersection is easier to predict for certain times of day than for others. During rush hours, flows are typically high, due to high traffic demand. At night time they are usually low, due to low demand. Both effects can reasonably be expected for most intersections and most days[48]. For off-peak hours, however, discharge flow rates may be hard to predict, due to ever-changing demand.

Based on these considerations, the variability of flow from one 3-minute-interval to the next is analyzed, in the hope of finding a link between the variability and saturation. High flow variability is expected for very high and very low demand. Thus, intervals with relatively high flows and low variability of flow are expected to be saturated.

Figure 18: Exemplary plot on the variability of flow at intersection RR.



Source: own plot

The relative change from one 3-minute-interval to the next was analyzed but the above behaviour could not be observed. While there were intervals with low variability of flow, saturation of

[48]Excluding effects of special events like big football games, etc.

these intervals could not be proven. Figure 18 shows the relative change in flow from one 3-minute-interval to the next at intersection RR.

B.3.2 Video Analysis

Table 22: Date, time and duration of the video analysis on the three intersections. Also shows number of total and saturated 3-minute-intervals ovserved.

Intersection	Date	Day	Start Time	End Time	#intervals	#saturated intervals
RR	10.05.17	Wed	10:38	11:45	22	16
	10.05.17	Wed	17:29	18:31	21	19
	17.05.17	Wed	15:30	16:25	17	17
				total	60	52
PD	17.05.17	Wed	13:12	15:06	38	23
	22.05.17	Mon	17:09	18:00	17	1
	23.05.17	Tue	10:10	11:21	23	5
				total	78	29
WH	11.05.17	Thu	16:16	18:00	34	0
	24.05.17	Wed	16:20	17:06	15	0
				total	49	0

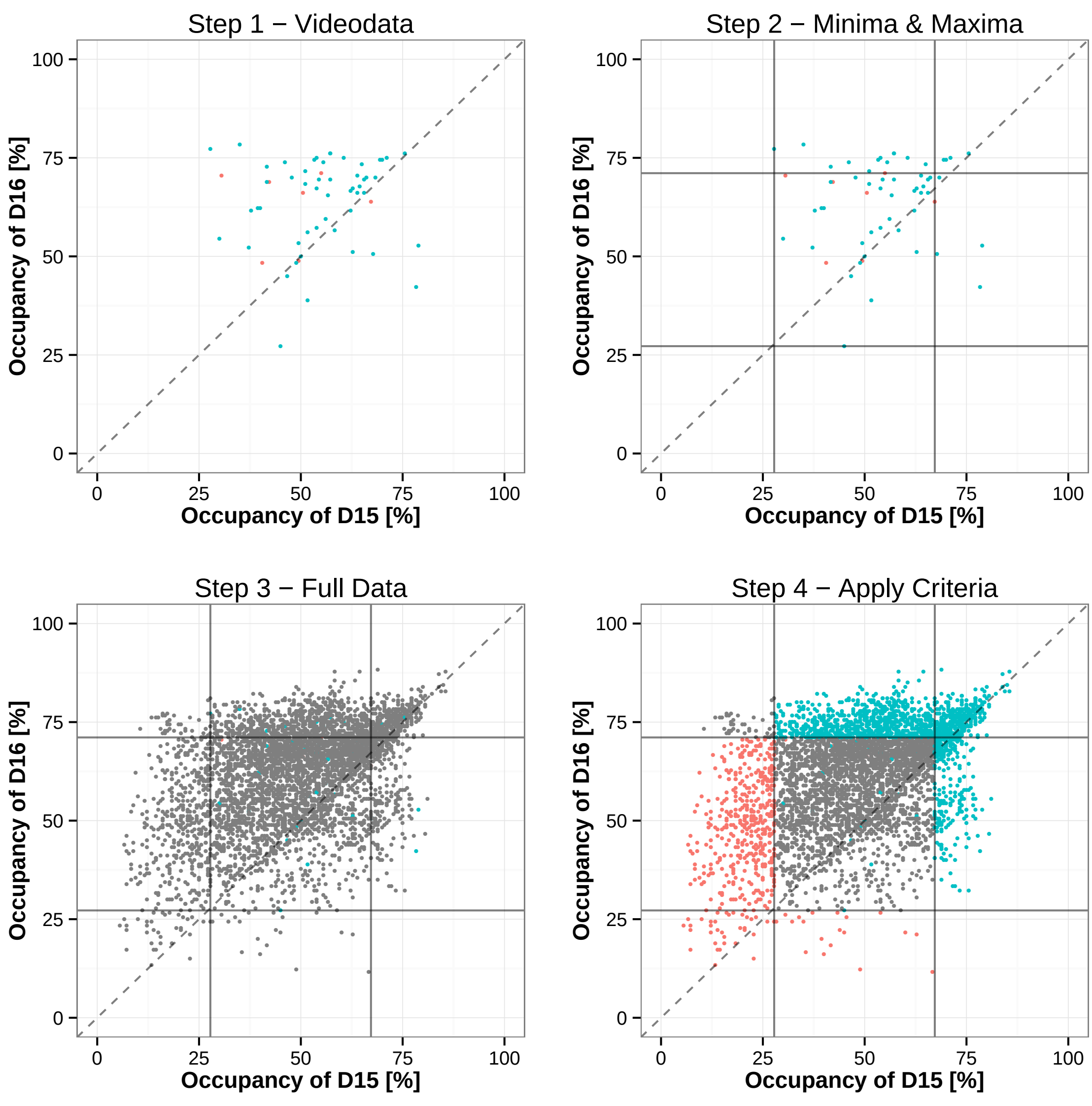
Source: own measurements

B.3.3 Increasing Sample Size of Saturated Flows at Intersection RR

Figure 19 shows how additional saturated intervals can be determined at intersections with two upstream detectors. The methodology uses the detector occupancy values of saturated and unsaturated 3-minute-intervals from a sample (in this research acquired in field analysis, step 1). The minimum occupancies of the sample's saturated intervals and the maximum occupancies of the sample's unsaturated intervals are determined for both detectors (step 2). Then, all intervals from the available data (step 3) are classified according to requirements a and b (step 4).

An interval is classified as saturated if (a) its occupancy value on one detector is higher than the highest observed occupancy value at this detector during intervals that were observed to be unsaturated (during the video analysis) while (b) its occupancy value at the other detector is not below the lowest observed occupancy value at that detector during intervals that were observed to be saturated (during the video analysis). The reversed criteria can be applied to identify additional unsaturated intervals.

Figure 19: Method using detector occupancy to increase sample size for intersection lanes with two detectors in the upstream.
Blue: saturated 3-min-intervals
Red: unsaturated 3-min-intervals
Gray: unspecified 3-min-intervals

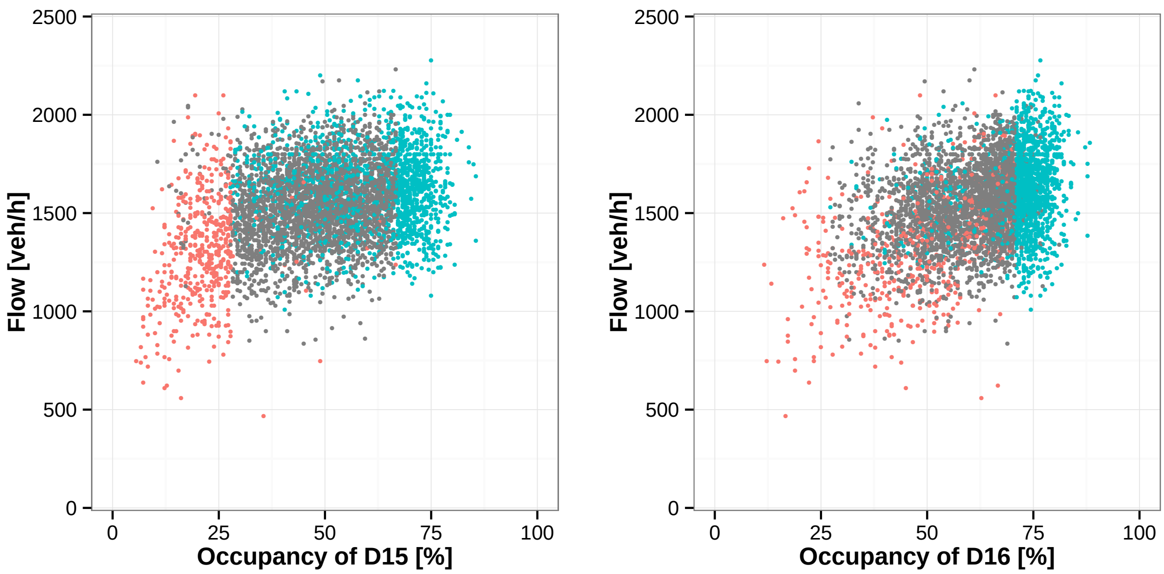


Source: own plots

As Figure 19 shows, a lot of intervals remain unspecified. The proposed method is therefore not a suitable tool if the saturation of certain individual intervals is of interest. In spite of this, it is useful for determining a large number of saturated intervals that can then be used to derive saturation flow. The method's rate of false positives (falsely determined saturation) is not examined in detail[49] but is expected to be relatively low due to the following reasons:

- The used criteria a and b (see above) are estimated to be relatively restrictive, especially considering that many saturated intervals of the reference sample have lower occupancy values than those determined by the method (see Figure 19).
- The mean saturation flow based on the method (mean = 1622 veh/h, sample size = 1577) deviates less than 4% from the mean saturation flow observed in the video analysis (mean = 1679 veh/h, sample size = 52). The result is thus in a realistic range.

Figure 20: Relation of flow and occupancy on the two detectors at intersection RR with saturation determined according to the presented methodology.
Blue: saturated 3-min-intervals
Red: unsaturated 3-min-intervals



Source: own plots

The application of the presented methodology is only recommended if no occurences in the intersection's downstream blocked discharge from the intersections. At intersection RR, where this method was applied, the next signalized intersection is more than 500 m downstream. Therefore, spill-back is considered unlikely. This is supported by Figure 20, which shows the interval's discharge flows in relation to the occupancy at both detectors. The plots show that there are no saturated intervals where flow rate is extremely low while occupancy is very high. However, in the method's current state, the existence of false positives cannot be ruled out completely.

[49] A detailed examination of the method's error rate would require a number of additional measurements and ultimately goes beyond the scope of this thesis.

Further research on the method and its applicability could be of great value. The error rate might be further reduced by introducing a criteria which determines intervals with low flow rate and high occupation at *both* detectors to be influenced by spill-back. However, methods that use flow values to rule out saturation should be used restrictively as shown at the example of percentile methods in 5.5.

C Appendix – Comparison

Projection Methods Results

Modifications a2 and b yield equal values for the adjusted saturation flow because for the given grade and heavy vehicle proportions, the same PCE-value for heavy vehicles is determined in the two methods.

Table 23: Ideal saturation flows, individual adjustment factors and resulting adjusted saturation flows for the 14 considered methods (see Section 3.2). All flow values are rounded to the nearest whole multiple of 10.

	avg_{VSS}		VSS				HCM				HBS			
Intersection **RR**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
f_w			1.03	1.03	1.03	1.03	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
f_r			0.91	0.91	0.91	0.91	0.85	0.85	0.85	0.85	0.93	0.93	0.93	0.93
f_g			0.88				0.97				0.85			
f_{HV} (for p_{HV} = 2.6%)		0.97	0.97				0.97				0.98			
$f_{\text{grade \& heavy vehicles}}$				0.85	0.79	0.84		0.93	0.86	0.91		0.82	0.77	0.81
Total Adjustment [-]		0.97	0.80	0.79	0.74	0.78	0.80	0.78	0.73	0.77	0.77	0.76	0.72	0.75
Adjusted Saturation Flow [veh/h]	1800	1760	1610	1580	1490	1570	1520	1490	1390	1470	1550	1520	1430	1510

	avg_{VSS}		VSS				HCM				HBS			
Intersection **PD**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
f_w			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
f_r			0.89	0.89	0.89	0.89	0.85	0.85	0.85	0.85	0.89	0.89	0.89	0.89
f_g			0.94				0.99				0.92			
f_{HV} (for p_{HV} = 9.0%)		0.92	0.92				0.92				0.93			
$f_{\text{grade \& heavy vehicles}}$				0.87	0.81	0.81		0.91	0.84	0.84		0.85	0.79	0.79
Total Adjustment [-]		0.92	0.77	0.79	0.72	0.72	0.77	0.77	0.71	0.71	0.76	0.76	0.71	0.71
Adjusted Saturation Flow [veh/h]	1800	1650	1530	1550	1430	1430	1460	1460	1350	1350	1520	1530	1420	1420

	avg_{VSS}		VSS				HCM				HBS			
Intersection **WH**	0	adj	base	a1	a2	b	base	a1	a2	b	base	a1	a2	b
Ideal Saturation Flow [PCE/h]			2000	2000	2000	2000	1900	1900	1900	1900	2000	2000	2000	2000
Average Saturation Flow [PCE/h]	1800	1800												
f_w			0.98	0.98	0.98	0.98	0.96	0.96	0.96	0.96	0.93	0.93	0.93	0.93
f_r			0.81	0.81	0.81	0.81	0.85	0.85	0.85	0.85	0.83	0.83	0.83	0.83
f_g			1.04				1.01				1.06			
f_{HV} (for p_{HV} = 0.7%)		0.99	0.99				0.99				0.99			
$f_{\text{grade \& heavy vehicles}}$				1.04	1.04	1.03		1.01	1.01	1.00		1.06	1.06	1.05
Total Adjustment [-]		0.99	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.81	0.82	0.82	0.82	0.81
Adjusted Saturation Flow [veh/h]	1800	1780	1640	1650	1650	1630	1550	1560	1560	1550	1640	1640	1640	1630

Source: own representation based on (VSS, 1997), (VSS, 1999), (VSS, 2008), (TRB, 2010), (Skabardonis et al., 2014) and (FGSV, 2015) in order of publication