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Bundesamt für Strassen
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Amélioration du modèle de comportement individuel du conducteur pour évaluer la sécurité d'un flux de trafic par simulation

Verbesserung des individuellen Fahrverhaltens-Modells zur Bewertung der Sicherheit eines Verkehrsflusses

Improvement of Individual Driver Behavior Model for the Safety Assessment of Traffic Flow Simulation

École Polytechnique Fédérale de Lausanne (EPFL)
Laboratoire des voies de circulation (LAVOC)
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Mandat de recherche ASTRA 2004/015 sur demande de l'Office fédéral des routes (OFROU)

Décembre 2012 **1406**

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Table des matières

Résumé

Cette étude, fait partie du projet COST 352, propose un cadre pour l'élaboration d'un système proactif visant à réduire le risque d'accident sur l'autoroute à l'aide des indicateurs de risque. Bien que les autoroutes soient de type de route en toute sécurité par la conception, un accident qui se produit serait vraiment grave. Réduire le risque d'accident sur l'autoroute serait de prévenir les accidents ou de diminuer leur gravité. À cet effet, les indicateurs de risque sont testés et validés avec des données réelles, y compris les données de trafic, des données météorologiques, et une base de données d'accidents d'un site d'étude en Suisse. Une directive sur la façon de définir les critères de risque est proposée et appliqué dans un modèle de simulation qui est la plate-forme de mise en œuvre d'un système proactif de prévention des risques. Le modèle est calibré afin de mieux représenter une section de route du site d'étude. Un exemple de mise en œuvre des mesures de prévention est mis en place et ses résultats sont évalués.

Mots-clés: indicateur de risque, critère de risque, mesure préventive, calibration des modèles de simulation, prévention de risques d'accidents.

Zusammenfassung

Diese Studie – Teil COST 352 - schlägt einen Rahmen für die Entwicklung eines pro-aktiven Systems zur Verringerung der Unfallrisiken auf Autobahnen vor, welches auf spezifischen Risikoindikatoren basiert. Obwohl das Autobahnnetz zu den sicheren Strassennetzen gehört, können allfällige Unfälle sehr schwerwiegend sein. Das Verringern des Unfallrisikos auf solchen Strassen würde es erlauben, solche Vorfälle zu vermeiden oder zumindest die Unfallschwere zu vermindern. Zu diesem Zweck wurden Risikoindikatoren getestet und mit realen Daten validiert. Diese Daten umfassten Verkehrs-, Wetter-, sowie Unfalldaten einer Versuchsstrecke in der Schweiz. Eine Richtlinie zur Definition der Risikokriterien wurde vorgeschlagen und in einem Simulationsmodell angewandt, welches einer Plattform für die Umsetzung eines pro-aktiven Systems zur Risikoverminderung entspricht. Das Model wurde kalibriert, um einen Abschnitt der Versuchsstrecke bestmöglich darzustellen. Ein Beispiel für die Umsetzung vorbeugender Massnahmen wurde ebenfalls umgesetzt, und dessen Ergebnisse ausgewertet.

Kennwörter: Risikoindikator, Risikokriterium, vorbeugende Massnahme, Kalibrieren von Simulationsmodellen, Unfallrisikovermeidung.

Summary

This study – part of the COST352 action - proposes a framework for developing a pro-active system aiming at reducing motorway crash risk based on risk indicators. Although motorways are of safe road type by design, a crash if happens would be really severe. Reducing motorway crash risk would prevent crashes or diminish their severity. To that purpose, risk indicators are tested and validated with real data including traffic data, meteorological data, and crash databases from a study site in Switzerland. A guideline about how to define the risk criteria is proposed and applied in a simulation model which is the platform for implementing a pro-active crash risk prevention system. The model is calibrated to best represent a road section at the study site. An implementation example of preventive measures is introduced and its outcome is evaluated.

Keywords: risk indicators, risk criteria, preventive measures, model calibration, crash risk prevention.

1 Introduction

1.1 COST 352 Project: Improvement of Individual Driver Behavior Model for the Safety Assessment of Traffic Flow Simulation

The main objective of the Action COST352 was to create a scientific base for road traffic and vehicle equipment legislation, safety evaluation methodology and rules for drivers' education and training for the appropriate use of In-Vehicle Information Systems (IVIS) in order to enhance road safety.

Secondary objectives are:

- establish the effects of increasing amounts of information available to drivers, through IVIS ;
- demonstrate how they contribute to driver distraction in road environments where outside information is normally provided in order to create a scientific base for :
    - safety evaluation methodology;
    - rules for drivers education and training;
    - road traffic and vehicle equipment legislation in the relevant area

The knowledge gained will inform policy makers and industry about how to respond to the increasing range and availability of IVIS equipment.

Technical development and commercial realization of IVIS are moving rapidly and Road Safety policy with respect to these systems therefore needs to be dynamic and flexible to encourage, restrict or influence developments. IVIS can be used by private and commercial drivers in a range of vehicles for different applications and under different traffic conditions. The impact on road safety will depend on how the systems are implemented, the users are informed and the devices used.

Existing policy instruments, such as Type Approval, national traffic regulations and driver education, may need to be adapted or applied in new situations arising from the availability of IVIS. Key issues will be identified and thus provide a robust scientific basis to inform policy actions in this developing area.

The participating countries in this Action are:

- Austria
- Czech Republic
- France
- Germany
- Italy
- Lithuania
- Netherlands
- Norway
- Poland
- Portugal
- Switzerland

The study reported in this report is the second part of the Work Package II.

The Working packages listed in Fig. 1.1 have been identified.

Fig. 1.1 Work Packages of COST Action 352

Work package	Title	Task	Contents	Results deliverables
I	Inventory of existing knowledge	I.1	In-vehicle information and guidance devices	Inventory
		I.2	Electronic messaging	
		I.3	Mobile telephone systems	
		I.4	Entertainment	
		I.5	Human information processing	
		I.6	Web site	
II	Round tables	II.1	First round table (preparation/event day)	Preparation of driver behavior studies
		II.2	Second round table (preparation/event day)	Preparation of further research requirements and dissemination
III	Driver behavior studies preparative	III.1	Common items to be addressed	Methodology
		III.2	Qualitative links between behavior and accident risks	
		III.3	Simulator	
		III.4	Real traffic conditions	
IV	Driver behavior studies by simulation	IV	In-vehicle systems / Driver categories / Road types	Research results
V	Driver behavior studies in real traffic conditions	V	In-vehicle systems / Driver categories / Road types	Research results

1.2 Objective of WP IV, part 2

Switzerland is one of the leading countries in road traffic safety. According to [BPA (Ed.), (2006)], in recent years, the number of deaths and heavy injuries by road traffic crashes is continuously decreasing. This positive trend is obtained thanks to a series of preventive measures such as speed enforcement, new speed limit regulations, improvement of pavement, etc. However, the total number of road traffic crashes is still very high with about 27 000 deaths and injuries in 2005, of which the number of motorway traffic deaths is about 400 and this number does not change much in comparison to previous years.

The work reported here is a step forward which aims at two objectives:

- Develop risk indicators to identify traffic crash risk on Swiss motorways.
- Implement preventive measures to reduce crash risk in a simulation platform.

1.3 Structure of this report

Chapter 2 discusses the study site and the data used in this project. In chapter 3, risk indicators are introduced and their sensitivity is tested against data observed from the study site. Results introduced in chapter 3 are applied in the work presented in chapter 4 about simulation model. Preventive measures are discussed in chapter 4. The conclusion the whole study is presented in chapter 5.

2 Data and study site

2.1 Introduction

The Swiss motorway network has, by the end of the year of 2’006, a total length of 1’758.2 km and is spanned on an area of 41’290 km2, which makes it one of the highest motorway densities in the world. As in the central of Europe, the Swiss motorway network does not only promote the internal mobility but also facilitates the trans-border transports between Switzerland and its other neighbors or between its neighbor countries with the Swiss motorway network as a shortcut. The Swiss motorway network is still being enlarged as shown in Fig 2.1.

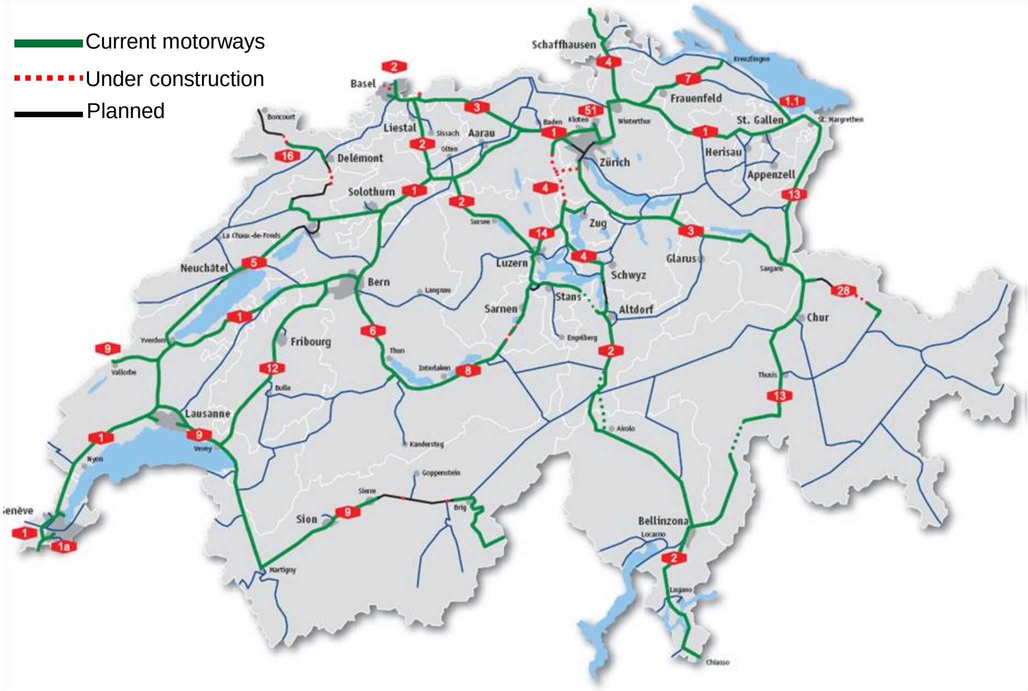


Fig 2.1 Swiss motorway networks

To operate the network, traffic detectors are installed on Swiss motorways. Most of the detectors can provide aggregate data while there are, till the end of 2’006, 67 traffic detectors that can provide vehicle information which is downloadable online. In the scope of this project, the individual data is essential. For this reason, the detectors that are potentially used in this project are the detectors that can provide downloadable data.

In this project, traffic data needs to be associated with crash data and weather data. This means that the positions of traffic detectors, weather stations, and crashes should be close enough to each other so that a creditable relationship between three types of data can be formed. This requirement will dominate the selection of the study site.

2.2 Traffic data

Switzerland is geographically divided into three main regions: the Plateau, Jura and Alps. While the Jura and Alps regions relate to two mountain chains in the north and the south of the country which are not populated, the Plateau is one of the most densely populated areas in Europe with very high economic concentration. Most of the national highways and inter-canton roads therefore, concentrate in the Plateau region.

In Switzerland, the network of Automatic traffic counts (ATC) has been installed since 1961. At the end of 2006, there were 259 permanent automatic traffic counting stations located in almost every motorways and main roads of the country, operated by FEDRO – the Swiss Federal Roads Office who is the federal authority responsible for national road infrastructure in the country. However, individual vehicle data from only 73 stations are downloadable online. Besides, the spacing between

ATCs is large, for example there is no ATCs that have a distance less than 5km.

Most of the downloadable ATCs are located on Swiss motorway network where the national motorways A1, A2 play the role of the backbones linking big cities from western part such as Geneva, Lausanne, Bern to the eastern cities of the country such as Zurich, St. Gallen or from the northern area such as Basel to the southern region like Bellinzona, Lugano. The other ATCs are positioned on the inter-canton roads having lower speed limits and accepting other types of vehicles such as scooter and bicycle.

Among the 73 ATCs that provide downloadable data, 3 ATCs were selected as potential study sites and are described in Fig 2.2.

Fig 2.2 Chosen ATCs (Switzerland)

Detector	Position Place/Canton/Road	Lanes	Notes
226	Crissier/Vaud/A1-A9	8	This is on the intersection between two national motorways allowing traffic in 4 directions: Geneva to Vevey and vice versa, Geneva to Yverdon-les-bain and vice versa. There are two lanes for each direction, resulting in 8 lanes in total.
149	Crissier/Vaud/A1	4	
116	Grandvaux/Vaud/A9	4	This motorway A9 links Lausanne to Vevey.

In Vaud canton, Switzerland, the motorway network contains 4 national motorways: A1, A5, A9, and A12. The road sections of these motorways in Vaud canton contain normally two lanes in each direction. Fig 2.3 shows the motorway network in Vaud canton.

Traffic data provided by 3 ATCs listed in Fig 2.2 are individual data. The 13 data fields in data files are described in Fig 2.4.

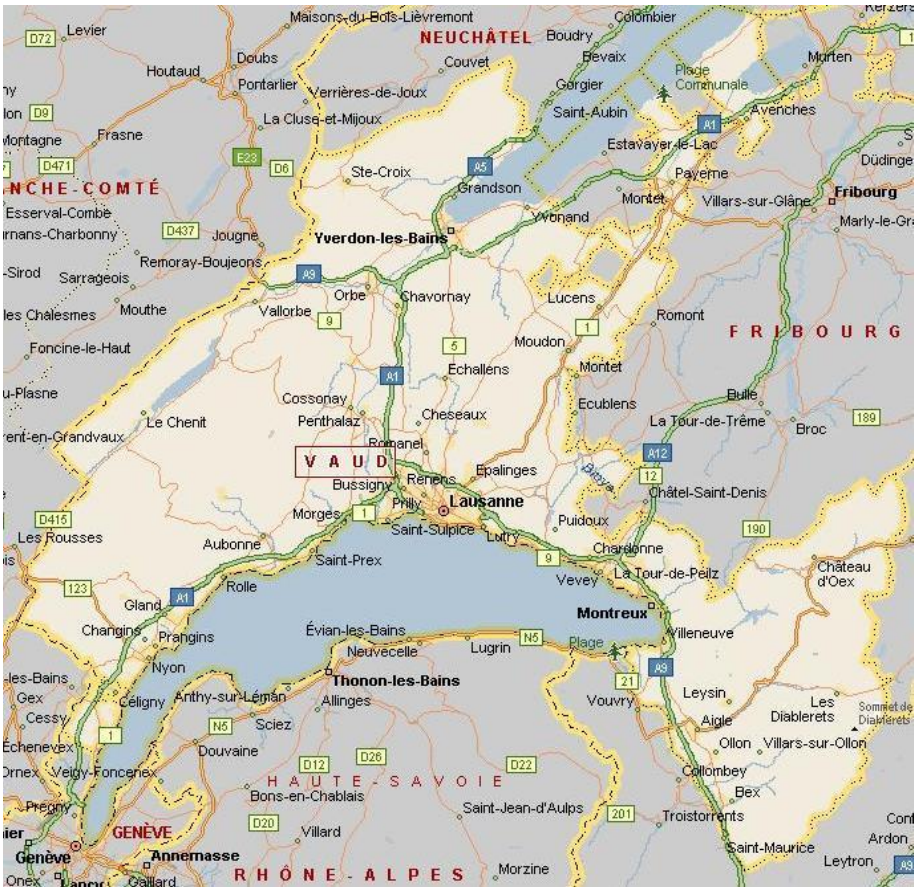


Fig 2.3 Motorway network in Vaud canton in Switzerland

Fig 2.4 Meaning of data fields in traffic data files

Data field	Meaning
Index	Automatic counting number
Time (DDMMYY, HHmm,Sec,1/100sec)	Moment when vehicle passes the ATC
Reserve-code	Reserved field
Lane	The lane where the vehicle is moving while passing the ATC
Direction	The direction of the lane
Time gap (Front-Front headway)	The duration between two moments where two consecutive vehicles start to pass the ATC. In second.
Headway (Rear-end)	The duration between the moment where the rear of the first vehicle passes the ATC and the moment where the front of the second vehicle starts to pass the ATC. In second.
Speed	Speed of vehicle when passing the ATC. In Km/h
Length	The length of the vehicle. In cm.
Vehicle classification	In Switzerland, vehicles are classified into 10 classes.

2.3 Meteorological data

In Switzerland, there are two sources of meteorological data that are accessible: one is from Boschung system and the other from the Federal Office of Meteorology and Climatology (MeteoSuisse). While meteorological data from MeteoSuisse have more general purposes, Boschung road weather stations are installed on road sections, specifically for monitoring the weather conditions on the Swiss road network. Therefore, we have chosen to use weather data from Boschung stations in this study. Boschung stations provide raw data collected every period of 6 minutes. Two important data fields in the data have been pre-processed namely, type of precipitation and intensity of precipitation. Only symbolic values for these fields are provided for those data fields. For the type of precipitation for example, three values are assigned: “R”, “S” and “-” for rain, snow, no-precipitation (or fine weather) conditions respectively. For any weather condition, the precipitation level is graded by "–", "F", "F1", "F2", "F3" which signify levels from nil to high intensity. Sample data lines extracted from a weather data file are shown in Fig 2.5.

Fig 2.5 Data format and example data provided by Boschung weather stations

Station number	Station name	Date & Time	Air Temp.	Ground Temp.	Relative Humid.	Dew Point	Kind Of preci.	Quantity Of Preci.
41.21.1.10	Petause M/4	01.02.05 00:04	1.9	-0.5	91	0.6	R	F2
41.21.1.11	Petause L/4	01.02.05 00:04	1.9	-1.5			R	F2
41.21.1.12	Jouxten L/5	01.02.05 00:04	0.5	-1.1			-	F
41.21.1.13	Jouxten M/5	01.02.05 00:04	0.5	0.4			-	F

In contrast, the MeteoSuisse weather stations are not necessarily positioned on roads. These stations provide numeric values of precipitation and this allows defining the threshold between "no rain" and "rain" conditions. These data are collected every 10 minutes.

Data from Boschung system are more suitable for the purpose of this research. Nevertheless, only data from Boschung weather stations in Vaud canton was available at the time of the study. That is why the selected traffic sites in Fig 2.3 are within the Vaud canton. For other traffic sites which are not in the Vaud canton, meteorological data used are provided by MeteoSuisse. The selected traffic sites are linked to the nearest weather stations. Fig 2.6 shows the link between selected traffic sites and weather stations.

Fig 2.6 Link between traffic sites and weather stations

Detector	Boschung stations
226	41.21.1.3
116	41.21.1.32
149	41.21.1.5

2.4 Crash data

Crash data is provided by the Federal Statistics Office - OFS. The crash database contains motorway crashes from 2002 to 2007 on all the Swiss motorways. When a crash happens, the police of the related canton fill in a questionnaire the information about the crash and send it to the OFS. Depending on each canton, the questionnaire can be completely or partially filled.

There are 46641 crashes happened on Swiss motorways during 6 years from 2002 to 2007. Fig 2.7 shows the crashes whose coordinates are provided. In most of Swiss cantons, coordinates of crashes are recorded. Some cantons such as Vaud, Geneva, Ticino, etc. do not provide crashes' coordinates. That is why in Fig 2.7, there is no crash sign in these cantons.

In Fig 2.8, 53.43% out of 46'641 crashes relate to only one vehicle. Rear-end collision is the second highest cause of crash which contributes more than ¼ out of all crashes, i.e. 11'682 rear-end crashes.

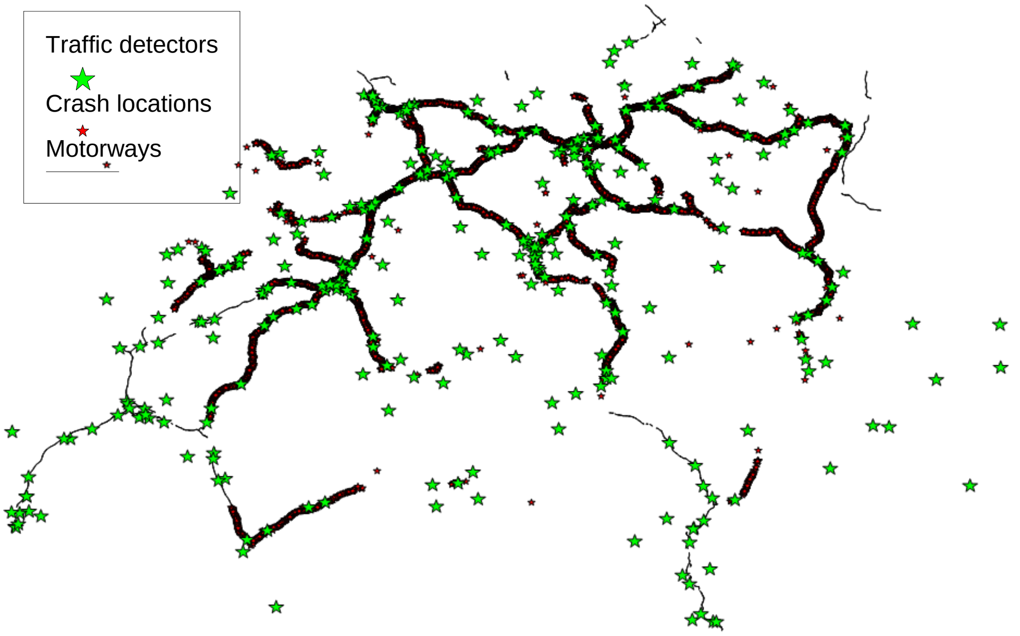


Fig 2.7 Motorways, traffic detectors and crash locations from 2002-2007 in Switzerland

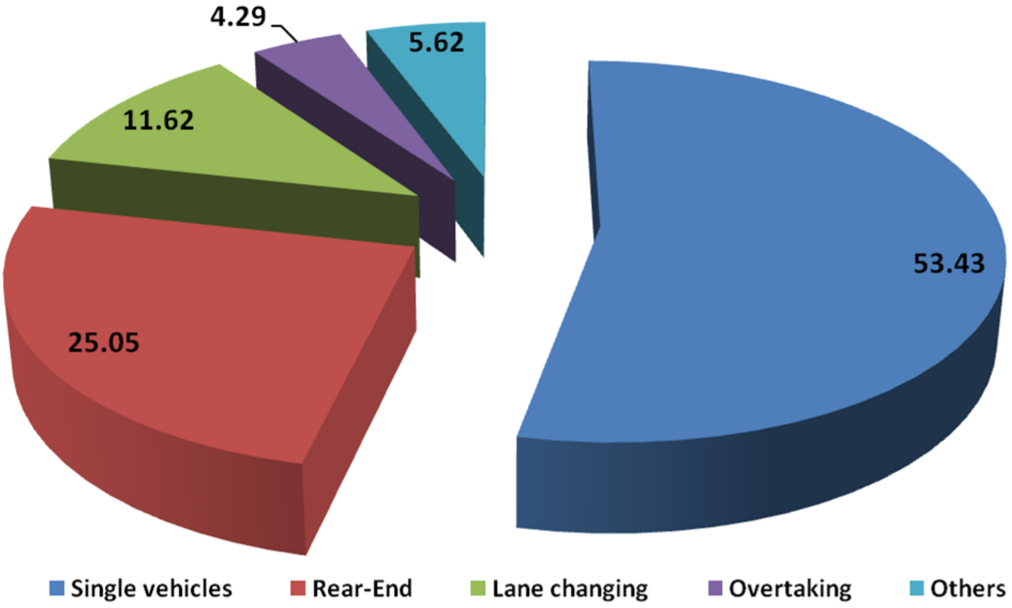


Fig 2.8: Crash distribution by crash types

2.5 Study sites

One of the conditions who dominate the selection of study site is that three types of data, i.e. crash data, traffic data, and road meteorological data should be altogether available at the one area and during the same period. By this condition, the study site is selected in Vaud canton because this is the only place where road meteorological data, i.e. data from Boschung station is available to this study. The selected study sites are presented in Fig 2.9.

Fig 2.9 Study site

3 Risk Indicators

3.1 State of the art

3.1.1 Introduction

Motorway system has a vital role in transport system in many countries. It links cities and allows fast movements that can only be undertaken by high speed supporting vehicles such as automobiles, trucks, and motorbikes. On the one hand, the fast movements on motorways allow reducing travel time. On the other hand, a series of motorway traffic safety regulations have to be applied to reduce traffic accident risks against such high speed traffic conditions where an accident on freeway once happens would be very serious.

Many studies have been undertaken to better understand and improve the motorway traffic safety. One of the research directions is to use traffic measures to characterize traffic situations for the purpose of identifying crash-prone traffic conditions and taking necessary operational actions to reduce the crash risk. These measures are called “risk indicators”, which are traffic parameters that allow describing partially crash risk status of a road section with its unchanged infrastructure configuration. The indicators can characterize traffic flow properties before the occurrences of events such as crashes, congestions, etc. that happened in the past. The indicators also allow identifying potential risky traffic conditions that may occur as the progress of the current traffic flow. It means in the both cases that traffic data have to be collected to be used in the analysis.

In this study, the term “risk indicator” may be replaced by the term “safety indicator”. For some indicators, their high values indicate high risk situations. They are called “risk indicators”. There are also indicators that their high values indicate safety situations. They are called “safety indicators”. If their values are low, they also indicate the crash risk.

3.1.2 Traffic conflict techniques

For many years, traffic engineers observed the traffic to find out the association between unsafe situations and operational problems. However, the procedures and conclusions were given based on opinion and judgment. The first traffic measure dated back in 1960s when a set of formal definitions and procedures for observing traffic conflicts at intersections was developed by two engineers working for General Motors [Perkins S. R., 1969]. This was then developed to Traffic Conflict Technique that is used in many countries (for instant, in Sweden since 1970, in England and in United States since 1979). The risk indicators proposed by these literatures are used to quantify the severity of hypothesized traffic crashes on many road types including motorways. Due to the lack of field data, the measures are principally based on traffic theory with the traffic assumptions and the manual count of vehicles by observers.

For a long time, Time-To-Collision (TTC) has been an important indicator that represents the time required for two vehicles to collide if they continue at their present speed and on the same path [Hayward, J. C., 1972]. TTC plays an important role in Traffic Conflict Techniques in many countries. [Van der Horst, R., et al., 1993] introduced TTC as a core parameter in the Collision Avoidance system. The TTC value of a vehicle implies the remaining time for that vehicle to collide with its front vehicle if both of them still keep their speed and travel direction. The smaller the TTC value, the more dangerous the situation is. Another condition for the collision to happen is that the speed of the considered vehicle has to be greater than the speed of its front vehicle.

$$TTC_i = \frac{v_{i-1} * Gap_i}{v_i - v_{i-1}} \qquad v_i > v_{i-1}$$

Where TTC_i, V_i, Gap_i are TTC value, speed, and time gap of vehicle I – the considered vehicle. V_{i-1} is the speed of its front vehicle.

Other similar to TTC indicators were then presented within the Traffic Conflict Techniques in different countries. Without being exhaustive, the following indicators could be named:

- Deceleration to safety time (DTS) [Hupfer, C., 1997],
- Post-Encroachment-Time (PET) [Cooper, P.J., 1984],

- Time-Exposed-TTC (TET) and Time-Integrated-TTC (TIT) [Minderhoud, M. et al, 2001].

[Uno, N. et al., 2005] proposed PICUD (Potential Index for Collision with Urgent Deceleration): Assuming that the leading vehicle applies its emergency brake and that the following vehicle brakes at the same rate after a reaction time of one second, the distance between the two vehicles considered when they completely stop is forecasted by the following formula :

$$\text{PICUD} = \frac{\left[V_1^2 - V_2^2\right]}{\gamma} - V_2.T + (x_1 - x_2)$$

V_1, V_2: initial velocities of leading car 1 and following car 2, respectively

x_1, x_2: initial positions of leading car 1 and following car 2, respectively

T driver's reaction time,

γ deceleration rate at stop.

The authors observed 62 lane changes during one hour on a part of a weaving section, and computed TTC and PICUD for the merging vehicle and also for the vehicle following the merging vehicle on the new lane.

The speed differences between the speed of the merging vehicles and the followers were not important, so TTC was rarely small, whereas PICUD was frequently negative (67% of the case, due to the inclusion of the reaction time of 1 second). Note that they obtain this result with a rather low emergency braking (3.3 m/s²), which strengthens their conclusion on the power of PICUD compared to TTC.

Although these risk indicators can be adapted to traffic conditions on motorways, they are mostly used to evaluate safety on urban roads, especially for intersections. Some of the above indicators, such as PET, TIT, and TET require more data input than the others, which is difficult to implement in reality.

3.1.3 Crash prevention models

[Pande, A. et al., 2006] look into the relationship between real traffic data and rear-end crashes. They use traffic and crash data collected from 36.3 miles of freeway in Orlando over 5 year 1999-2005 for the analysis. To refine the crash time, they pre-processed crash data by using a refined rule-based shockwave method to find more accurate crash time. Then, for each crash data from 7 detectors (named from B to H where B-F are upstream detectors and B is the farthest from the accident position; and G and H are downstream detectors) within 30 minutes before the crash are processed. The 30-minute period is then divided into 6 5-minute time slices for each of which raw data are aggregated. For each slice, 6 parameters are considered: average and standard deviation of speed, volume and lane occupancy. The parameters from traffic condition before the crash are then compared to the same parameters from the other traffic conditions of the same time, on the same day of other weeks by using several data mining techniques. By this method, they find that rear-end crashes often occur under extended congestion or when average speeds were high. With this conclusion they suggest that there should be some warnings to drivers when they are under such traffic situations and the development of variable speed limit strategies should be realized to reduce the risk of rear-end crashes.

Based on crash report from the police, [Lee C. et al., 2006a] classified crashes into different types of which rear-end type has a very high rate and the rate of sideswipe type is the second highest (52% and 16% respectively. [Lee C. et al., 2006a] investigated into sideswipe crashes which occur when vehicles change lanes in comparison to rear-end crashes. They used the same traffic and crash data as described in [Pande, A. et al., 2006] with the help of logistic regression. The seven indicators examined were average speed, flow, occupancy of each 30 second over 5-10 minutes before the crash, coefficient of variation in speed (CVS), coefficient of variation in flow (CVF), peak/off-peak period and the curvature of the road section. A new indicator – Overall Average Flow Ratio (OAFR) was defined and is a revised parameter from the AFR (Average Flow Ratio) accounting for imbalance of lane flow across neighbouring lanes during short time periods (5 minutes) proposed by [Chang, G. and Kao, Y., 1991].

$$AFR_i(t) = \frac{\frac{V_{i-1}(t)}{V_i(t)}\ NL_{i-1,i}(t)}{NL_{i-1,i}(t) + NL_{i-1,i-2}(t)} + \frac{\frac{V_{i+1}(t)}{V_i(t)}\ NL_{i+1,i}(t)}{NL_{i+1,i}(t) + NL_{i+1,i+2}(t)}$$

for lane i where V_i is the speed and $NL_{i,j}$ the number of lane changes from lane i to lane j

for the left most lane i=1, the part corresponding to lane i-1 does not exist

for the right most lane i=n, the part corresponding to lane i+1 does not exist.

OAFR(t)= $\sqrt[n]{AFR_1(t)\ \text{x}\ \ AFR_i(t)\ \ .\ .\ .\ .\text{x}AFR_n(t)}$ for the whole set of lanes

The traffic condition before the crash is characterized by the detector station immediately upstream. After the evaluation of the effect of each indicator, only three of them are concluded to be useful for the analysis. They are OAFR, CVF and peak/off-peak. With the assumption that the fractions of lane changes from lane 2 to lane 1 and to lane 3 are equal, they find that OAFR is in general higher for sideswipe crashes than for rear-end ones and this indicates that higher variations of flows across lanes contribute more to occurrences of sideswipe than rear-end crashes. They also find that sideswipe crashes occur more often under uncongested condition (off-peak) when it is easier for drivers to find gaps to change lanes. The increase of lane changes may raise the risk of sideswipe crash. For this reason, reducing OAFR and CVF by balancing flow across lanes and allowing smooth lane change over longer distance may help reduce high risk of sideswipe crashes. However, this conclusion was drawn when weather effect was not taken into the account and crash time was believed to be accurately recorded.

Through other studies (published from 2004 to 2007) on the same dataset, [Abdel-Aty, M. et al., 2004a], [Abdel-Aty, M. et al., 2004b], [Abdel-Aty, M. et al., 2006], [Abdel-Aty, M. et al., 2007a], and [Abdel-Aty, M. et al., 2007b] separate crash prone conditions from normal freeway traffic with an algorithm based on 30 second traffic data. For every crash, he gathered a set of data including speed, occupancy, volume, the temporal variation of speed, the upstream occupancy, the differential in speeds upstream and downstream, weather data, other characteristics such as the presence of ramps and horizontal curvature.

For rear-end crashes, he found a bi-modal distribution of average 5- minute speed just before the accident (15-30 miles/hour and 55-60 miles/hour) and consequently developed two separate models for the two groups of crashes. The main explanatory variables were:

In the low-speed model: a high coefficient of variation of speed, a high average occupancy, a low standard deviation of volume. Traffic conditions for this regime are not frequent (6%) but correspond to 46% rear-end crashes, which warrants implementing a warning system in the high-speed model: a high average occupancy (downstream) and a lower occupancy upstream, a difference between upstream and downstream speeds, a high standard deviation of the speed downstream. A warning system, derived from the results of this model should capture (for the given thresholds) up to 53.7% of rear-crashes of this regime, being switch on in 30 % of the time.

For lane changes crashes, the main explanatory variables were the average speed, average occupancy, the average of absolute difference between 30-second occupancy observations on adjacent lanes, the standard deviation of volume and the standard deviation of speed.

The crash database has been split in two distinct sets, one used for calibration and one for validation (these crashes were not used for calibration).

A very interesting piece of work of [Abdel-Aty, M. et al., 2006], using the same dataset (3,146 crashes and 30 seconds traffic data from a 36.25 miles instrumented corridor of I-4 in Orlando, Florida metropolitan region from 1999 to 2002) consisted in five binary categorizations of the crashes (multiple/single crashes, peak/off peak, dry/wet pavement, daytime/dark hour, property damaged/injury crashes), and estimated separately the corresponding models of the frequency of each of the two groups. To account for correlation between the disturbance terms arising from omitted variables, seemingly unrelated negative binomial (SUNB) regression was used. The explanatory variables were:

- the annual average daily traffic,
- peak 15-minute volume,
- average speed, represented by its 75th percentile,
- standard deviation of speed, represented by its 75th percentile. Standard deviation of speed was computed from the series of speed either on a 5- minute period (the ten 30 seconds recorded speed during this period), 10-minute or 15-minute period,
- 75th percentile of coefficient of variation of speed, number of lanes, median type,
- median width, pavement surface type,
- road curvature,
- presence of on-or off- ramp, etc.

Road curvature and presence of on-or off- ramps were found to be significant factors. AADT was significant in most models and the 15-minutes coefficient of variation of speed was significant for frequency of daytime and peak-period crashes.

In the models described in this section, the same data set was used and the data is aggregate. Due to the lack of data source, weather information is not integrated into the models.

3.1.4 Previous studies in LAVOC

In LAVOC there was a study in traffic safety and is reported in [Dumont, A.-G. et al, 2005] and [Huguenin, F., et al, 2005]. The authors proposed a new safety indicator called Unsafety Density – UD and defined as below:

$$\boxed{UD = \frac{\sum_{s=1}^{S_t}\sum_{v=1}^{V_t} U_{v,s} \cdot d}{T \cdot L}} \quad [\mathrm{m/s^2}]$$

With

UD : unsafety density $[\mathrm{m/s^2}]$

$U_{v,s}$: unsafety of vehicle v in simulation step s $[\mathrm{m^2/s^2}]$

V_t : number of vehicles in the link [-]

S_t : number of simulation steps within aggregation period [-]

d : simulation step duration [s]

T : aggregation period duration [s]

L : section length [m]

Where:

$$\boxed{U = \Delta S \cdot S \cdot R_d} \quad [\mathrm{m^2/s^2}]$$

With

U : unsafety parameter $[\mathrm{m^2/s^2}]$

ΔS : speed difference between two vehicles at collision time [m/s]

S : speed of the follower's vehicle at collision time [m/s]

$R_d = \frac{R}{R_{max}}$: ratio between the deceleration of the leader vehicle and its maximum deceleration capacity [-]

3.1.5 Other risk indicators

Recently, [Aron, M. et al, 2003] presented two indicators: IBTR (Individual Braking Time Risk) or G-value and PBTR (Platoon Braking Time Risk) or J-values. The G-value represents the rear-end crash risk of a vehicle when its front vehicle stops. G-value is characterized by the relationship between the speed of that vehicle and the time gap between that vehicle and its front vehicle. If the speed is high while the time gap is low, the risk becomes higher.

$$G(i) = \mathrm{Max}\left[0, \log_2\left(\frac{Tb}{2.Gap_i}\right)\right] = \mathrm{Max}\left[0, \log_2\left(\frac{1}{2}.\frac{v_i}{|\gamma_{max}|}.\frac{1}{Gap_i}\right)\right]$$

Where γ_{max}, V_i, Gap_i are the maximum deceleration rate, the speed, and the time gap of the considered vehicle. The maximum acceleration rate is defined based on pavement conditions and should reflect meteorological conditions on the road. The definition of IBTR also comply with the standard values of time gap and speed limit: if a vehicle keeps standard time gap and travels at speed limit, its crash risk will be zero.

The risk of collision with the preceding vehicle increases when vehicles are considered in a platoon: the number of “events” which can affect the current vehicle is proportional to the number of events which may occur on the preceding vehicles, so to the number of these vehicles.

That is why the Platoon Braking Time Risk (PBTR) or J-value for the vehicle “i” of the platoon is defined by the following formula:

- for the leader of the platoon: J(1)= 0
- for the vehicle “i”, the PBTR is: if G(i) >0; J(i)=G(i)+J(i-1)
- if G(i) =0; J(i)=0

For the vehicle “i” the PBTR is additive, but vanishes as soon as one vehicle is “safe”. PBTR is an accumulative risk indicator. The values of this parameter can be obtained based on individual vehicle data, e.g. time gap between two consecutive vehicles, individual vehicle speed which can be collected from motorway traffic loop detectors.

3.2 Potential Risk Indicators

In section 3.1, different types of risk indicators are presented. The selection of risk indicators for this study is undertaken. As this study focuses on motorway traffic safety in Switzerland and due to the particularity of individual traffic data collected from traffic detectors, the selected risk indicators should match to the following criteria:

- Can be obtained based on a single traffic detector. Some of risk indicators in section 3.1 can be calculated based on a single traffic detector but their usage requires the presence of many adjacent detectors. The risk indicators like that are also excluded from this study.
- Can be obtained based on individual traffic data.

The potential risk indicators are listed in Fig 3.1. These are the risk indicators that satisfy the aforementioned criteria.

Fig 3.1 List of potential risk indicators

Risk Indicators	Description
TTC	Time To Collision
IBTR/G-value	Individual Braking Time Risk
PBTR/J-Value	Platoon Braking Time Risk
PICUD	Potential Index for Collision with Urgent Deceleration
SOSL	Speed Over Speed Limit

3.3 Sensitivity analyses of risk indicators using field data

3.3.1 Objectives

This study looks at the status of traffic safety through the patterns of risk indicators. The understanding about the performance of risk indicators under traffic conditions in the past is important to determine if the risk indicators are applicable to monitor road traffic risk and if they can be integrated into a pro-active risk monitoring system.

The sensitivity test of risk indicators is a way to find out the performance of risk indicators with data in the past, including traffic data, meteorological data, and crash data. The sensitivity test composes of two following steps:

- Find out the distributions of risk indicators from which, extreme values as well as bound of risk indicators can be identified.
- Check the evolution of risk indictors' values just before crashes. As risk indictors bear information about crash risk on the road, their values should reflect the crash risk. However, it is not always evident to observe a traffic conditions with high crash risk. Therefore the traffic conditions just before crashes are good moments to check the risk indicators because the crash risk at that time turned into real crash later.

The sensitivity test should provide the performance evaluation of risk indicators which serves as the guideline for the later usage of risk indicators in other stages of this study.

3.3.2 Traffic safety influencing factors

Traffic safety can be influenced by many factors such as geometry of the road section, meteorological conditions, traffic flow as well as drivers' behaviors. This research aims at improving traffic safety by reducing accident risk that can be caused by the meteorological conditions and by traffic flow. Hence, the selections of a straight and flat ATC-equipped section and of non-DUI crashes help minimize the effect of other factors.

The factors below are considered in this research:

Traffic Flow
Motorway traffic flow is self-adapted with the interaction of vehicles with various operational regulations such as speed limit, minimum time gap, etc. Traffic flow can affect traffic safety in many ways. In this paper, different ranges of traffic volume will be considered to test the sensitivity of safety indicators.

Meteorological Conditions
There are three weather conditions according to precipitation types from weather data: no precipitation (fine weather), rain, and snow. With the selected safety indicators, the effect of each weather condition can be quantified by a parameter such as the maximum deceleration rate (see [Chung (Ed.), 2007]) as shown in Fig 3.2

Fig 3.2 Maximum deceleration rate is a function of weather conditions

Weather conditions	Maximum deceleration rate(m/s^2) - γ_{max}
Fine weather	6.87
Rain	4.81

3.3.3 Sensitivity analyses

Processing steps
Traffic data and meteorological data for the whole year of 2005 were used to observe the distribution of selected risk indicators. The algorithm to process the data includes following steps:

- Calculate risk indicator values for each vehicle. This calculation involves traffic data and meteorological data. For each vehicle the selected data set, there is a set of values calculated for it such as speed, headway, time gap, vehicle type, and risk indicators' values.
- Aggregate the data by lanes with the aggregation interval of 5 minutes. The aggregation process should provide the following information for each 5-minute interval:
    - The traffic volume of the lane during that 5-minute interval
    - The average speed
    - The distribution of vehicle types
    - The distributions of risk indicators.
- Determine the flow range of that 5-minute interval.

Flow ranges are used because traffic properties under each traffic flow range are very different. To understand the performance of risk indicators, they should be considered under separated flow ranges. There are five flow ranges used in this study representing the traffic flow conditions from free flow to dense flow. The five flow ranges are: 0-500vhp, 500-800vph, 800-1100vhp, 1100-1500vph, and greater than 1500vph.

In the next sub-sections, the distributions of selected risk indicators are presented. Some of the risk indicators may be influenced by the meteorological conditions and as consequence, their distributions under two meteorological conditions are essential. The two meteorological conditions are fine meteorological conditions and rain conditions. At the study site, road sections contain two traffic directions. For each direction, there are two lanes. The left lane is called the slow lane while the right lane is called the fast lane.

As the goal of this study is to focus on risky traffic situations, the safety domains of risk indicators are not presented in the figures of section 3.3.3. Depending on risk indicators, the risk domains of risk indicators can be their high or low values.

IBTR or G-value distribution

In Fig 3.3, Fig 3.4, Fig 3.5, and Fig 3.6, the IBTR distributions are presented for the slow lane and the fast lane, under fine conditions or rain conditions.

On the same lane, IBTR tends to be higher under rain conditions. This trend is expressed in two ways:

- Upper bound values under rain conditions are higher than upper bound values under fine meteorological conditions. For example, on the slow lane, the maximum IBTR value can reach to 9 under fine meteorological conditions (see Fig 3.3, flow range greater than 1500vph) and 10 under the rain conditions (see Fig 3.5, flow range greater than 1500vph).
- Higher percentage of vehicles having near IBTR upper bound value under rain conditions than under fine meteorological conditions. This means that there are more vehicles seen as risky by IBTR.

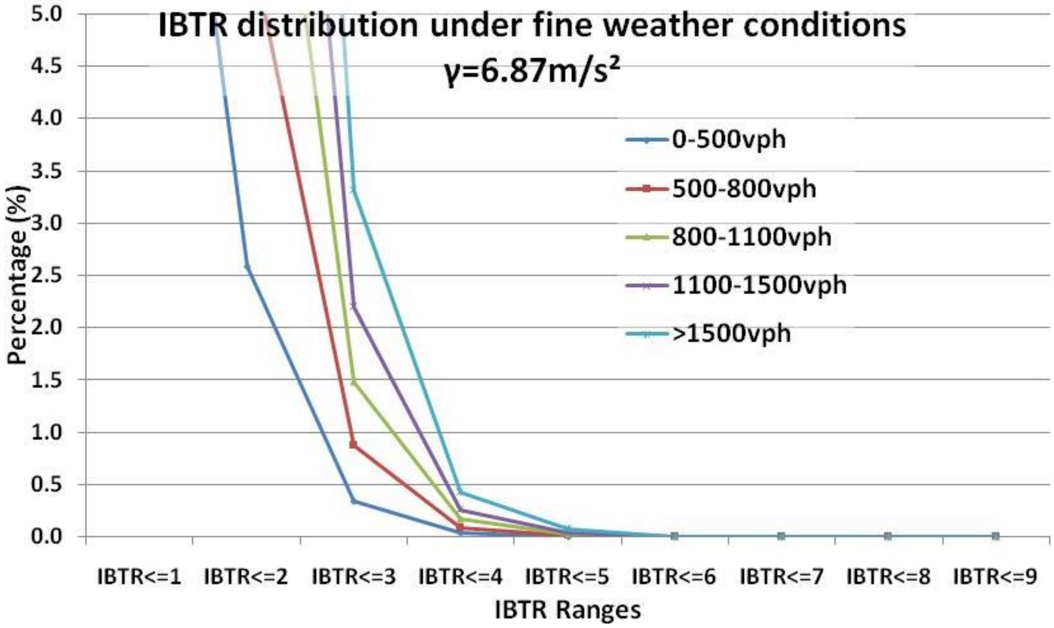


Fig 3.3 IBTR distribution under fine meteorological conditions on the slow lane

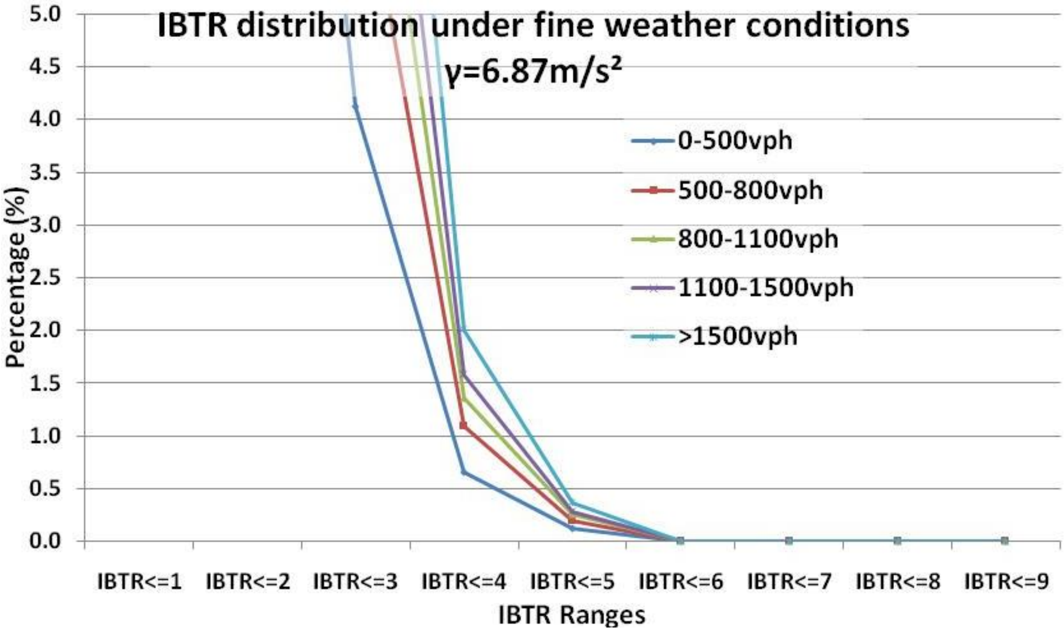


Fig 3.4 IBTR distribution under fine meteorological conditions on the fast lane

Under the same meteorological conditions, IBTR values tend to be higher on the fast lane than on the slow lane. Comparing Fig 3.3 with Fig 3.4 (fine meteorological conditions) or Fig 3.5 with Fig 3.6 (rain conditions), for the same IBTR range, higher percentage of vehicles having that IBTR range on the fast lane than on the slow lane. This is due to the higher average speed on the fast lane.

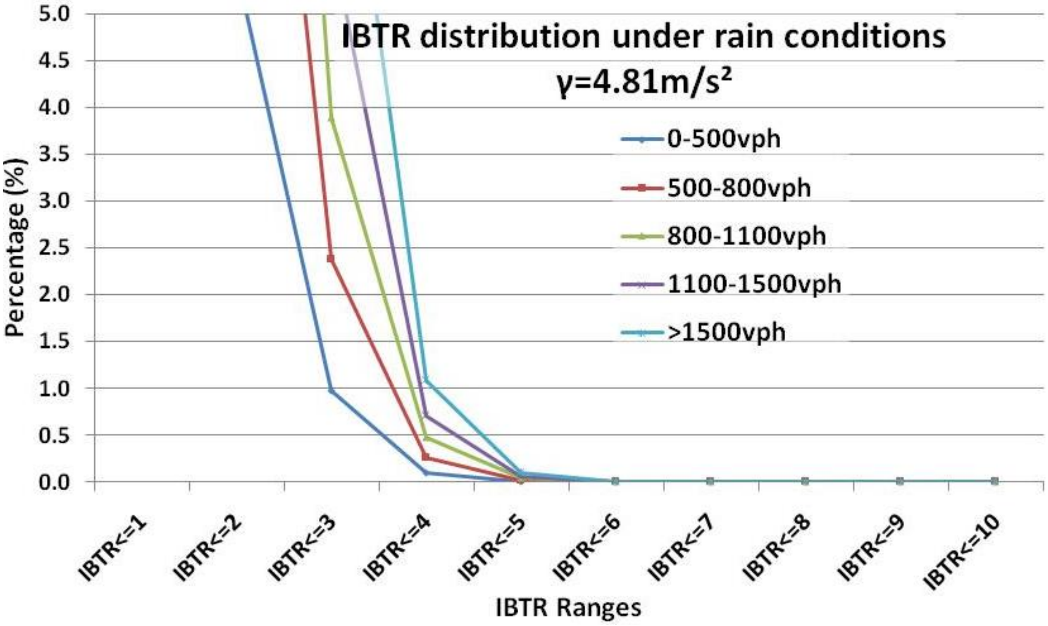


Fig 3.5 IBTR distribution under rain conditions on the slow lane

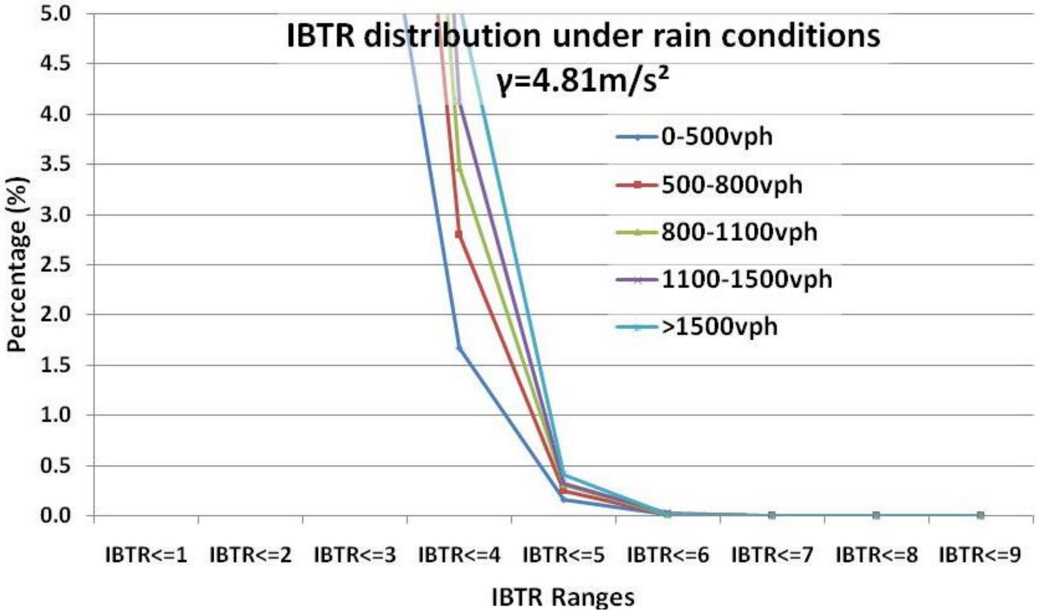


Fig 3.6 IBTR distribution under rain conditions on the fast lane

PBTR or J-value distribution

PBTR is an accumulation of IBTR over vehicles in a platoon. Therefore, value ranges of PBTR are extended and the trends that are observed with IBTR become bolder. Under fine meteorological conditions, PBTR values can raise up to 37 on the slow lane (see Fig 3.7) and more than 49 on the fast lane (see Fig 3.8).

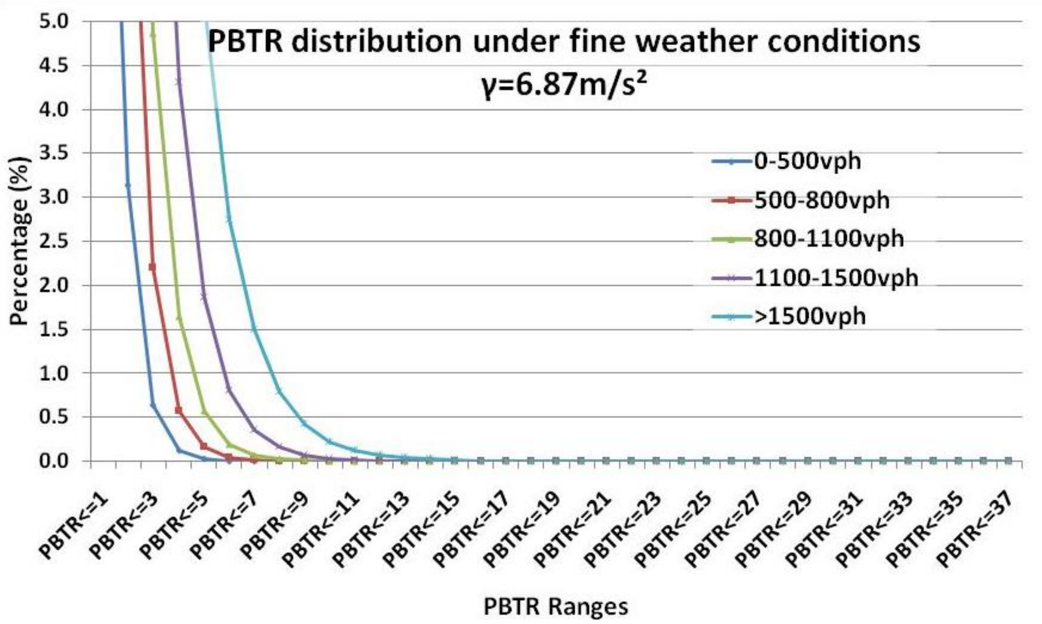


Fig 3.7 PBTR distribution under fine meteorological conditions on the slow lane

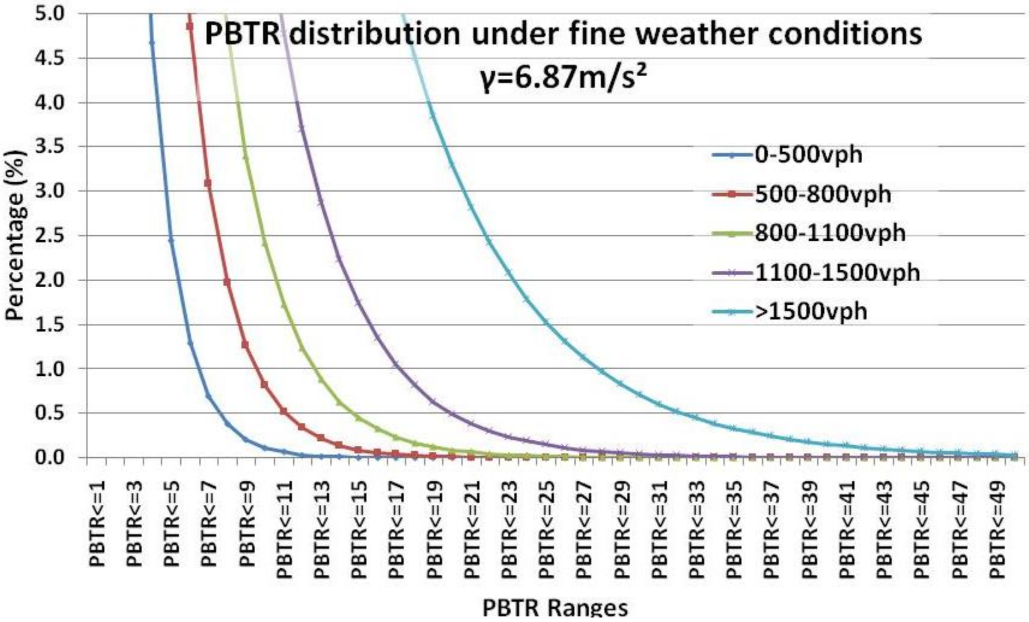


Fig 3.8 PBTR distribution under fine meteorological conditions on the fast lane

PBTR is more sensitive to meteorological conditions. For example, on the slow lane, the upper bound of PBTR is 37 for high flow range when the meteorological conditions are fine and can rise up to 43 when it rains.

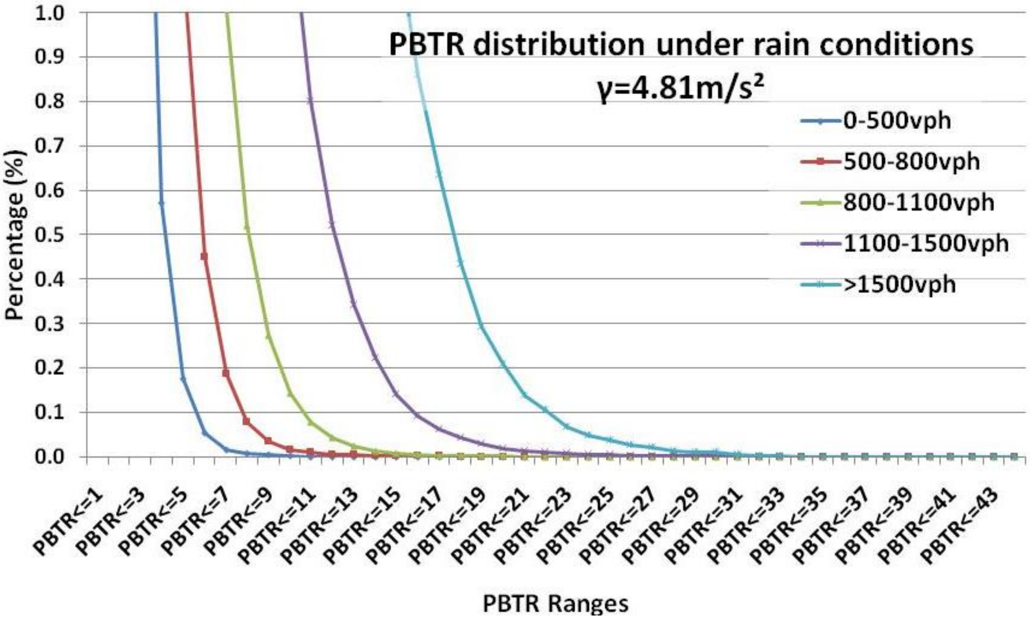


Fig 3.9 PBTR distribution under rain conditions on the fast lane

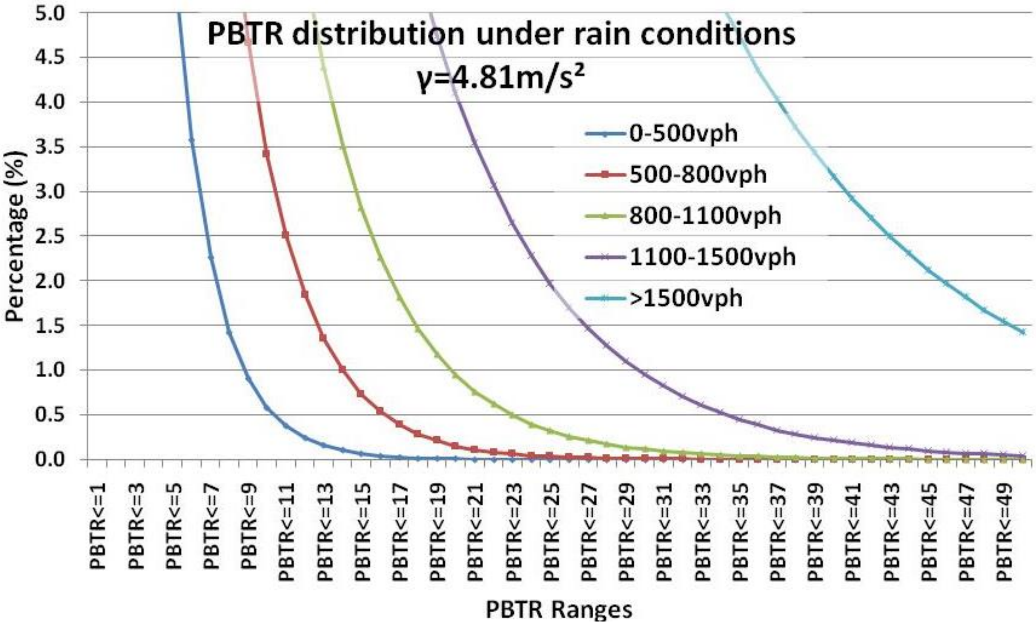


Fig 3.10 PBTR distribution under rain conditions on the fast lane

PBTR is also more sensitive to flow ranges with the clearer separation of curves in Fig 3.7, Fig 3.8, Fig 3.9, and Fig 3.10. Under higher flow ranges, vehicles move closely while maintaining high speed which causes high percentage of vehicles having high PBTR values.

TTC distribution

On the contrary to IBTR and PBTR, TTC indicates higher crash risk when its value decreases. Therefore, low TTC values are more interesting in this study than its high values. Fig 3.11 and Fig 3.12 show the TTC distributions on the slow lane and on the fast lane respectively.

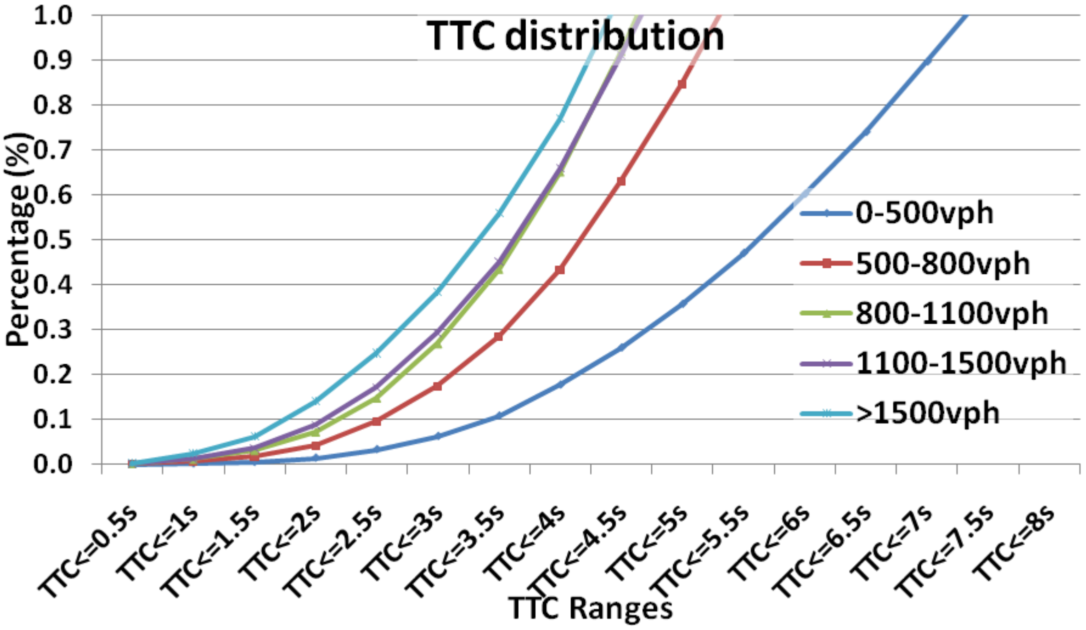


Fig 3.11: TTC distribution on the slow lane

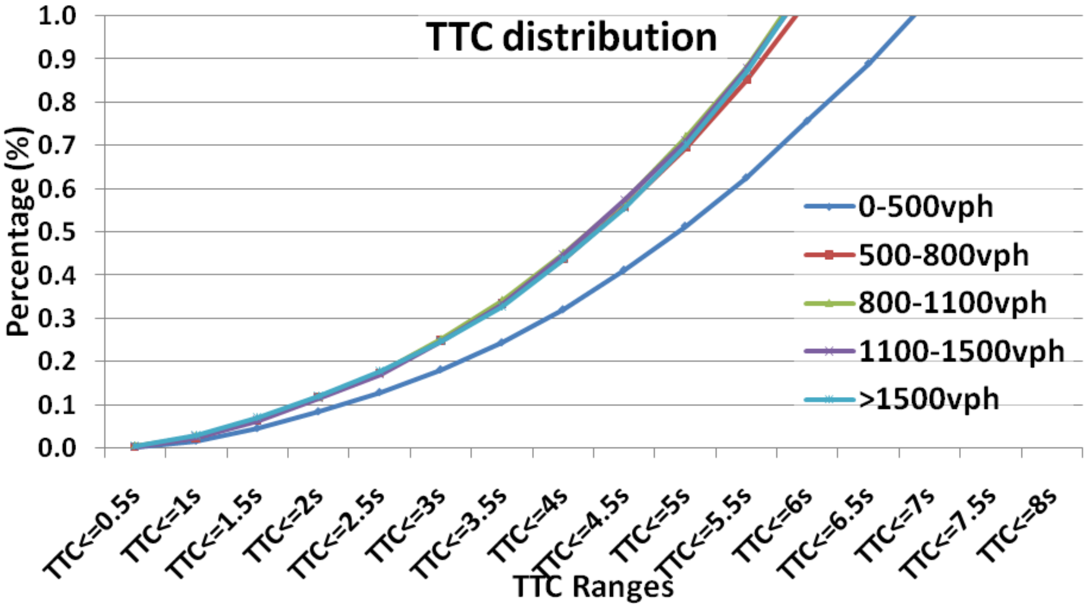


Fig 3.12: TTC distribution on the fast lane

In Fig 3.11, TTC distribution under free flow is very different from other flow ranges while for two flow ranges, 800-1100vph and 1100-1500vph, TTC distributions are almost overlapped. This means that TTC distribution is relatively stable under a wider flow range.

In Fig 3.12, TTC distribution under free flow is separated from TTC distributions under other traffic flows ranges. Apart from the free flow range, there are always 1% of vehicles having TTC smaller than 5.5 seconds.

It can be seen from Fig 3.11 and Fig 3.12 that TTC is less sensitive to traffic flow than IBTR or PBTR.

SOSL distribution

Motorway speed limits are speed levels that are recognized by law and any violation to the speed limits is considered as risky. Fig 3.13 and Fig 3.14 shows the distributions of SOSL on the slow lane and on the fast lane.

For the same flow range, the percentages of vehicles running faster than speed limits on the slow lane are lower than on the fast lane. This is normal because vehicles usually come to the fast lane to overtake and then come back to the slow lane.

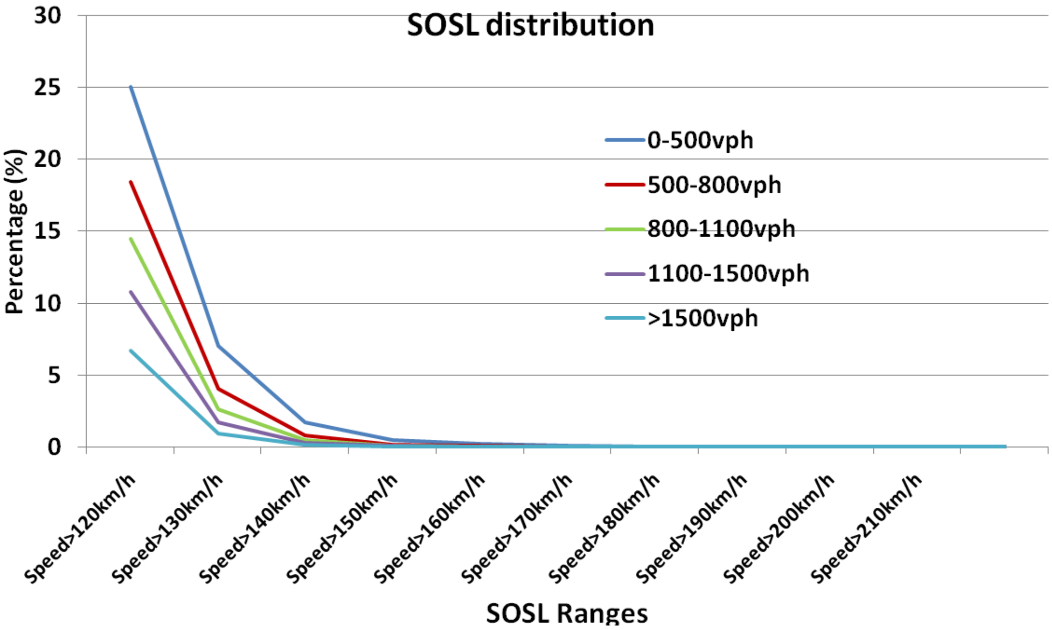


Fig 3.13 SOSL distribution on the slow lane

SOSL is also sensitive to flow conditions: under higher flows, it is difficult for vehicles to accelerate because the traffic is dense. On the contrary, the percentage of vehicles with speed greater than the speed limit is the highest under the free flow when drivers can find font space to accelerate their vehicles.

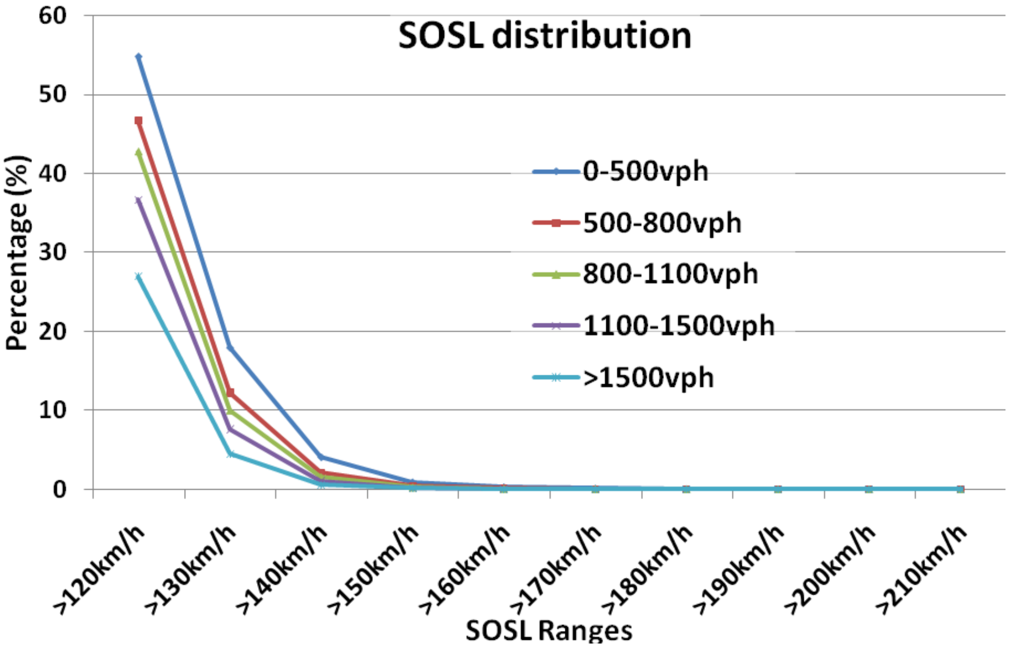


Fig 3.14 SOSL distribution on the fast lane

PICUD distribution

PICUD represents the remaining distance between two vehicles when the front vehicle brakes. If that distance is smaller, the situation becomes more risky. Therefore, low PICUD values are more interesting to this study.

PICUD distributions are shown in Fig 3.15, Fig 3.16, Fig 3.17, and Fig 3.18. In these figures, the common trend is that PICUD distribution curves under different traffic flow ranges are clearly separated. This means that traffic flow has a sharp influence to PICUD distribution. When the traffic flow increases, the distant gap between vehicles is reduced and the vehicle speed becomes more homogenous which causes smaller PICUD values. That is why there are more vehicles having low

PICUD under higher flow ranges.

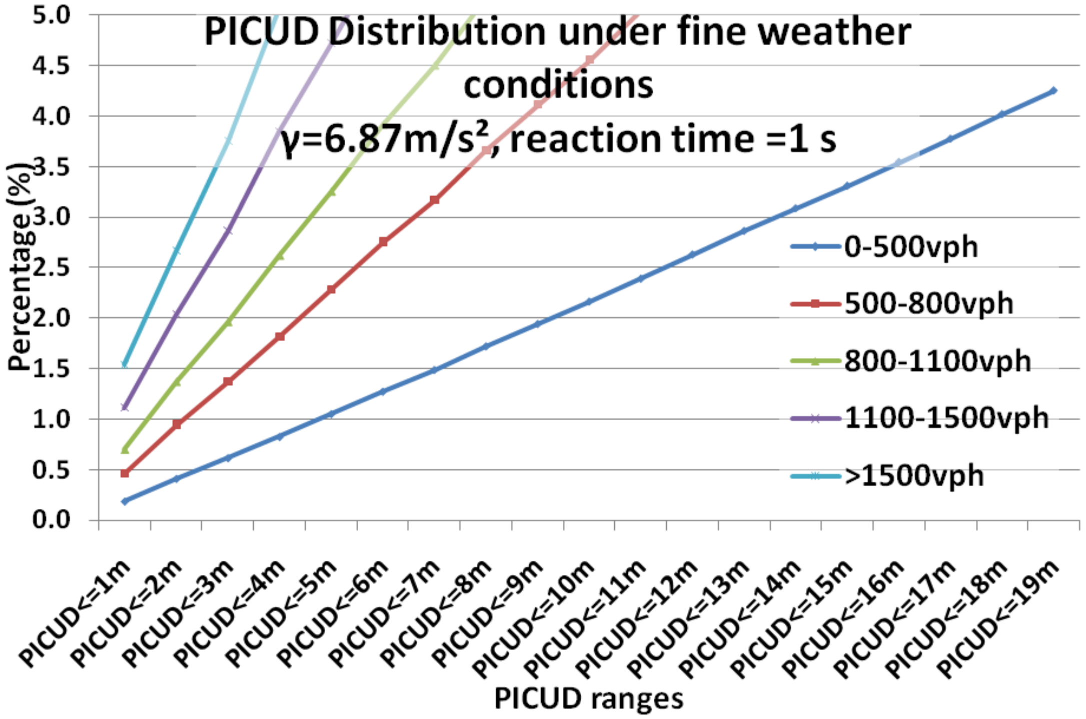


Fig 3.15 PICUD distribution under fine meteorological conditions on the slow lane

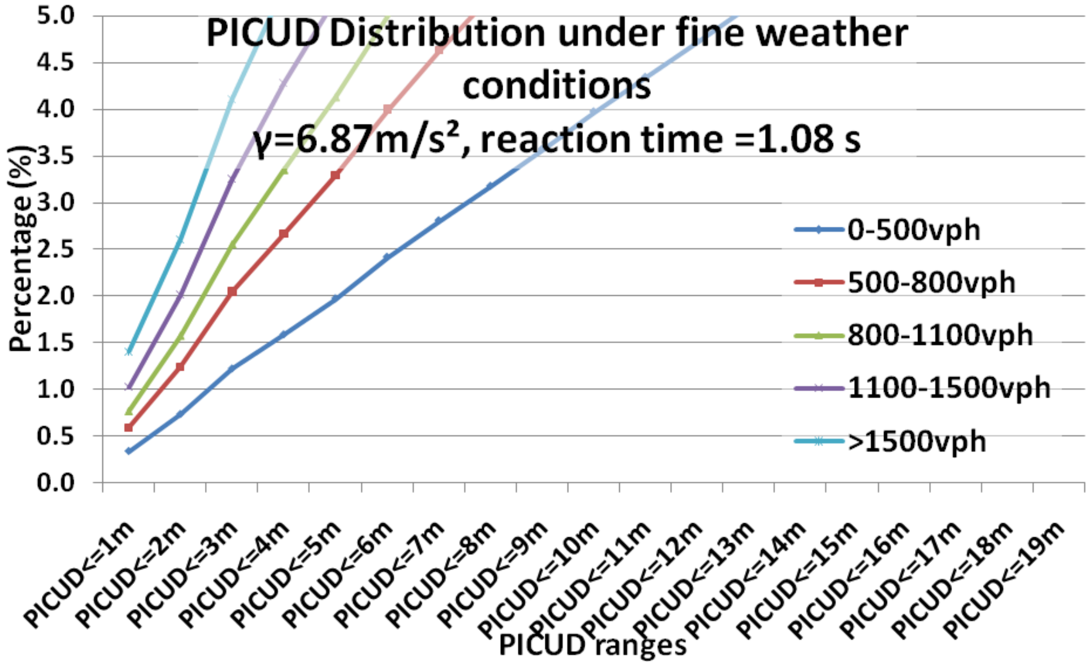


Fig 3.16 PICUD distribution under fine meteorological conditions on the fast lane

Under the same meteorological conditions, more vehicles on the fast lane are indicated as risky by PICUD. For example, under fine meteorological conditions and under high flow range (greater than 1500vph); there are 3.5% out of vehicles having PICUD smaller than 3m on the slow lane while this percentage is 5% on the fast lane.

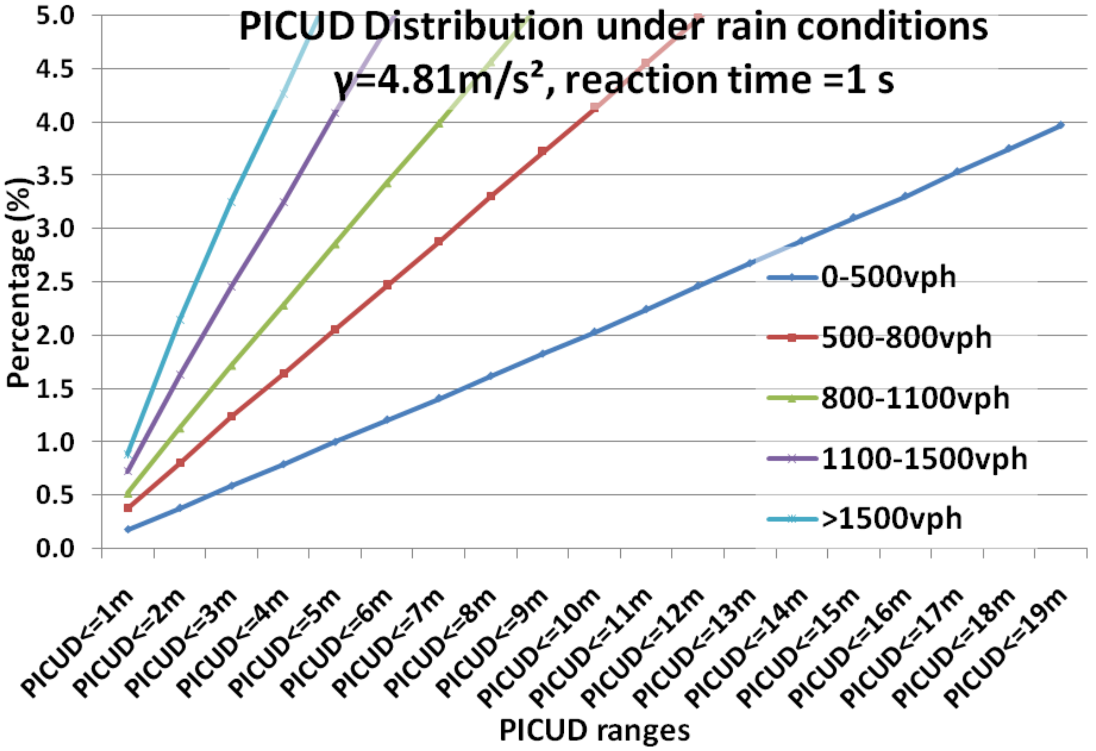


Fig 3.17 PICUD distribution under rain conditions on the slow lane

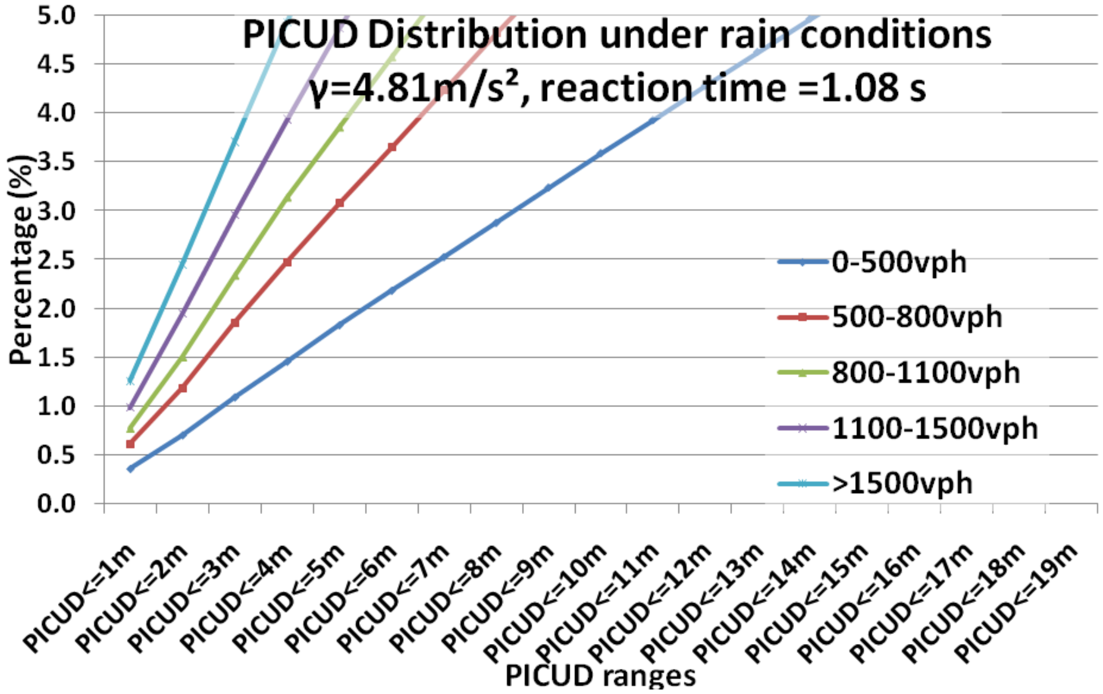


Fig 3.18 PICUD distribution under rain conditions on the fast lane

About the influence of meteorological conditions, PICUD tends to be lower under fine meteorological conditions than under rain conditions. For example, considering the slow lane, in Fig 3.15, there are 5% out of vehicles under high flow range (greater than 1500vph) and under fine meteorological conditions having PICUD smaller than 4m while this percentage is about 4.3% under the same flow range and under rain conditions (see Fig 3.17). This result is unexpected because rain conditions are supposed to have bad effect on traffic safety. For this reason, PICUD will not be considered in the later stage of this study.

3.3.4 Crash cases

Crash cases are important in testing the performance of risk indicators because the traffic situations just before occurrences of crashes contain crash risk which ended in crashes. These traffic situations are the only known risky situations so far.

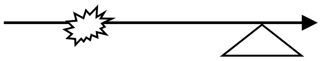
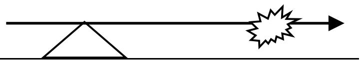
In this sub-section, the details about the evolution of risk indicators before several crashes are described and the results with all processed crashes are summarized.

Motorway crash data at the study site (see section 2.5), contain information about crash positions, date and time of crashes, weather conditions, pavement conditions, lighting conditions, types of crashes, and drivers' status during the crashes. Data in the files are coded according to OFS code (Office fédéral de la statistique). The time of a crash is stored under the form of a one-hour period where the crash happened: e.g. if a crash happened sometime between 02:00PM and 03:00PM, then the time of the crash recorded is 14:00-15:00. During 4 years (2002-2005), there are totally 3693 crashes on motorway sections in Vaud canton, among which 470 (12.7% of all crashes) are DUI crashes.

All DUI-crashes are not interested in this study and they are excluded. The non-DUI crash database contains 3223 crashes. The study focuses only on study site including two motorways N1 (the section from Lausanne to Bern – N1LB), and N9 (the section from Lausanne to Sion, N9LS) where traffic data are available from the two ATCs 116 and 149. Eliminating crashes on other road sections, the number of crashes is 1940.

To test the evolution of traffic before and during a crash, the traffic patterns leading to the crash need to be analyzed. However, the constraint of distance between a crash and the location of the nearest ATC causes the strong reduction of number of crashes that can be used in this study. The locations of the ATCs are fixed points. The crashes are selected if they are inside a buffer of 500m around an ATC. Fig 3.19 shows the number of Non-DUI crashes for each ATC.

Fig 3.19 Crash distribution by ATCs' locations

ATC	Upstream crashes 	Downstream crashes 
116	15	14
149	24	18

Many tests were done to validate different risk indicators and are reported in different publications. In [Pham, M.-H. et al., 2007], TTC and PBTR are tested with a crash described in Fig 3.19. A method was presented to find the exact time of the crash and to analyze the evolution of TTC and PBTR.

About the influence of meteorological conditions, PICUD tends to be lower under fine meteorological conditions than under rain conditions. For example, considering the slow lane, in Fig 3.15, there are 5% out of vehicles under high flow range (greater than 1500vph) and under fine meteorological conditions having PICUD smaller than 4m while this percentage is about 4.3% under the same flow range and under rain conditions (see Fig 3.17). This result is unexpected because rain conditions are supposed to have bad effect on traffic safety. For this reason, PICUD will not be considered in the later stage of this study.

Fig 3.20 Crash information

Date & Time	Sunday, 2005-01-23, between 20:00-21:00
Position	A9, direction Sion-Lausanne: 15km500. This crash happened at upstream from the ATC 116.
DUI-crash?	No
Type of accident	Single vehicle accident
Weather	Fine
Pavement	Dry
Lighting condition	dark

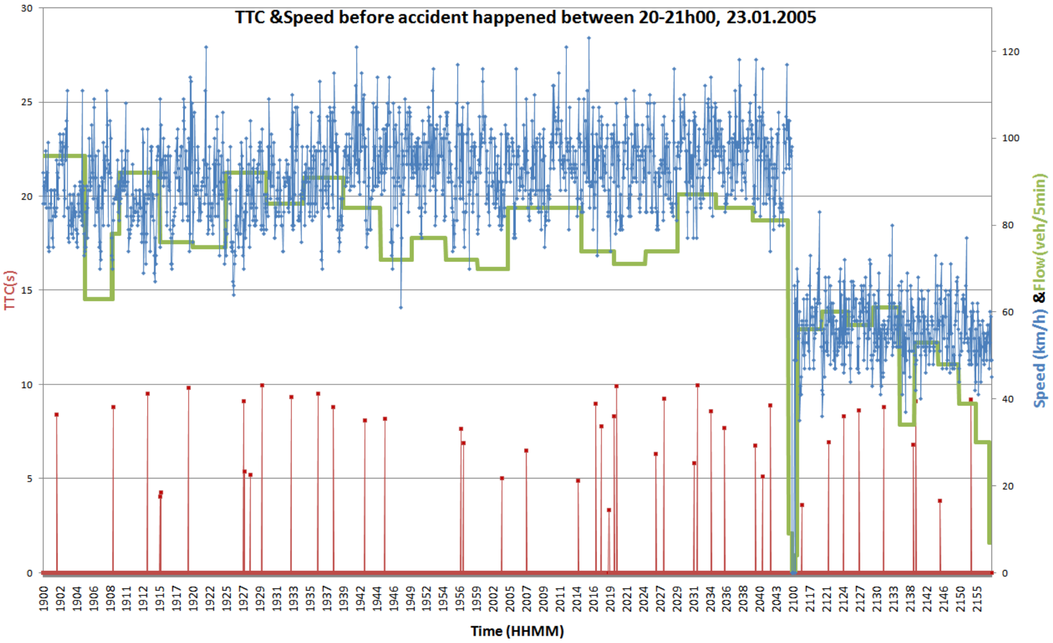


Fig 3.21 Flow and Individual TTC, speed on the normal lane when the crash happened. TTC values greater than 10s are made equal to zero.

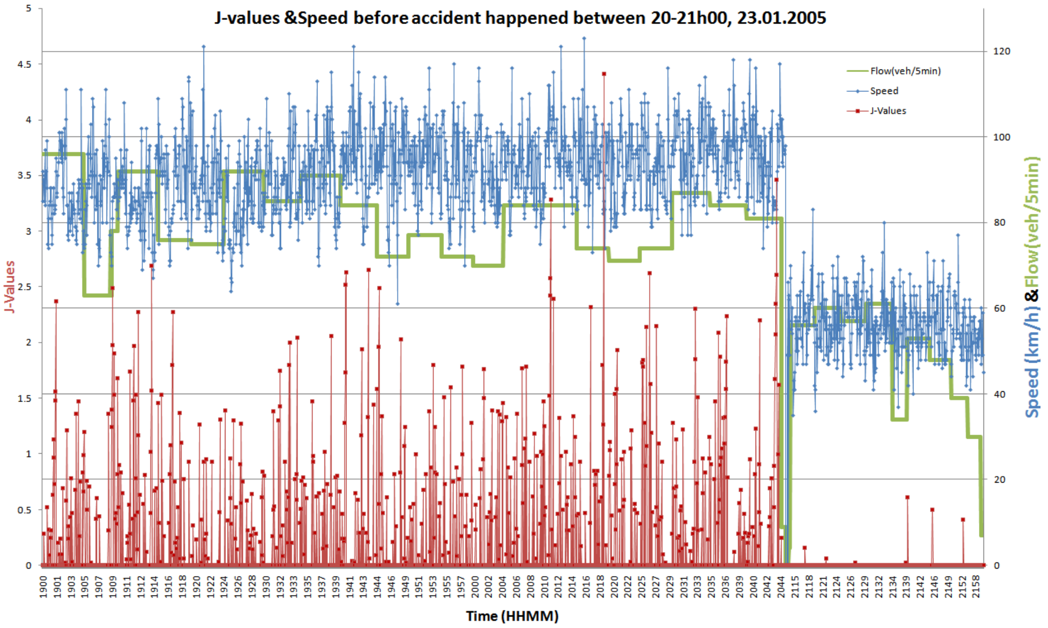


Fig 3.22 PBTR evolution on the slow lane

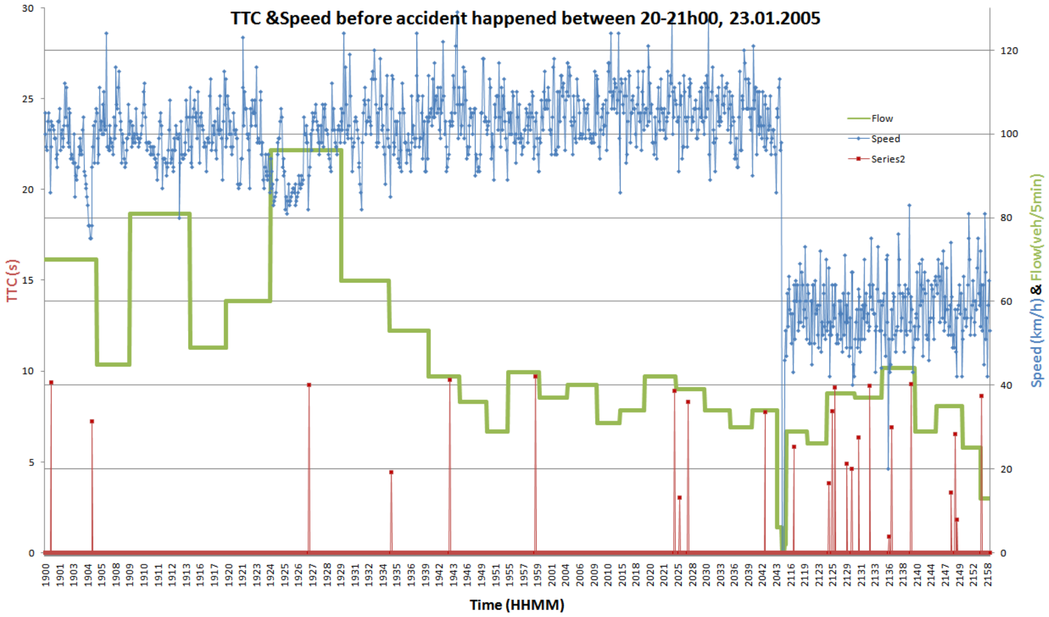


Fig 3.23 TTC evolution on the fast lane

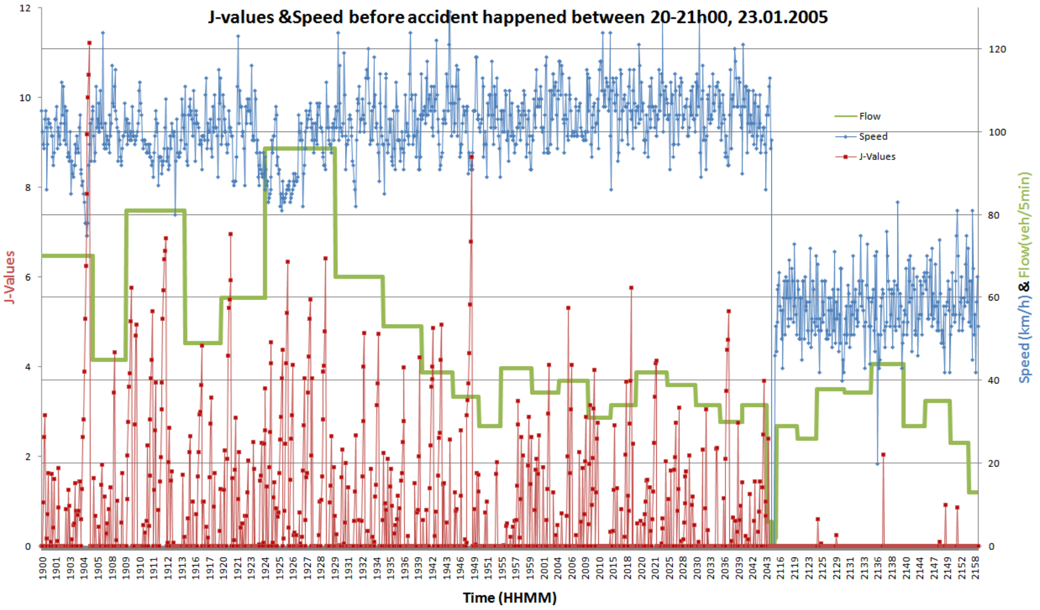


Fig 3.24 PBTR evolution on the fast lane

In Fig 3.25 and Fig 3.27, TTC distributions on the slow lane and on the fast lane are presented. The two right most columns contain information about the index of 5-minute interval and the flow during that interval. One day contains 24 hours which are 1440 minutes, i.e. 288 5-minute intervals. If the period from 00:00 to 00:05 is the first 5-minute interval of a day with an index of 0, the 5-minute interval from 20:55-21:00 has the index of 251. The other columns on the left represent the percentages of vehicles during each 5-minute interval having TTC values represented by the columns.

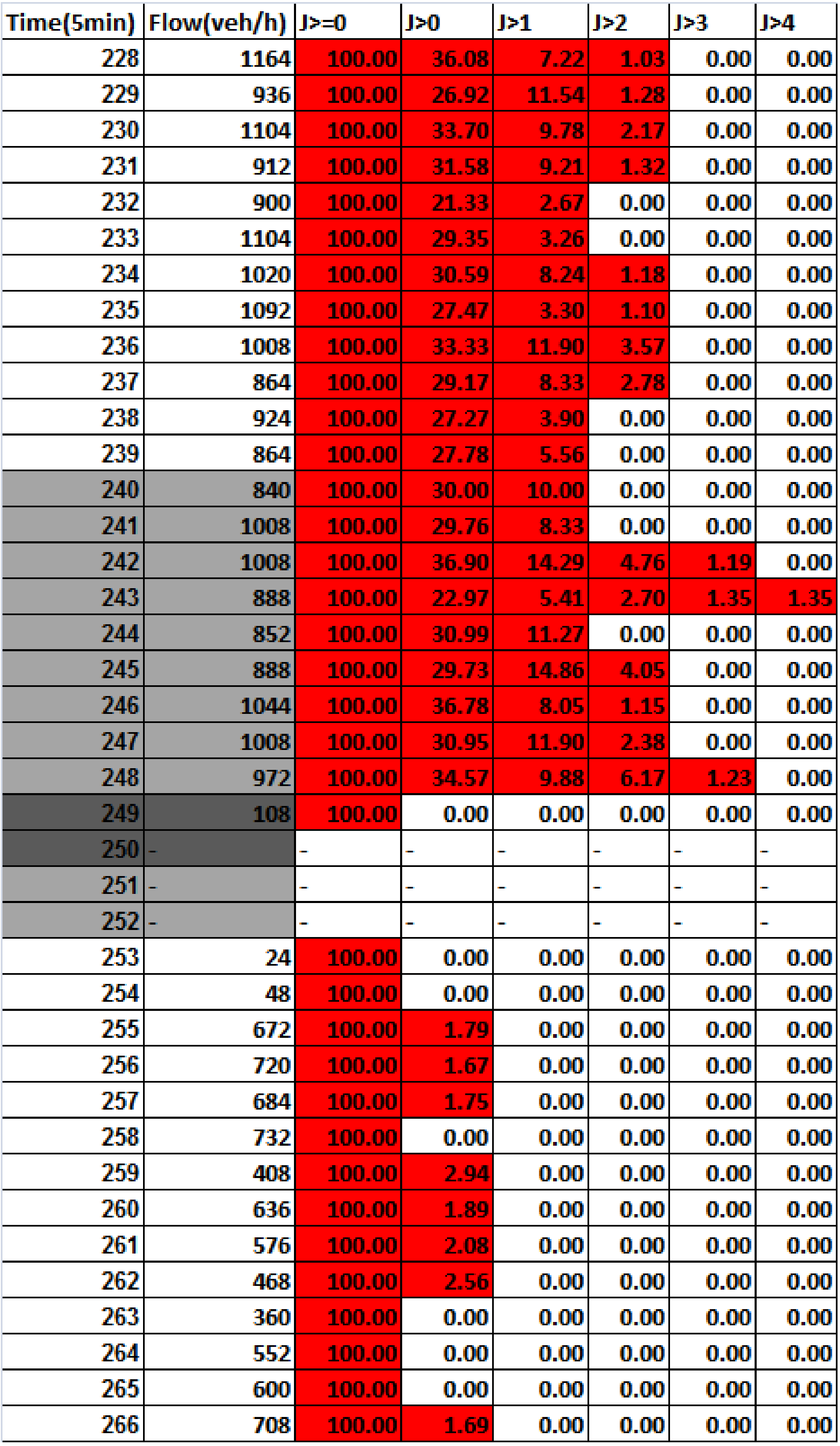
The traffic situation just before the crash is the 5-minute interval having index of 248. On the slow lane, the lowest TTC value during this interval is from 5.0 to 5.5seconds which is not as low as the lowest TTC value during the interval 243. Besides, by comparing the percentage of vehicles having TTC smaller than 7seconds during these two 5-minute intervals, the percentage is higher during the interval 248. This suggests that risk indicators may have better performance if not only their bound values are considered but also the percentages of vehicles that can approach those bound values.

The same conclusion can be taken from PBTR distributions in Fig 3.26 and Fig 3.28.

Fig 3.25 TTC distribution on the slow lane

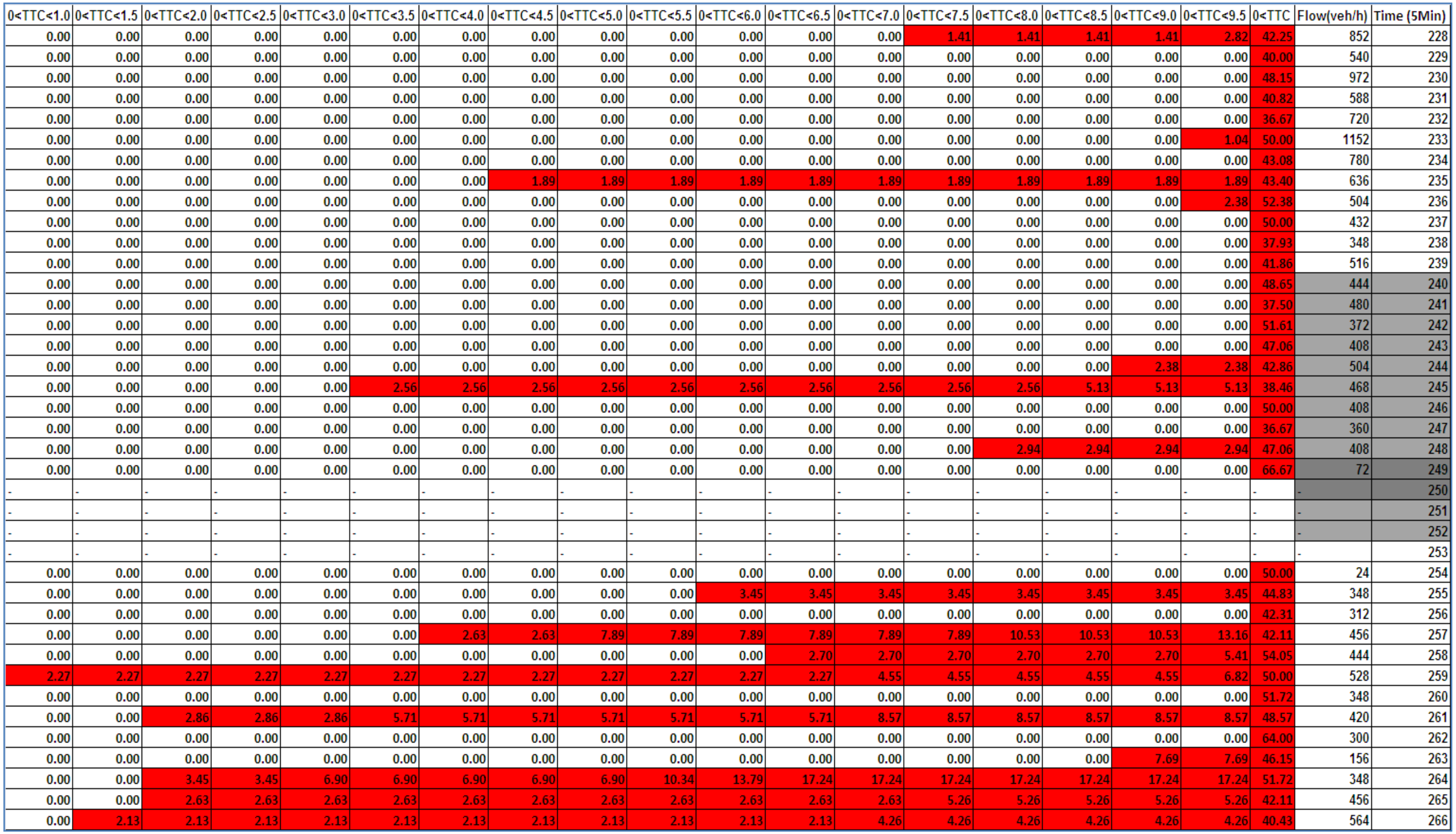
0<TTC<2.5	0<TTC<3.0	0<TTC<3.5	0<TTC<4.0	0<TTC<4.5	0<TTC<5.0	0<TTC<5.5	0<TTC<6.0	0<TTC<6.5	0<TTC<7.0	0<TTC<7.5	0<TTC<8.0	0<TTC<8.5	0<TTC<9.0	0<TTC<9.5	0<TTC	Flow(veh/h)	Time (5Min)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.03	1.03	1.03	45.36	1164	228
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.28	1.28	48.72	936	229
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.09	47.83	1104	230
0.00	0.00	0.00	0.00	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	46.05	912	231
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.67	900	232
0.00	0.00	0.00	0.00	0.00	0.00	2.17	2.17	2.17	2.17	2.17	2.17	2.17	2.17	3.26	48.91	1104	233
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.18	52.94	1020	234
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.10	2.20	47.25	1092	235
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.19	1.19	1.19	44.05	1008	236
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.39	1.39	1.39	44.44	864	237
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	49.35	924	238
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.39	1.39	2.78	2.78	2.78	2.78	51.39	864	239
0.00	0.00	0.00	0.00	0.00	0.00	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.43	55.71	840	240
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.19	1.19	1.19	1.19	1.19	1.19	1.19	50.00	1008	241
0.00	0.00	0.00	0.00	0.00	1.19	1.19	1.19	1.19	1.19	1.19	1.19	1.19	1.19	1.19	39.29	1008	242
0.00	0.00	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35	2.70	2.70	4.05	4.05	45.95	888	243
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.41	1.41	1.41	47.89	852	244
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.35	1.35	1.35	1.35	1.35	1.35	2.70	40.54	888	245
0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.15	1.15	1.15	1.15	1.15	1.15	2.30	2.30	42.53	1044	246
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.19	1.19	1.19	1.19	40.48	1008	247
0.00	0.00	0.00	0.00	0.00	0.00	1.23	1.23	1.23	2.47	2.47	2.47	2.47	3.70	3.70	50.62	972	248
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	44.44	108	249
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	250
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	251
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	252
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	24	253
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	48	254
0.00	0.00	0.00	1.79	1.79	1.79	1.79	1.79	1.79	1.79	1.79	1.79	1.79	1.79	1.79	53.57	672	255
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.67	1.67	1.67	3.33	3.33	3.33	51.67	720	256
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.75	1.75	45.61	684	257
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.64	1.64	50.82	732	258
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.94	2.94	2.94	2.94	2.94	2.94	47.06	408	259
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.89	50.94	636	260
0.00	0.00	0.00	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	39.58	576	261
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.56	48.72	468	262
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	360	263
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.17	2.17	2.17	52.17	552	264
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	46.00	600	265
1.69	1.69	1.69	1.69	1.69	3.39	3.39	3.39	3.39	3.39	3.39	3.39	5.08	5.08	6.78	44.07	708	266

Fig 3.26 PBTR distribution on the slow lane



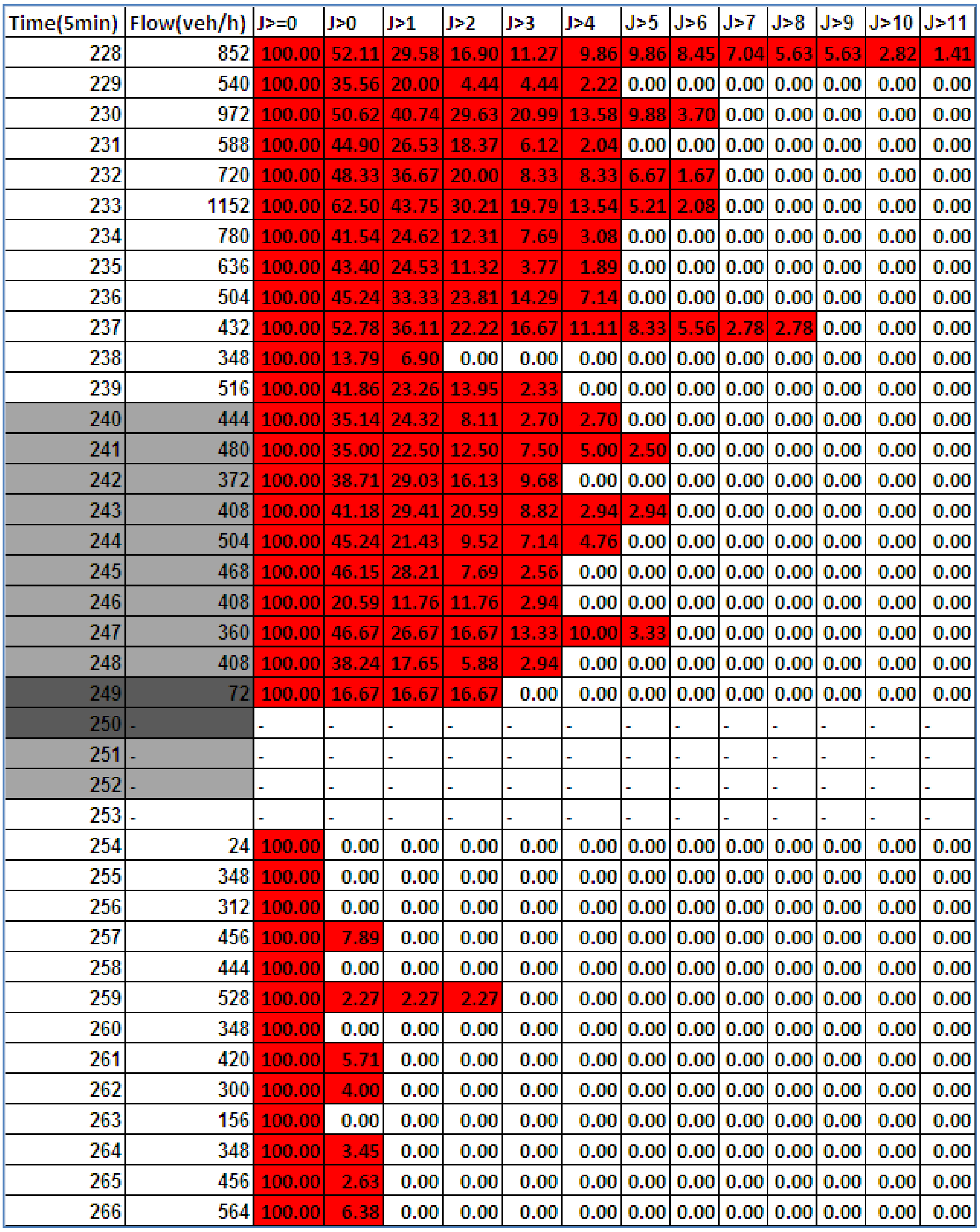
Time(5min)	Flow(veh/h)	J>=0	J>0	J>1	J>2	J>3	J>4
228	1164	100.00	36.08	7.22	1.03	0.00	0.00
229	936	100.00	26.92	11.54	1.28	0.00	0.00
230	1104	100.00	33.70	9.78	2.17	0.00	0.00
231	912	100.00	31.58	9.21	1.32	0.00	0.00
232	900	100.00	21.33	2.67	0.00	0.00	0.00
233	1104	100.00	29.35	3.26	0.00	0.00	0.00
234	1020	100.00	30.59	8.24	1.18	0.00	0.00
235	1092	100.00	27.47	3.30	1.10	0.00	0.00
236	1008	100.00	33.33	11.90	3.57	0.00	0.00
237	864	100.00	29.17	8.33	2.78	0.00	0.00
238	924	100.00	27.27	3.90	0.00	0.00	0.00
239	864	100.00	27.78	5.56	0.00	0.00	0.00
240	840	100.00	30.00	10.00	0.00	0.00	0.00
241	1008	100.00	29.76	8.33	0.00	0.00	0.00
242	1008	100.00	36.90	14.29	4.76	1.19	0.00
243	888	100.00	22.97	5.41	2.70	1.35	1.35
244	852	100.00	30.99	11.27	0.00	0.00	0.00
245	888	100.00	29.73	14.86	4.05	0.00	0.00
246	1044	100.00	36.78	8.05	1.15	0.00	0.00
247	1008	100.00	30.95	11.90	2.38	0.00	0.00
248	972	100.00	34.57	9.88	6.17	1.23	0.00
249	108	100.00	0.00	0.00	0.00	0.00	0.00
250	-	-	-	-	-	-	-
251	-	-	-	-	-	-	-
252	-	-	-	-	-	-	-
253	24	100.00	0.00	0.00	0.00	0.00	0.00
254	48	100.00	0.00	0.00	0.00	0.00	0.00
255	672	100.00	1.79	0.00	0.00	0.00	0.00
256	720	100.00	1.67	0.00	0.00	0.00	0.00
257	684	100.00	1.75	0.00	0.00	0.00	0.00
258	732	100.00	0.00	0.00	0.00	0.00	0.00
259	408	100.00	2.94	0.00	0.00	0.00	0.00
260	636	100.00	1.89	0.00	0.00	0.00	0.00
261	576	100.00	2.08	0.00	0.00	0.00	0.00
262	468	100.00	2.56	0.00	0.00	0.00	0.00
263	360	100.00	0.00	0.00	0.00	0.00	0.00
264	552	100.00	0.00	0.00	0.00	0.00	0.00
265	600	100.00	0.00	0.00	0.00	0.00	0.00
266	708	100.00	1.69	0.00	0.00	0.00	0.00

Fig 3.27 TTC distribution on the fast lane.



0<TTC<1.0	0<TTC<1.5	0<TTC<2.0	0<TTC<2.5	0<TTC<3.0	0<TTC<3.5	0<TTC<4.0	0<TTC<4.5	0<TTC<5.0	0<TTC<5.5	0<TTC<6.0	0<TTC<6.5	0<TTC<7.0	0<TTC<7.5	0<TTC<8.0	0<TTC<8.5	0<TTC<9.0	0<TTC<9.5	0<TTC	Flow(veh/h)	Time (5Min)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.41	1.41	1.41	1.41	2.82	42.25	852	228
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	40.00	540	229
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	48.15	972	230
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	40.82	588	231
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	36.67	720	232
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.04	50.00	1152	233
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	43.08	780	234
0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	43.40	636	235
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.38	52.38	504	236
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	432	237
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	37.93	348	238
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	41.86	516	239
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	48.65	444	240
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	37.50	480	241
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	51.61	372	242
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	47.06	408	243
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.38	2.38	42.86	504	244
0.00	0.00	0.00	0.00	0.00	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56	2.56	5.13	5.13	5.13	38.46	468	245
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	408	246
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	36.67	360	247
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.94	2.94	2.94	2.94	47.06	408	248
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	66.67	72	249
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	250
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	251
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	252
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	253
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.00	24	254
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.45	3.45	3.45	3.45	3.45	3.45	3.45	3.45	44.83	348	255
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	42.31	312	256
0.00	0.00	0.00	0.00	0.00	0.00	2.63	2.63	7.89	7.89	7.89	7.89	7.89	7.89	10.53	10.53	10.53	13.16	42.11	456	257
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.70	2.70	2.70	2.70	2.70	2.70	5.41	54.05	444	258
2.27	2.27	2.27	2.27	2.27	2.27	2.27	2.27	2.27	2.27	2.27	2.27	4.55	4.55	4.55	4.55	4.55	6.82	50.00	528	259
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	51.72	348	260
0.00	0.00	2.86	2.86	2.86	5.71	5.71	5.71	5.71	5.71	5.71	5.71	8.57	8.57	8.57	8.57	8.57	8.57	48.57	420	261
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	64.00	300	262
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	7.69	7.69	46.15	156	263
0.00	0.00	3.45	3.45	6.90	6.90	6.90	6.90	6.90	10.34	13.79	17.24	17.24	17.24	17.24	17.24	17.24	17.24	51.72	348	264
0.00	0.00	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63	5.26	5.26	5.26	5.26	5.26	42.11	456	265
0.00	2.13	2.13	2.13	2.13	2.13	2.13	2.13	2.13	2.13	2.13	2.13	4.26	4.26	4.26	4.26	4.26	4.26	40.43	564	266

Fig 3.28 PBTR distribution on the fast lane.



Time(5min)	Flow(veh/h)	J>=0	J>0	J>1	J>2	J>3	J>4	J>5	J>6	J>7	J>8	J>9	J>10	J>11
228	852	100.00	52.11	29.58	16.90	11.27	9.86	9.86	8.45	7.04	5.63	5.63	2.82	1.41
229	540	100.00	35.56	20.00	4.44	4.44	2.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00
230	972	100.00	50.62	40.74	29.63	20.99	13.58	9.88	3.70	0.00	0.00	0.00	0.00	0.00
231	588	100.00	44.90	26.53	18.37	6.12	2.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00
232	720	100.00	48.33	36.67	20.00	8.33	8.33	6.67	1.67	0.00	0.00	0.00	0.00	0.00
233	1152	100.00	62.50	43.75	30.21	19.79	13.54	5.21	2.08	0.00	0.00	0.00	0.00	0.00
234	780	100.00	41.54	24.62	12.31	7.69	3.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00
235	636	100.00	43.40	24.53	11.32	3.77	1.89	0.00	0.00	0.00	0.00	0.00	0.00	0.00
236	504	100.00	45.24	33.33	23.81	14.29	7.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00
237	432	100.00	52.78	36.11	22.22	16.67	11.11	8.33	5.56	2.78	2.78	0.00	0.00	0.00
238	348	100.00	13.79	6.90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
239	516	100.00	41.86	23.26	13.95	2.33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
240	444	100.00	35.14	24.32	8.11	2.70	2.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00
241	480	100.00	35.00	22.50	12.50	7.50	5.00	2.50	0.00	0.00	0.00	0.00	0.00	0.00
242	372	100.00	38.71	29.03	16.13	9.68	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
243	408	100.00	41.18	29.41	20.59	8.82	2.94	2.94	0.00	0.00	0.00	0.00	0.00	0.00
244	504	100.00	45.24	21.43	9.52	7.14	4.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00
245	468	100.00	46.15	28.21	7.69	2.56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
246	408	100.00	20.59	11.76	11.76	2.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
247	360	100.00	46.67	26.67	16.67	13.33	10.00	3.33	0.00	0.00	0.00	0.00	0.00	0.00
248	408	100.00	38.24	17.65	5.88	2.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
249	72	100.00	16.67	16.67	16.67	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
250	-	-	-	-	-	-	-	-	-	-	-	-	-	-
251	-	-	-	-	-	-	-	-	-	-	-	-	-	-
252	-	-	-	-	-	-	-	-	-	-	-	-	-	-
253	-	-	-	-	-	-	-	-	-	-	-	-	-	-
254	24	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
255	348	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
256	312	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
257	456	100.00	7.89	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
258	444	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
259	528	100.00	2.27	2.27	2.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
260	348	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
261	420	100.00	5.71	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
262	300	100.00	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
263	156	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
264	348	100.00	3.45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
265	456	100.00	2.63	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
266	564	100.00	6.38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

In [Pham, M.-H. et al., 2008], four crashes are used for testing six indicators. The four crashes are near traffic detector 149 and described in Fig 3.29. The risk criteria are defined based on the desired alarm rates and then applied to traffic conditions before the four crashes.

A guideline about how to define the risk criteria is also presented in [Pham, M.-H. et al., 2008]. This guideline can be applied in both cases where the crash database is available or not. If the crash database is available and crash time can be observable in traffic data, pre-crash conditions are used to validate the risk criteria. If the crash database is not available or the number of usable crashes is limited, the desired alarm rate can be used to propose the risk criteria. In that case, the risk criteria are not validated.

Fig 3.29 Description of four crashes

ID + Site number (detector code)	Date (yyyymmdd)	Hour period (hhmm)	Light (**D**ay / **N**ight)	Weather (**Dr**y / **R**ain)	Road surface(**D**ry / **W**et)	Distance from traffic detector (m)	Position (**U**p- / **D**ownstream) from detector	Direction (+ for lanes 1 and 2 / -for lanes 3 and 4)
1 149	20030308	1600-1700	D	D	D	696	D	+
2 149	20030310	0900-1000	D	D	D	44	D	-
3 149	20040116	1900-2000	N	R	W	59	U	+
4 149	20040922	1700-1800	D	D	D	224	U	+

3.4 Conclusions

In this section, the sensitivity of safety indicators is tested against the traffic flow, the meteorological conditions and the lanes of the road.

PBTR, IBTR, and PICUD with meteorological elements in their formula are sensitive to meteorological conditions. Most risk indicators except TTC are sensitive to flow ranges. The risk indicators are also sensitive to the lanes of the road. The sensitivity test shows that PICUD is not reliable because it indicates that the traffic under fine meteorological conditions is more unsafe than under rain conditions. Therefore, PICUD should not be used further in this study.

The best way to test the performance of risk indicators is to check the evolution of risk indicators during the traffic conditions just before the crash when it is sure that there is crash risk before the crash. With the crash cases tested, it is recommended that when considering if a traffic situation is risky or not, not only bound values of risk indicators but also the percentages of vehicles are essential.

3.5 Risk criteria

The risk criteria are a set of rules based on risk indicators used for evaluating risk status of a traffic situation. There are two thresholds for each risk indicator that need to be defined according to results from section 3.3.4. The first threshold is the individual level at which a vehicle is considered as having crash risk. For example, [Hayward, 1972] suggested that 4 seconds was the TTC level at which a vehicle can have crash risk. Today vehicles are equipped with good security devices and hence, TTC level might have changed. If TTC threshold is still fixed at 4s there will be 0.2% out of all vehicles which are risky on the slow lane according to TTC distribution.

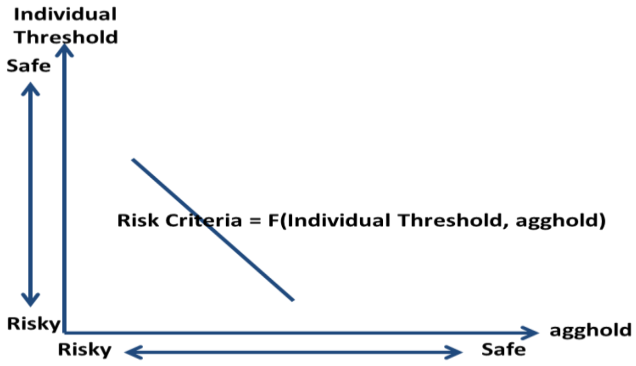


Fig 3.30 Risk criteria function.

The second threshold of a risk indicator is the maximum percentage of vehicles during a traffic situation having value out of the safe range of the risk indicator. This threshold is based on multiple vehicles during an aggregation interval and thus the aggregate threshold aims at determining if the traffic situation is risky or not.

There several ways to choose the individual thresholds and the aggregate thresholds.

If the individual TTC threshold is too small, i.e. it is rare to observe a vehicle having this TTC value, high TTC aggregate threshold will consider all the traffic situations as “safe”. On the contrary, if the individual TTC threshold is too large, a low TTC aggregate threshold will classify all the traffic situations as “risky”. Determining the thresholds could be based on the trade-off between the suitable percentage of correctly alarmed traffic situations and a percentage of false alarms. For example, if a period of 5 minutes is used as a traffic situation, there are 288 situations in a day. From the practical point of view, the percentage of alarmed periods could be decided so that drivers would remain confident with the alarm. Fig 3.30 shows the risk criteria line, which is a function of the individual threshold and the aggregate threshold if this method of defining the thresholds is applied.

The threshold selection in this research is undertaken for each flow range and meteorological conditions as the results of the sensitivity analysis of risk indicators in the previous section. Taking into account the risk criteria function, Fig 3.31 shows the defined thresholds for the high flow range (>1500vph) under the fine weather conditions on the slow lane. If 0.1% of vehicles are considered as “risky” according to each risk indicator and if 0.1% of traffic situations are classified “risky”, the thresholds should be defined as in Fig 3.31.

Fig 3.31 Risk criteria for high flow range (>1500vph) under fine weather conditions on the slow lane

Risk Indicators	Individual Thresholds	Aggregate threshold (%)
IBTR (G-values)	5	2.00
PBTR (J-values)	13	3.92
TTC (s)	1.5	2.5
SOSL (km/h)	150	0.78
Headway (s)	0.5	14.50

However, there are other methods to set the thresholds. The risk indicators by themselves imply somehow crash risk. Each risk indicator has its own detection rate, according to its performance, which must be determined first. With the crash records and normal traffic situations, the false alarm rate could be set. Thresholds will be set based on the detection rate and the false alarm rate.

Consider the reverse process: determining the false alarm rate from non-crash traffic situations. For each crash case, there are 4 non-crash situations selected for the same time of day, on the same day of the week, under the same weather conditions. Fig 3.32 shows the 16 selected normal situations for each crash.

Fig 3.32 Selection of non-crash situations

Pre-crash situations	Selected dates with normal situations
20030308 1500-1700	20030301, 20030315, 20040228, 20040313
20030310 0800-1000	20030303, 20030317, 20040301, 20040315
20040116 1800-2000	20040109, 20040123, 20050114, 20050121,
20040922 1600-1800	20040915, 20040929, 20050921, 20050928,

The same procedure is undertaken for the other risk indicators and on the 15 other selected dates from Fig 3.32 leading to a false alarm rate based on 16 observations. Fig 3.33 summarizes the detection rates and false alarm rates given by each individual safety indicator - ISI. (4 observations): for instant, IBTR captured crash risk in three cases out of four (i.e. 75%).

Fig 3.33 Detection rates and false alarm rates for each risk indicator with 4 *crashes and 16 selected normal situations.*

	IBTR	PBTR	TTC	SOSL	Headway	PICUD
Detection rate (%)	75.0	75.0	100.0	75.0	50.0	75.0
False alarm rate (%)	56.2	50.0	75.0	62.5	43.7	50.0

As shown in Fig 3.33, the false alarm rates by risk indicators are high, which is caused by criteria defined. Moreover, the aggregate thresholds are not taken under consideration in this case. If the risk criteria are more carefully defined and if the aggregate thresholds are considered, the detection rate should increase and the false alarm rate should decrease. This is to say that with given detection rates and false alarm rates, it is possible to find the individual thresholds and the aggregate thresholds satisfying high detection rates and low false alarm rates.

Finally, practical aspects could also be taken under consideration, so that the alarm would not annoy the drivers. Each ISI has its own risk criteria and the combination of the risk indicators could make many alarms during a day. In case there is a dense traffic detector network installed along motorways, the thresholds should also be determined based on the overview performance of risk indicators at each detector.

4 Effect of preventive measures on traffic safety

4.1 Introduction

The analyses in chapter 2 and 3 are undertaken based on data collected from traffic detectors installed on Swiss motorways in Vaud canton. The results obtained from crash analyses and sensitivity analyses of risk indicators can provide an insight into traffic situations leading to crashes. The obtained results are used in the next step to identify crash-prone conditions in real-time and then take necessary operational measures to prevent the occurrence of the potential crashes. However, before it can be applied in the real life, experiments on simulation by computer need to be undertaken first to evaluate the outcome of the solution.

4.1.1 Objectives

The simulation model in this project is built based on a real road section on the Swiss motorway A9. The traffic conditions similar to the reality for one direction of traffic are simulated based on observed input data. Fig 4.1 shows the work flow of the simulation part in this project. After the first simulation model is built, it is calibrated to best represent the real world. It is important to note that the items to be calibrated should match the objectives of the simulation. Traffic simulation cannot represent all aspects of the real world, the best simulation settings are found so that the simulation model fits the requirement of this study. During the calibration process, the limits of simulator are also recognized, which would partially help explaining the results at later stage. The calibrated model will serve as a tool for studying traffic safety with the implementation of the proactive preventive measures. The results from the simulation model are evaluated to determine the effectiveness of the preventive measures tested.

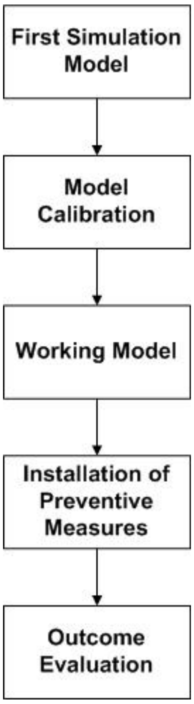


Fig 4.1 Simulation diagram

The main objective of the simulation model is to allow installing traffic detectors to capture traffic information and monitoring traffic risk via the preventive measures. By implementing such a simulation model, the outcome of the measures can be evaluated.

In the other word, the simulation model provides a platform for applying the risk criteria obtained from chapter 3.5. The risk criteria are integrated into a management plan together with a strategy for implementing preventive measures. The results - including the improvement of traffic safety and the traffic cost - of the simulation model with and without a management plan will be compared to evaluate the effectiveness of the preventive measures installed.

The strategy for implementing preventive measures and an implementation example are presented in this chapter.

4.1.2 Structure of this chapter

In the next section of this chapter, the introduction of the first simulation model is presented. The model is then calibrated and the calibration process is presented in section 4.3. Section 4.4 introduces the integration of the management plan into the model. Section 4.5 presents the outcomes of the integration. Finally, conclusions and further discussions are presented in the last section.

4.2 Simulation model

4.2.1 Simulation site

To be consistent with the results obtained from previous chapters, the site simulated is selected among study sites in chapter 3. There are several criteria for the selection:

- The section must be on a Swiss motorway. The goal of this research is to study the effect of the management plan on traffic safety. The simulation should, thus, be as similar to the real traffic configuration as possible.
- The section should be a road section where one of traffic detectors studied in Section 2 is installed. With traffic data collected from those traffic detectors, it is possible to compare the data with simulated data.

One section of a Swiss motorway, the motorway A9, was selected as simulation site, which is the section from Chexbres to Crissier where traffic detector 116 is available. Due to the independence of two traffic direction, only one direction is simulated: that is the direction from Chexbres to Crissier.

Fig 4.2 Road section profile	
Total length	20km
Number of lanes	2 or 3
Direction	Chèxbre-Crissier
Number of entries	4
Number of exits	4
Number of tunnels	4
Rest area	1
Number of traffic detectors	1

The statistics about the road section is shown in Fig 4.2. On this road section, there are 15km of two-lane road section and 5km of three lane section. The rest area in the section is not considered as a pair of an entry and an exit; and for this reason, the rest area is not simulated. A part of the road section lies on a mountainous area and therefore, there are 4 tunnels on the road section. The general speed limit on Swiss motorway is 120km/h and the speed limit of the tunnel sections of the motorway is 100km/h. One traffic detector is used to collect the real traffic data, i.e. the detector 116.

Fig 4.3 Selected section and traffic direction on motorway A9

Fig 4.3 shows the position of the road section on the motorway A9 taken from Google Maps and Fig 4.4 presents the road section built in the simulation model.

Fig 4.4 Simulation section

In the simulation model, the number of entries and exits as well as the number of lanes at each section are respected excluding the rest area. Road sections in the simulation model are constructed by overlapping them with satellite photos from Google images so that road curvatures are also respected.

4.2.2 Input data

The geographical structure of the road section is just the first step in building the simulation model. More data need to be collected and introduced to the model to build the simulation model as illustrated in Fig 4.4. Field data used can be collected from A9 for calibrating the simulation model and they include:

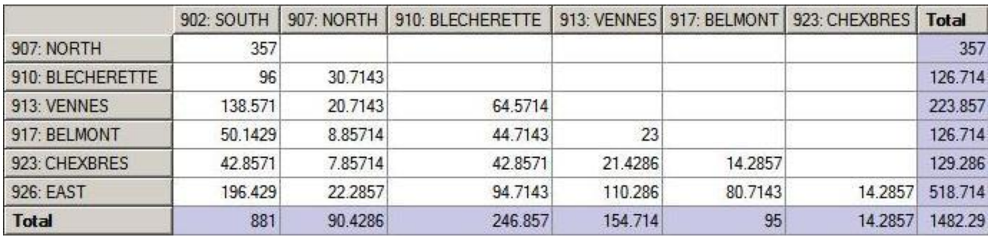
- Traffic data from traffic detectors installed on A9.
- Profile data of the road section such as the speed limit, the section length, and the curves and vehicle statistics such as vehicle length, maximum desired speed.
- The Origin-Destination matrix (OD matrix) of the road section calibrated from other studies.

The traffic data is introduced in chapter 2 with traffic detector 116. A part of the road section profile data is discussed in section 4.2.1. The information about the other data is summarized in Fig 4.5.

Fig 4.5 Values set for parameters at the first model

Parameters	Values
Speed limits	Set according to A9 road profile
Section curvatures	Set according to A9 road profile
Section length	Set according to A9 road profile
Vehicle length (m)	Average: 4.5; Deviation: 0.37; Min: 3.15; Max: 5.19
Maximum desired speed (Km/h)	Average: 110; Deviation: 10.6; Min: 70; Max: 150
OD Matrix	Inherited from previous projects [Bert, E. et al., 2006] and [Leyvraz, J. et al., 2006]
Traffic composition	100% cars
Traffic demand	Set according to data collected from detector 116

Vehicle length is calculated based on the real vehicle length distribution captured by traffic detector 116. In this simulation, heavy vehicles are not taken into account. The maximum desired speed of vehicles is also calculated based on the speed preferred by drivers under free flow conditions from the real traffic data.



	902: SOUTH	907: NORTH	910: BLECHERETTE	913: VENNES	917: BELMONT	923: CHEXBRES	**Total**
907: NORTH	357						357
910: BLECHERETTE	96	30.7143					126.714
913: VENNES	138.571	20.7143	64.5714				223.857
917: BELMONT	50.1429	8.85714	44.7143	23			126.714
923: CHEXBRES	42.8571	7.85714	42.8571	21.4286	14.2857		129.286
926: EAST	196.429	22.2857	94.7143	110.286	80.7143	14.2857	518.714
Total	881	90.4286	246.857	154.714	95	14.2857	1482.29

Fig 4.6 OD matrix used in this study

The OD matrix used in this study is inherited from previous research projects in EPFL. Fig 4.6 shows that OD matrix for the period from 07:00 to 08:00 AM on a week day. The OD matrices for the other periods of the day are not available. More information about this project can be found in [Bert, E. et al., 2006] and [Leyvraz, J. et al., 2006].

In Fig 4.6, the four pairs of entries/exits are, namely, Blécherette, Vennes, Belmont, and Chexbres. The traffic origins or destinations such as North, South, and East are the points where the road section is cut from the whole motorway A9.

The simulation model will be run for duration of 24 hours. It would be preferable if hourly OD matrices were available for each of 24 hours. In this study, the only available traffic demand from 07:00 to 08:00 is scaled to other hourly durations of the day. The scale is based on the average traffic volume obtained from the real traffic data provided by the detector 116. Fig 4.7 shows the evolution of that scale during one day.

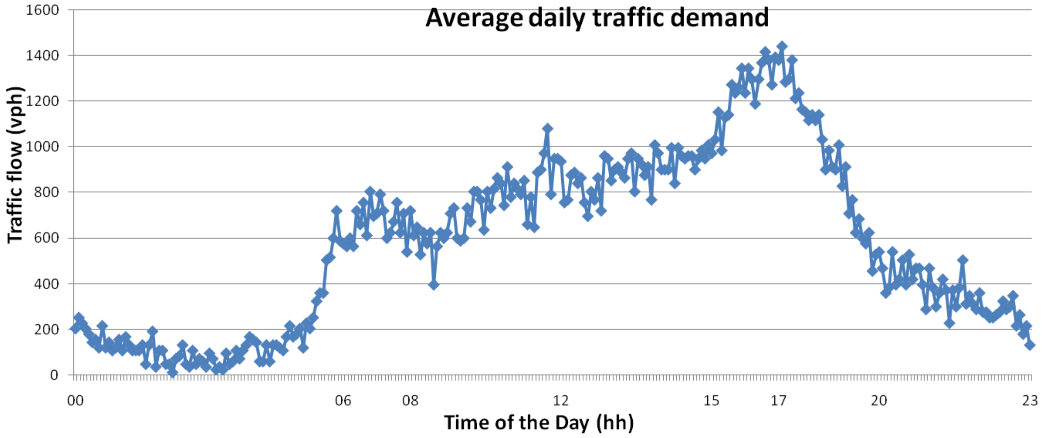


Fig 4.7 Average 5 minute traffic demand of a day

With the input data set collected, the simulation model is built. This simulation model needs to be calibrated before it can be employed as a sample of the real world.

4.3 Model calibration

4.3.1 Calibration strategy

According to the calibration guideline in [Dowling, R. et al, 2004], the model calibration is a loop of comparisons and adjustments until the final acceptable model is reached. Then calibration is considered as a process of review and adjustment of numerous model parameters that can impact the simulation results and can have mutual influence. The calibration process thus can turn out to be a never-ending circular process. For this reason, a calibration strategy is recommended in [Dowling, R. et al, 2004].

In this study, a calibration strategy is also proposed based on objectives of the simulation model before any calibration task is undertaken. As the selected simulation site is a motorway road section, certain guideline points in [Dowling, R. et al, 2004] about non-motorway roads are not applied.

The calibration strategy includes a sequence of calibration sub-tasks and the stop criteria for each sub-task. For each sub-task, a list of model parameters is reviewed and adjusted so that their best value ranges are acknowledged. During the next sub-task, some of the parameters that have been reviewed and adjusted continue to be reviewed and adjusted in the acknowledged value ranges from previous sub-task. There are also new model parameters that are calibrated in the next step so that after the calibration process, all involved model parameters are refined to make the model ready for experiments.

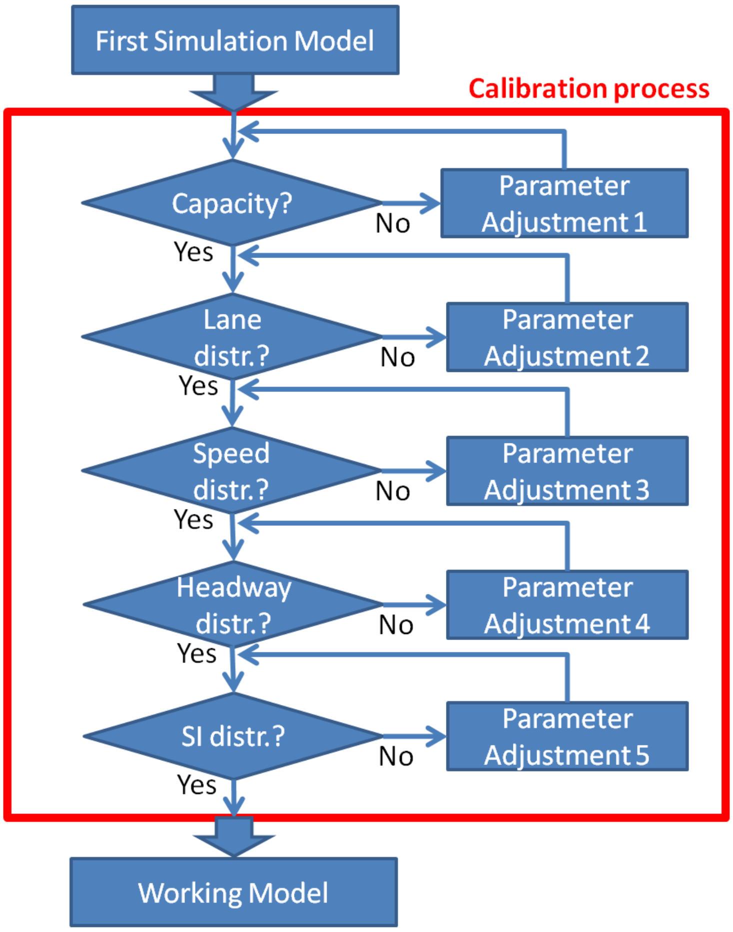


Fig 4.8 Model calibration process

To comply with the objectives of the simulation in this study as stated in section 4.1.2, the calibration process should be driven to highlight the aspect of traffic safety oriented study. Fig 4.8 shows the steps of the calibration process. The calibration for the capacity, the lane distribution, and the speed distribution should reflect the motorway modeling while the calibration for the headway distribution and the risk indicator distribution can determine if the model is reliable for the study about traffic safety.

In Fig 4.9, each parameter adjustment involves a set of model parameters. One model parameter can be tuned many times and it should be more refined in the later parameter adjustment compared to the earlier parameter adjustment.

The parameters that can be changed depend on the simulator used to construct the model. As AIMSUN 6.0 is used in this study, the parameters can be divided into 3 groups as suggested in AIMSUN manual: Global parameters, Road section parameters, and Vehicles parameters. Fig 4.9 shows the list of tuning parameters of each group and the steps in Fig 4.10 where parameters are changed.

The calibration process takes the first simulation model with all its default parameter values as the input and results in the working model which will be used in the later stage. AIMSUN 6.0 provides default values to all parameters listed in Fig 4.9. The calibration process starts with default values and ends with value ranges for each parameter. Values of the parameters are chosen inside the value ranges and fixed in the working model. This means that at later stages of this study, parameter values will not change.

Fig 4.9 Groups of tuning parameters

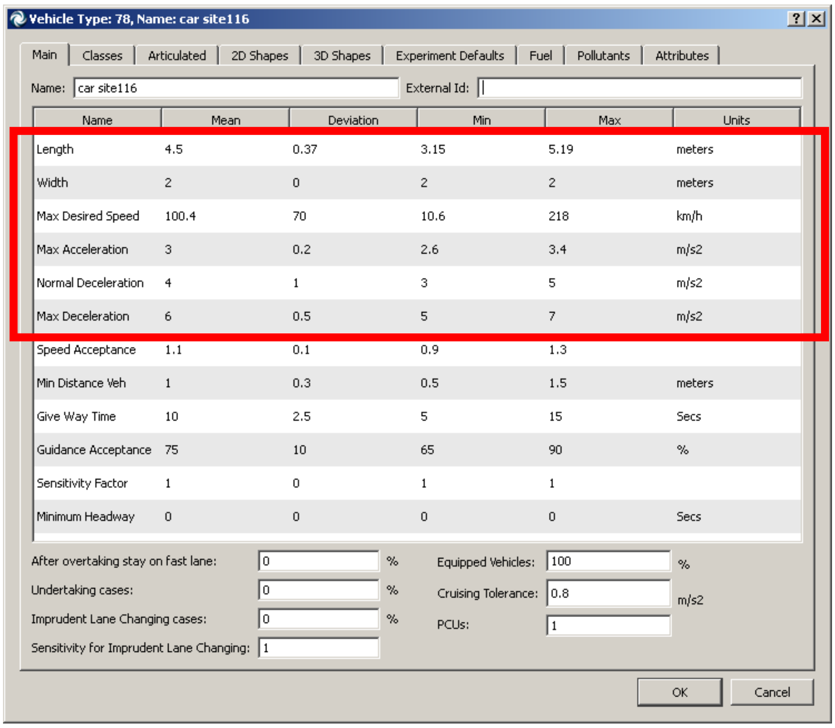
Parameter Groups	Parameters
Global Parameters	Reaction Time
	Reaction Time at Stop
	Queue up speed
	Queue leaving speed
	Percent overtake
	Percent recover
	Maximum speed difference
	Maximum on/off-ramp speed difference
Road Section Parameters	Speed Limit
	Visibility distance
Vehicles Parameters	Vehicle length
	Maximum desired speed
	Maximum acceleration
	Normal deceleration
	Maximum deceleration
	Minimum distance between vehicles
	Give way time

The calibration process in Fig 4.8 is a sequence of five iterating sub-tasks. As mentioned in AIMSUN 6.0 Manual and in [Dowling, R. et al., 2004], there is no simulation model that represents exactly the real world. Therefore, each of the sub-tasks should be iterated till a point where the model matches the best the real world. The stop criterion at each sub-task can then be defined to drive the model to an acceptable echo of the real world. Fig 4.10 below describes different stop criteria for every item that needs to be calibrated. More details about these criteria and results will be provided in the next sub-section.

Fig 4.10 Stop criteria for the calibration process

Items to calibrate	Stop criteria
Capacity	From 2300-2500vph per lane
Speed distribution	Described in section 4.3.4
Lane distribution	Described in section 4.3.5
Headway distribution	Described in section 4.3.6
SI distribution	Described in section 4.3.6

As AIMSUN 6.0 is used for this study, parameters that are tuneable are surrounded by read rectangles in Fig 4.11, Fig 4.12, Fig 4.13, and Fig 4.14. In Fig 4.14, the reaction time should always be smaller than the reaction time at a stop.



Name: car site116 External Id:

Name	Mean	Deviation	Min	Max	Units
Length	4.5	0.37	3.15	5.19	meters
Width	2	0	2	2	meters
Max Desired Speed	100.4	70	10.6	218	km/h
Max Acceleration	3	0.2	2.6	3.4	m/s2
Normal Deceleration	4	1	3	5	m/s2
Max Deceleration	6	0.5	5	7	m/s2
Speed Acceptance	1.1	0.1	0.9	1.3	
Min Distance Veh	1	0.3	0.5	1.5	meters
Give Way Time	10	2.5	5	15	Secs
Guidance Acceptance	75	10	65	90	%
Sensitivity Factor	1	0	1	1	
Minimum Headway	0	0	0	0	Secs

After overtaking stay on fast lane: 0 %

Undertaking cases: 0 %

Imprudent Lane Changing cases: 0 %

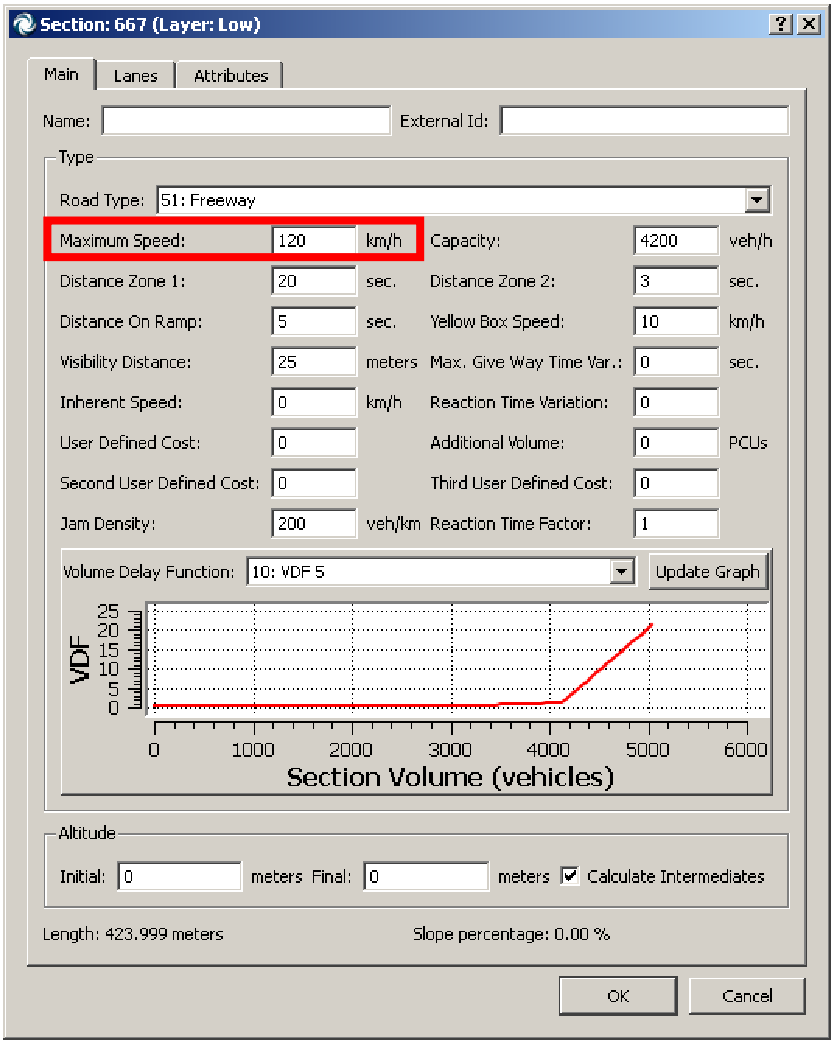
Sensitivity for Imprudent Lane Changing: 1

Equipped Vehicles: 100 %

Cruising Tolerance: 0.8 m/s2

PCUs: 1

Fig 4.11 Tunable vehicle parameters



Name: External Id:

Road Type: 51: Freeway

Maximum Speed: 120 km/h

Capacity: 4200 veh/h

Distance Zone 1: 20 sec.

Distance Zone 2: 3 sec.

Distance On Ramp: 5 sec.

Yellow Box Speed: 10 km/h

Visibility Distance: 25 meters

Max. Give Way Time Var.: 0 sec.

Inherent Speed: 0 km/h

Reaction Time Variation: 0

User Defined Cost: 0

Additional Volume: 0 PCUs

Second User Defined Cost: 0

Third User Defined Cost: 0

Jam Density: 200 veh/km

Reaction Time Factor: 1

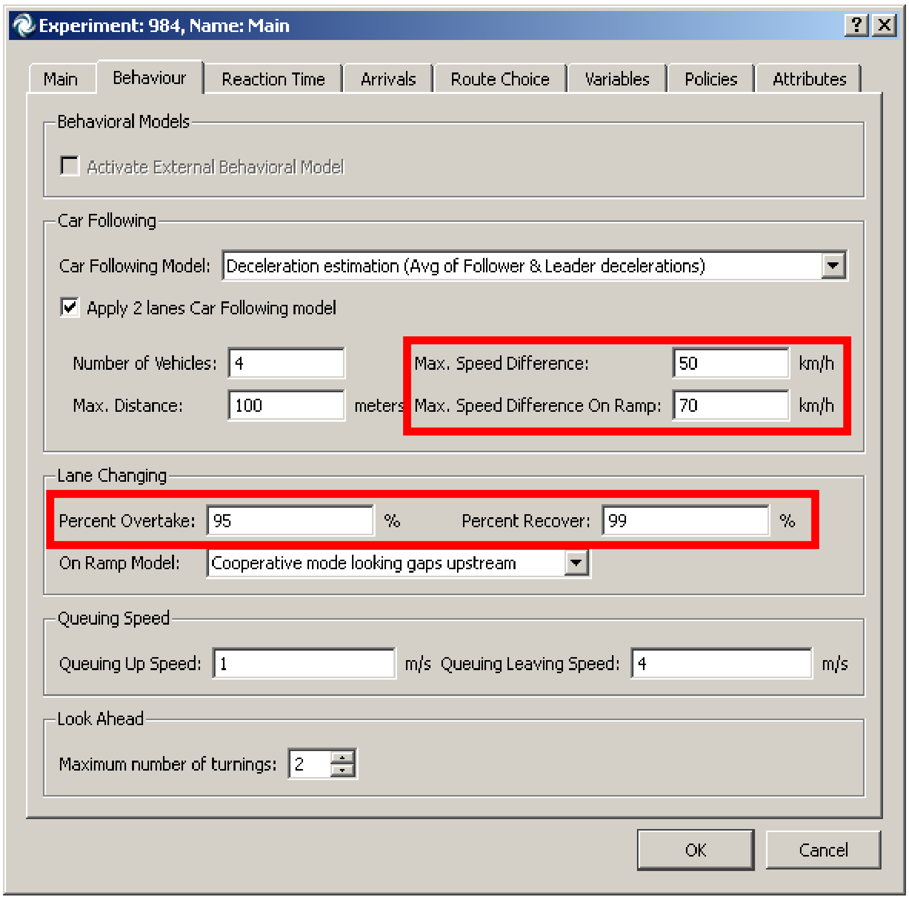
Volume Delay Function: 10: VDF 5

Initial: 0 meters Final: 0 meters ☑ Calculate Intermediates

Length: 423.999 meters

Slope percentage: 0.00 %

Fig 4.12 Tunable section parameters



Behavioral Models

☐ Activate External Behavioral Model

Car Following

Car Following Model: Deceleration estimation (Avg of Follower & Leader decelerations)

☑ Apply 2 lanes Car Following model

Number of Vehicles: 4

Max. Distance: 100 meters

Max. Speed Difference: 50 km/h

Max. Speed Difference On Ramp: 70 km/h

Lane Changing

Percent Overtake: 95 %

Percent Recover: 99 %

On Ramp Model: Cooperative mode looking gaps upstream

Queuing Speed

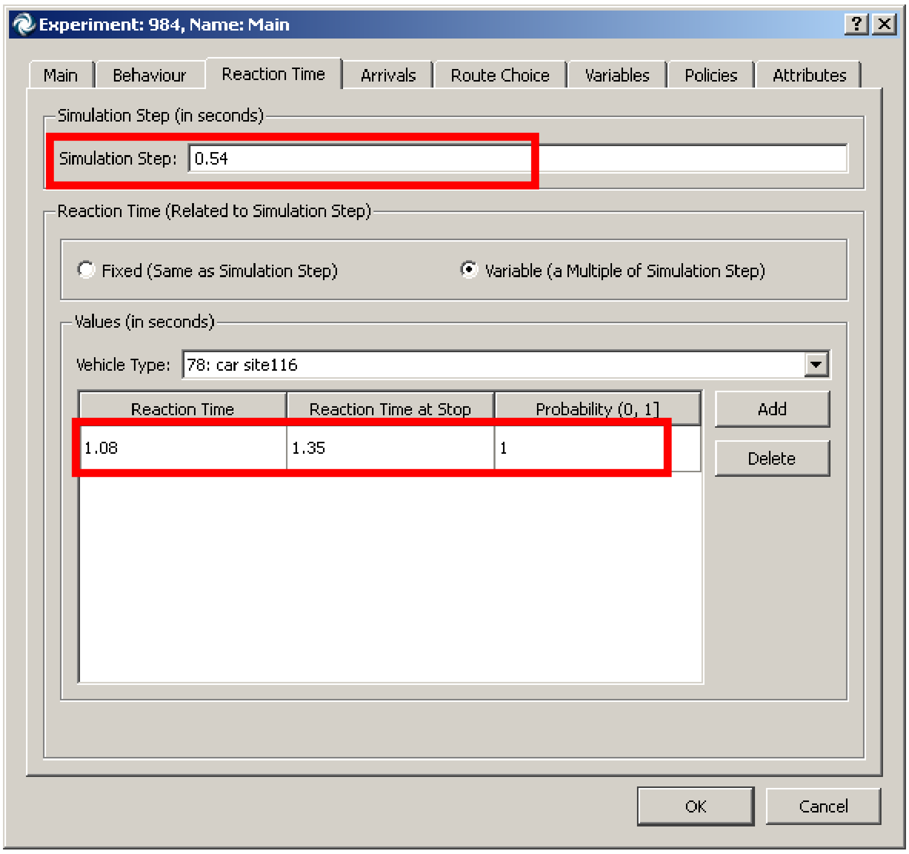
Queuing Up Speed: 1 m/s

Queuing Leaving Speed: 4 m/s

Look Ahead

Maximum number of turnings: 2

Fig 4.13 Tunable driving behavior parameters



Simulation Step (in seconds)

Simulation Step: 0.54

Reaction Time (Related to Simulation Step)

○ Fixed (Same as Simulation Step)

● Variable (a Multiple of Simulation Step)

Values (in seconds)

Vehicle Type: 78: car site116

Reaction Time	Reaction Time at Stop	Probability (0, 1]
1.08	1.35	1

Fig 4.14 Tuneable reaction time

4.3.2 Simulation model

This is the first simulation model constructed and it is the model before any calibration task as shown in Fig 4.8. The parameters that will not be changed during the calibration process in this model include:

- Speed limits on different road section. There are two speed limits in the model: 120km/h for normal road sections and 100Km/h for tunnel road sections. See Fig 4.12.
- Traffic composition. In this study, only cars are considered.
- A car following model is applied. That is deceleration estimation. The 2-lane car following model is also applied with the number of vehicles equal to 4, maximum distance 100m. The maximum speed difference and the maximum on/off-ramp speed difference are not fixed and will be tuned for capacity calibration later. See Fig 4.13.
- Parameters for the speed in the queue and the maximum number of movements to be forecasted are not changed. See Fig 4.13.
- Vehicles length. Vehicles lengths can be obtained from the observed data. The statistics about the real vehicles lengths can be applied to the vehicles in the model. The statistics includes: the average vehicle lengths (only cars count), the deviation, the maximum, the minimum of the vehicles lengths.

4.3.3 Calibration of lane capacity

The traffic flow is an important parameter that serves as the base for traffic safety study: All the considerations relating to the distribution of risk indicators are based on different traffic flow ranges. Therefore, flow ranges need to be calibrated before any other calibration to assure that flow ranges obtained from the model are similar to the flow ranges obtained from the observed data.

Fig 4.15 shows the Speed/Flow relationship of the field data. Each data point in the figure represents the traffic flow for 5-minute aggregation interval. The maximum flow with the observed data is about 2000 vph and the traffic under this flow is still uncongested. This means that capacity of the lane must be higher than 2000vph. Therefore, the necessary condition for the lane capacity is that it should be greater than 2000vph. In addition, the lane capacity in the model should be smaller than 2500vph.

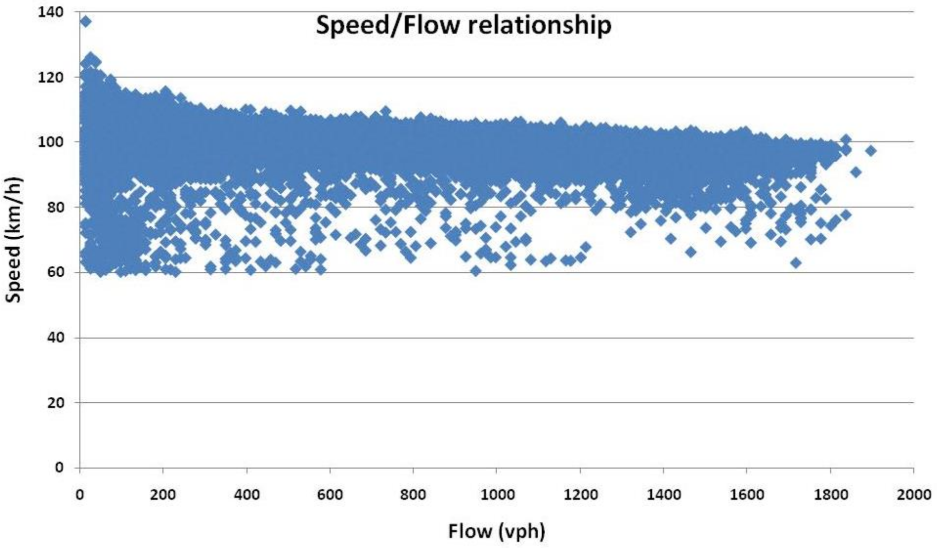


Fig 4.15 Speed/Flow relationship with observed data

According to AIMSUN 6.0 Manual, the parameters from Fig 4.9 that can affect the capacity of a motorway section are:

- 1-Reaction time.
- 2-Minimum distance between vehicles.
- 3-Maximum speed difference
- 4-Maximum on/off-ramp speed difference

The lane maximum flow for 5-minute intervals where the average speed drops down is used as the proxy for lane capacity. By maintaining the default value of the minimum distance between vehicles (which is equal to 1.0 (m)), the reaction time is increased by a step of 0.1 second from 0.1 to 1.35 seconds.

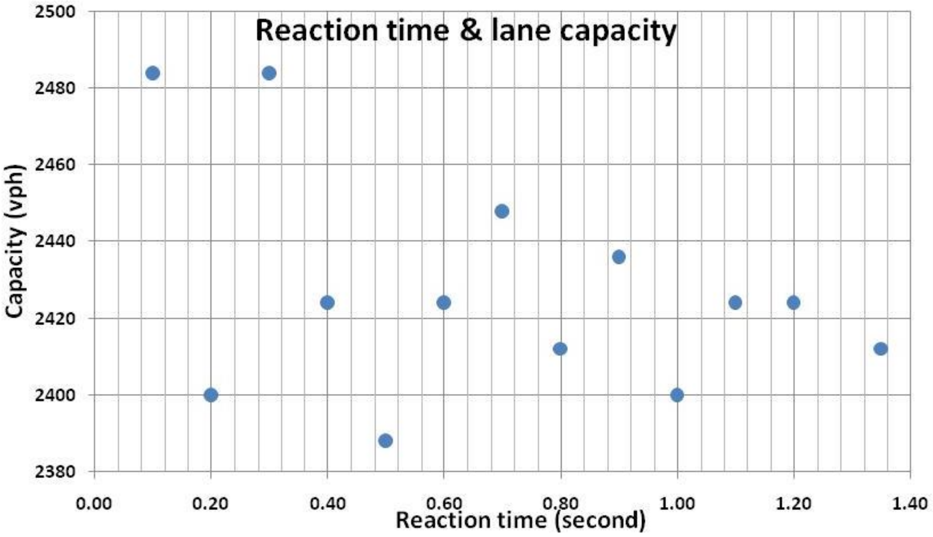


Fig 4.16 Influence of reaction time on lane capacity

As shown in Fig 4.16, the reaction time has influence on the lane capacity. When the reaction time increases, the lane capacity decreases. However, the influence is small and with the value range of the reaction time from 0.1 to 1.35seconds, the lane capacity is inside the desired capacity value range (from 2000 to 2500vph).

This means that the reaction time can be any value from 0.1 to 1.35sseconds. The reaction time of 1.08 second is then chosen because it is the reaction time recommended by Swiss authority. With the reaction time equal to 1.08second, and with the default values of the minimum distance between vehicles equal to 1m, the lane capacity is 2412 vph.

By fixing reaction time and adjusting minimum distance between vehicles, the influence of the minimum distance between vehicles on lane capacity is shown in Fig 4.17.

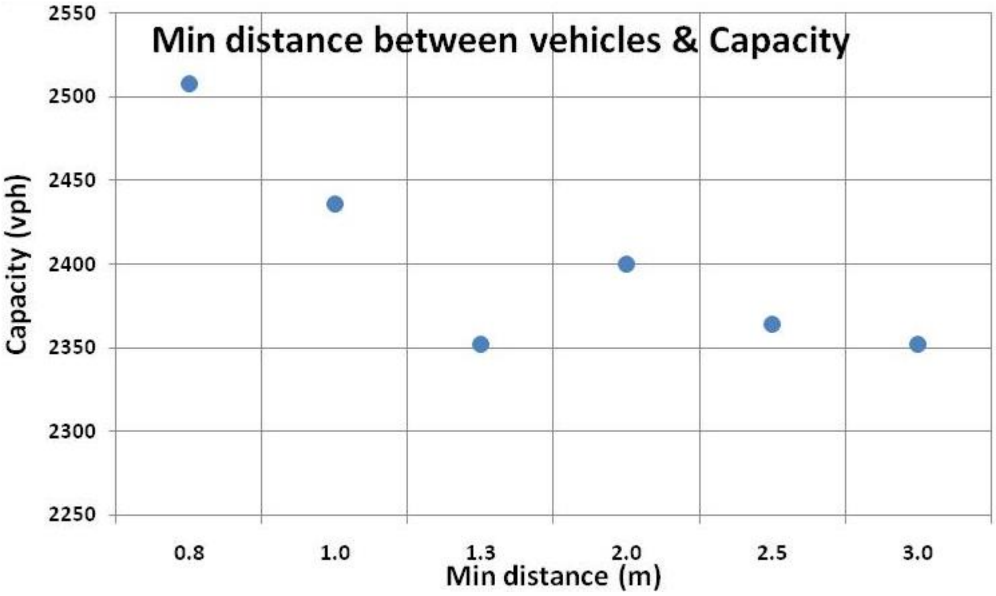


Fig 4.17 Influence of the minimum distance between vehicles on capacity

Results in Fig 4.16 and Fig 4.17 show that reaction time and minimum distance between vehicles have influence on lane capacity. However, with tested value ranges of these parameters (0.1-1.35s for reaction time and 0.8-3.0 for minimum distance between vehicles), lane capacity still falls into the desired capacity value range. This means that these two parameters can still be tuned in the next calibration tasks.

Two other parameters: maximum speed difference and maximum on-ramp speed difference are set with default values while tuning the reaction time and minimum distance between vehicles. By fixing the reaction time to 1.08second and minimum distance between vehicles to 1m, the two parameters about speed difference can be tuned.

The two parameters about the maximum speed differences are parts of the two-lane car-following model. In this model, the desired speed of a vehicle can be influenced by speeds of vehicles on its right adjacent lane (in case of driving on right side of the road). If the right adjacent lane is an on-ramp section, the maximum on-ramp speed difference is used. Otherwise, the maximum speed difference is applied. By applying this 2-lane car-following model, speeds of vehicles are interdependent and this can somehow influence the capacity of the road section.

Experiments were undertaken so that the maximum speed difference changes from 10 to 50 km/h by step of 5km/h and the on-ramp maximum speed difference changes from 30 to 10 km/h by a step of 10km/h. However, experiments' results do not show any influence of these parameters on lane capacity.

The summary of tuned parameters is in Fig 4.18.

Fig 4.18 Tuned parameters for lane capacity

Parameters	Values
Reaction time (seconds)	1.08
Minimum distance between vehicles (meters)	1.00
Maximum speed difference (km/h)	50.00
Maximum on/off-ramp speed difference (km/h)	70.00

4.3.4 Calibration of speed distribution

The next important factor that needs to be validated is the speed distribution. The parameters to be tuned in this step include statistics about the maximum desired speed of vehicles.

To produce the speed distribution, traffic flow is divided into 6 flow ranges (0-500vph, 500-800vph, 800-1100vph, 1100-1500vph, 1500-2000vph, >2000vph). In Fig 4.19, speed distributions of the observed data under each flow ranges are represented. Under the first 4 traffic flow ranges (0-500vph, 500-800vph, 800-1100vph, and 1100-1500vph), the speed medians are about 100km/h. Under higher flow range, the speed median is reduced to 95km/h.

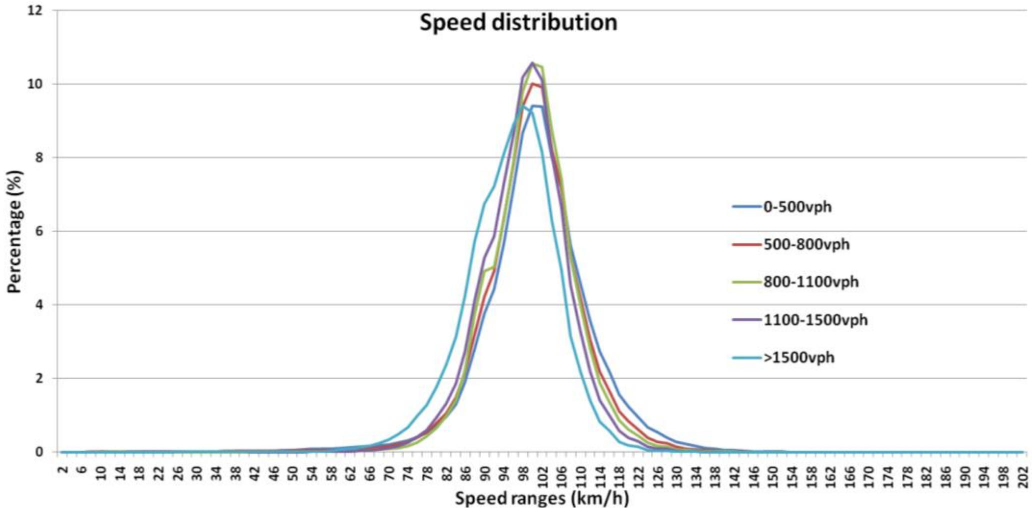


Fig 4.19 Speed distribution with real data

The maximum desired speed is the speed at which, drivers would like to drive when the traffic is fluid and there is nothing preventing the drivers from choosing the speed they want. This speed can be obtained from the reality by counting only the vehicles under free flow conditions, i.e. under the flow range from 0 to 500vph.

There are four parameters that need to be set for the maximum desired speed: the average speed, the deviation, the minimum, and the maximum. By changing the four parameters, the speed distribution can be formed. Among the four parameters, the average speed and the deviation are taken from real data under flow range 0-500vph while the maximum and minimum speeds are taken from real data under all flow ranges. In the statistics in Fig 4.20, only cars are considered.

Fig 4.20 Maximum desired speed with observed data

Parameters	Values
Average speed (km/h)	100.4
Deviation	10.5
Maximum Speed (km/h)	150.0
Minimum speed (km/h)	60.0

The tuning process for the four parameters in Fig 4.20 is below:

- The average speed is fixed. This is important because the speed distribution will be formed based on the average speed.
- The deviation is tuned so that the model can reproduce the speed range for all the traffic flow ranges. The deviation in Fig 4.20 is only for the free flow conditions.
- The maximum and minimum speed can be set to the maximum and minimum speed detected in reality.

The tuning process is stopped when three following parameters are similar in the simulation and the reality:

- The average speed under each flow range
- The deviation under each flow range
- The median speed under each flow range

Experiments are undertaken for the deviation values from 10 to 80 with the step of 10. By applying the stop criteria to the experiments results, a deviation of 70 is chosen as by that value the model can reproduce the best speed distribution.

Fig 4.21 shows speed distribution with the simulation data under 6 flow ranges. When the traffic is dense the 6th flow represents the congested traffic conditions that cannot be observed with real data. The 5th and 6th flow ranges (1500-2000vph and >2000vph, respectively) are different from the reality. This can be explained that in the simulation, there are congestions that cause the drop of speed, which is not available in reality.

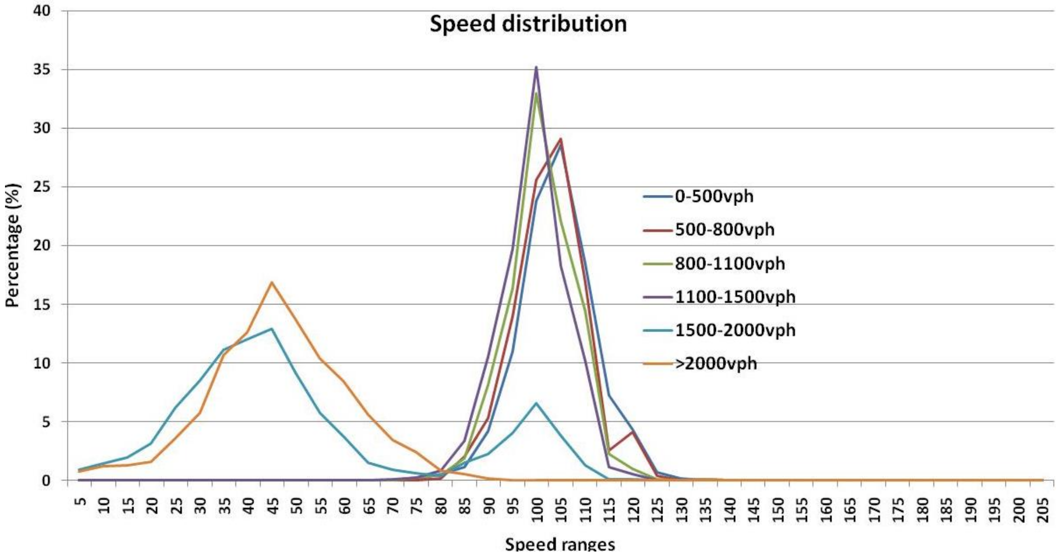


Fig 4.21 Speed distribution with simulation data

Fig 4.22 shows the mean, median and standard deviation values of speed under each flow range and for the two data sets. Although the maximum desired speed of vehicles in the simulation is set based on the free flow speed of the observed data (0-500vph), the average speed obtained from the simulation is smaller than the average observed speed.

Fig 4.22 Statistics of observed and simulation data

Flow ranges (vph)	Observed data			Simulation data		
	Mean (km/h)	StdDev	Median (km/h)	Mean (km/h)	StdDev	Median (km/h)
0-500	100.4	10.5	101	101.5	7.97	101
500-800	99.1	06.6	100	100.4	3.23	100
800-1100	99.1	04.6	100	98.7	2.48	99
1100-1500	98.0	04.8	100	97.5	2.34	97
1500-2000	95.1	05.8	96.0	55.8	24.69	38
>2000	-	-	-	43.8	10.08	43

Fig 4.23 summarizes the tuned values for parameters of maximum desired speed. Values in Fig 4.23 will not be changed during other calibration.

Fig 4.23 Tuned parameters for maximum desired speed

Parameters	Values
Average speed (km/h)	100.4
Deviation	70.0
Maximum Speed (km/h)	218.0
Minimum speed (km/h)	10.6

4.3.5 Calibration of lane distribution

Parameters to be tuned include:

- Percent Overtake
- Percent Recover

Percent Overtake is the percentage of the desired speed of a vehicle below which the vehicle may decide to overtake. This means that whenever the leading vehicle is driving slower than Percent Overtake % of the follower’s desired speed, the following vehicle will try to overtake. The default value is 90%.

Percent Recover is the percentage of the desired speed of a vehicle above which a vehicle may decide to get back into the slower lane. This means that whenever the lead vehicle is driving faster than Percent Recover % of the follower's desired speed, the following vehicle will try to get back into the rightmost lane. The default value is 95.

Fig 4.24 and *Fig 4.25* show the lane distribution with observed data (*Fig 4.24*) and simulation data (*Fig 4.25*). The principle rule applied for the lane distribution is that vehicles should use the normal lane with their desired speed. The overtaking lane is only used when drivers would like to overtake their front vehicles and then come back to the normal lane to continue with their desired speed. Therefore, under free flow, traffic volume on the overtaking lane is low and the occupation rate of the normal lane is higher than on the overtaking lane.

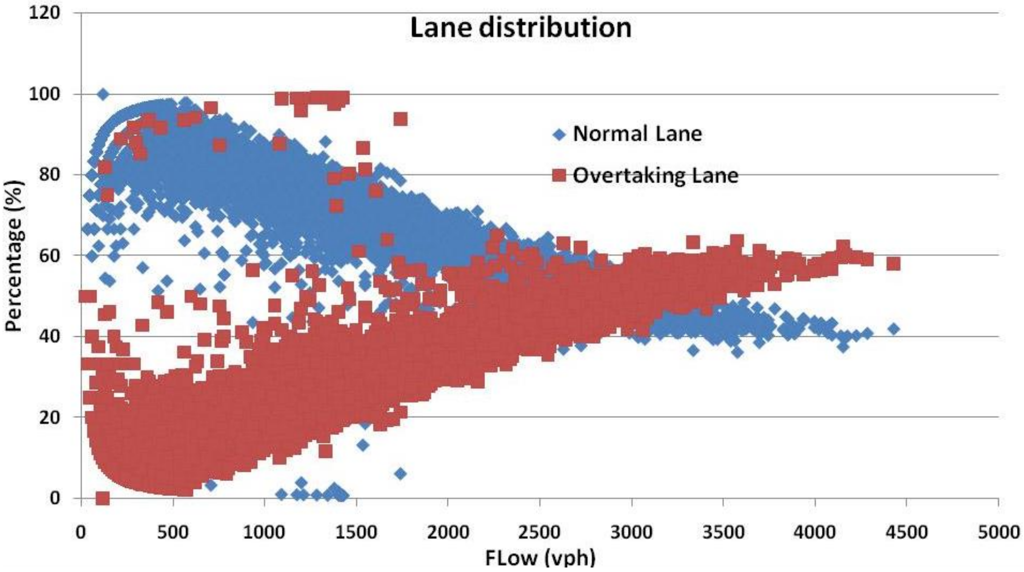


Fig 4.24 Lane distribution with observed data

In *Fig 4.24*, under free flow (smaller than 1000vph over two lanes) or average flow (from 1000-2000vph over two lanes) conditions, many vehicles tends to stay on the normal lane where they can reach their desired speed. When the traffic flow increases, more and more vehicles cannot circulate with their desired speed and they try to find their desired speed on the overtaking lane. On the overtaking lane, the desired speed is still not satisfied and thus vehicles keep staying on the overtaking lane, which makes the occupancy on the overtaking lane become higher than on the normal lane.

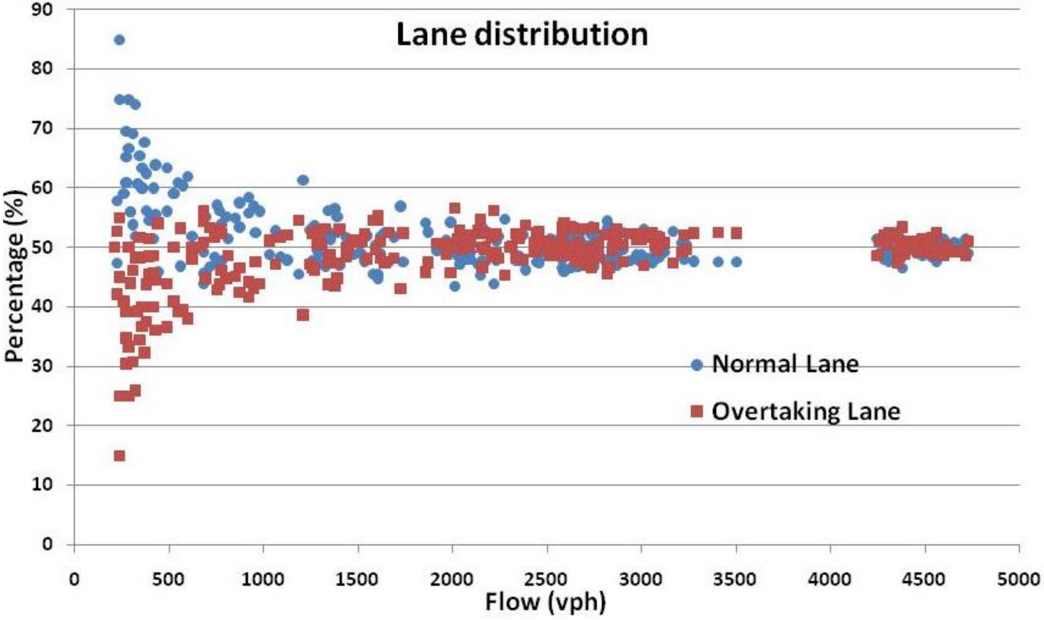


Fig 4.25 Lane distribution with simulation with default values of lane changing

Fig 4.25 shows the lane distribution with the default value of two parameters: Percent Overtake and Percent Return. It can be seen that the separation of the two lanes for traffic flows less than 2000vph over two lanes is not clear. This means that vehicles change to the overtaking lane too often.

The two parameters: Percent Overtake and Percent Return are tuned by applying different values. Percent Overtake is set from 80 to 96 with a step of 2 while Percent Return is set from 85 to 99 with a step of 2 and Percent Return is always greater than Percent Overtake. There are totally 51 value combinations of Percent Overtake and Percent Return that have been tested.

To compare the results given by each combination, the average percentage difference between the overtaking lane and the normal lane is calculated for flow ranges smaller than 2000vph over two lanes. The average percentage difference is then normalized by being divided by the average percentage difference with the observed data. This normalizes rate is called NAPD which stands for the normalized average percentage difference. Finally, the combination with the highest NAPD is chosen.

The highest NAPD is obtained from the combination of 96-99, i.e. the Percent Overtaking is 96 while the Percent Return is 99. The highest NAPD value is 0.51. This means that the average percentage difference in the simulation model is lower than in the observed data, i.e. the separation between two lanes in the simulation is still not as clear as in observed data.

Fig 4.26 shows the lane distribution with the combination of 96-99. The separation between two lanes is already better than the combination of 90-95 as in Fig 4.25. However, it is still far away compared to the reality.

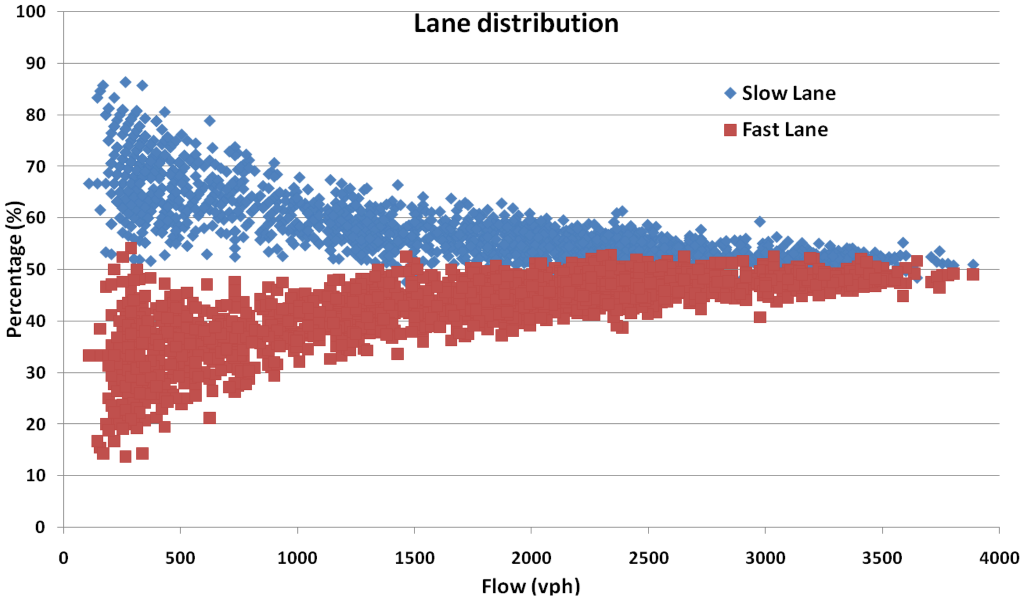


Fig 4.26 Lane distribution with simulation with tuned values of lane changing

Finally, the values are set for the two parameters as below:

- Percent Overtake=96
- Percent Return =99

4.3.6 Calibration of headway distribution and risk indicator distribution

Fig 4.27 shows the cumulative headway distribution with observed data. The proportion of vehicles having low headway is an important factor in traffic safety study. The headway above 6.0 seconds is still calculated but not shown in Fig 4.27 because this study does not consider the high headway values. When the traffic flow increases, it is more difficult for vehicles to find a large space in front of it. That is why the percentage of vehicles having headway smaller than 6 seconds is high (more than 60% out of all vehicles) for all flow ranges except the free flow range.

The smallest headway values observed are about 0.3 second. Under high flow conditions (i.e. for the flow range >1500vph), there are 50% out of all vehicles that have headway smaller than or equal to 1.5 seconds. If it takes 1.08 second to react against an incident on the motorway, about 20% out of all vehicles under high flow conditions are running too fast and would not be able to react if there were incidents in front of them.

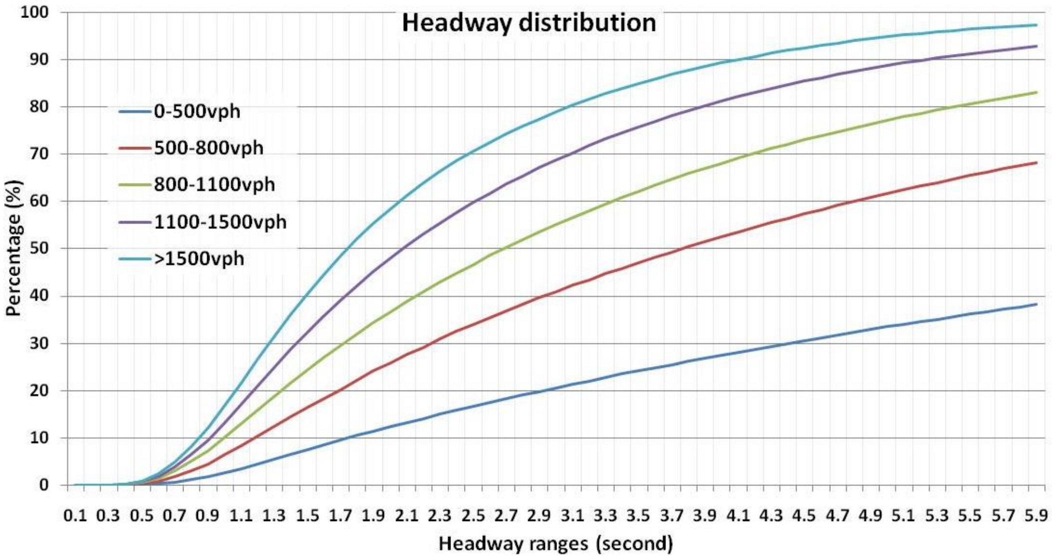


Fig 4.27 Cumulative Headway distribution with observed data.

In Fig 4.28, the percentages of different headway intervals are shown. The highest points on each curve represent the headway intervals that vehicles have the most frequently. All the highest points fall into the headway interval from 1.2 to 1.5 seconds and when the flow increases, the headway at those points increases.

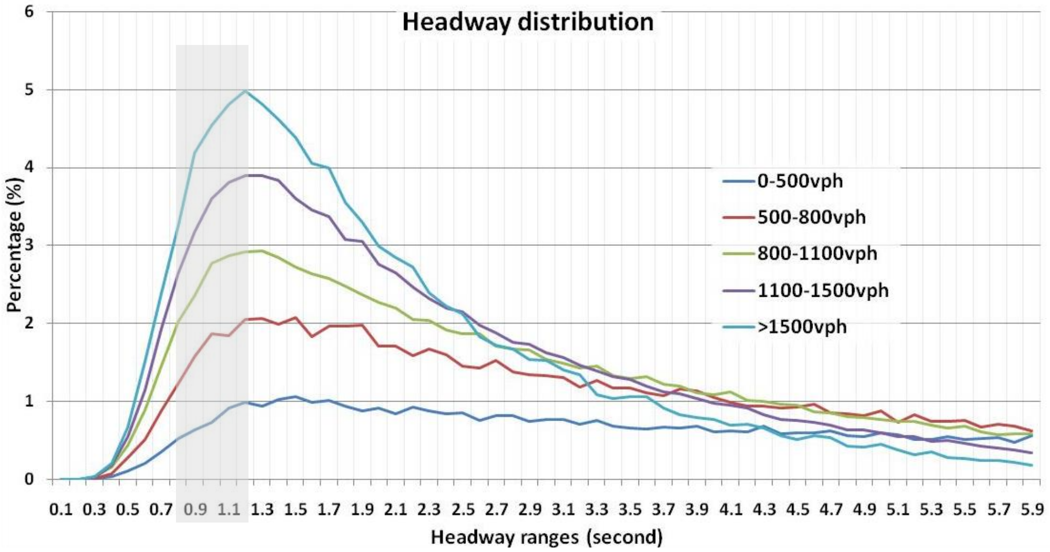


Fig 4.28 Headway distribution with observed data

From the observation with the real data, the following headway criteria or HWC are proposed as stop criteria of the calibration process so that the simulation model can reproduce situations similar to the real world:

- HWC1: The percentage of vehicles having headway smaller than or equal to 6 seconds is within that percentage in reality plus/minus 10%. This is applied to all flow ranges.

- HWC2: The headway value interval that the most number of vehicles have should be from 1.2 to 1.5 seconds. This is applied to all flow ranges.
- HWC3: The lowest headway should be from 0.3 to 0.5 seconds.
- HWC4: The percentage of vehicles having headways belonging to one headway interval should not be greater than 6%.

Once the three criteria HWC1, HCW2, and HCW3 are met, the model is believed to reproduce situations that are useful for studying traffic safety.

There are three parameters in the model that can be tuned for the headway calibration. They are:

- Reaction time
- Minimum distance between vehicles
- Simulation step.

After the calibration for lane capacity, the reaction time is set to 1.08 second and the minimum distance between vehicles to 1m. The simulation step used is 0.75 seconds. Fig 4.29 shows the distribution and Fig 4.30 show the cumulative distribution of headway with previously fixed values of three parameters. It can be seen that HWC1 and HWC2 are met except that the minimum time headway is too high compared to the observed data.

As mentioned in AIMSUN manual (page 191), "*a simulation step may affect not only the computing performance but also some simulation outputs, such as the section capacities, for example. The smaller the simulation step is, the higher capacity values are obtained. The reason for this is that the drivers are "more skillful", as they have shorter reaction times. They can drive closer to the preceding vehicles, they can find gaps more easily, they can accelerate earlier, they have more opportunities to enter the network, etc.*" This means that to obtain lower headway values, the simulation step has to be lowered. However, this is a paradox because reducing the simulation step would cause the increase of lane capacity which is already high in this simulation model.

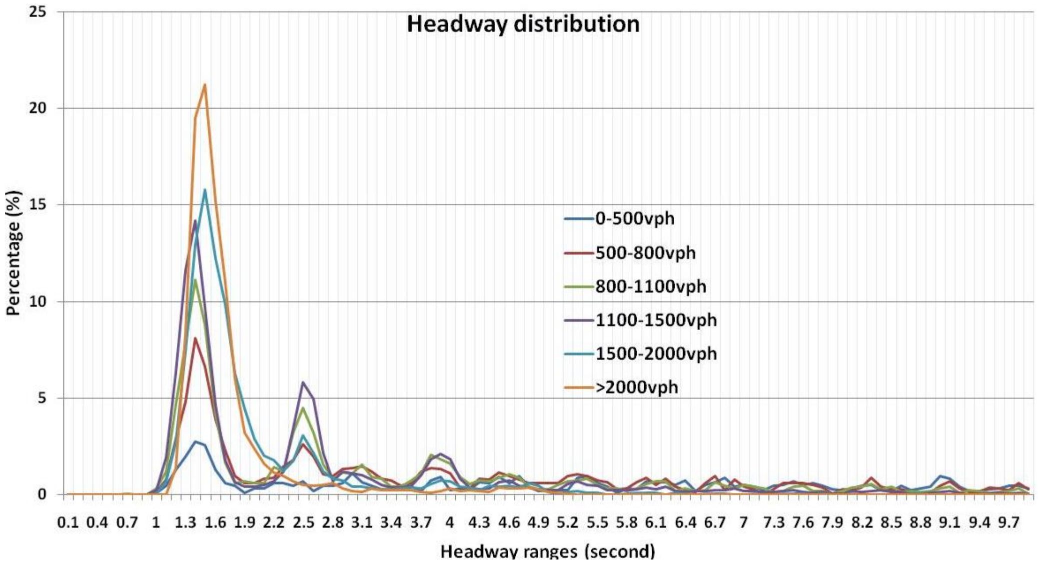


Fig 4.29 Headway distribution with simulation data

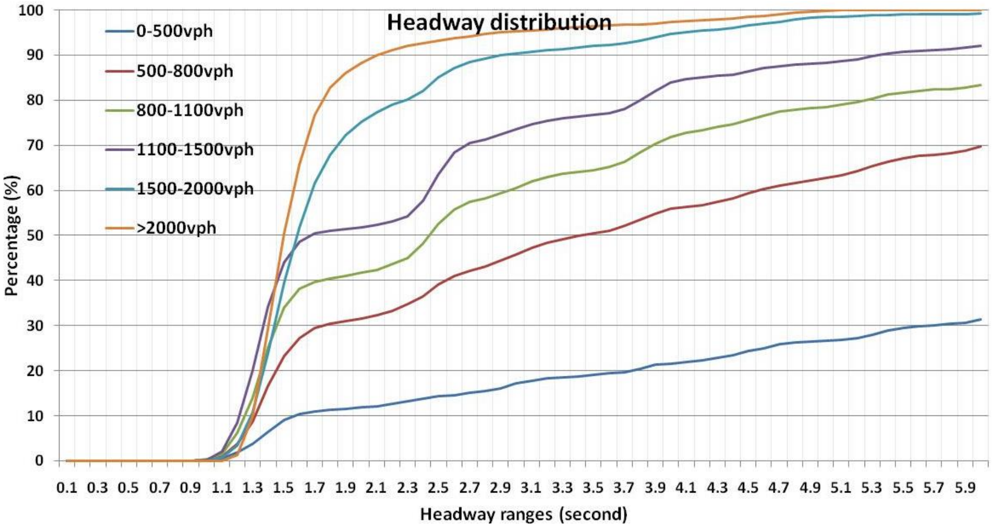


Fig 4.30 Cumulative headway distribution with simulation data

The minimum time headway is greater than the simulation step, a simulation step of 0.4 second is necessary to obtain the time headway of 0.4 second. Fig 4.31 shows the headway distribution with the new simulation step. It can be seen that the model has been improved so that HCW3 is met. However, the HWC2 is not met as the highest points are pushed to the headway interval from 0.8 to 0.9 seconds, which cause sharp percentage reductions compared to the reality. Another issue is that HWC4 is not met as the highest percentages are too high compared to the reality. As in Fig 4.29 and Fig 4.30, most of the percentages are higher than 10%, i.e. there are too many vehicles keeping headway of those headway intervals.

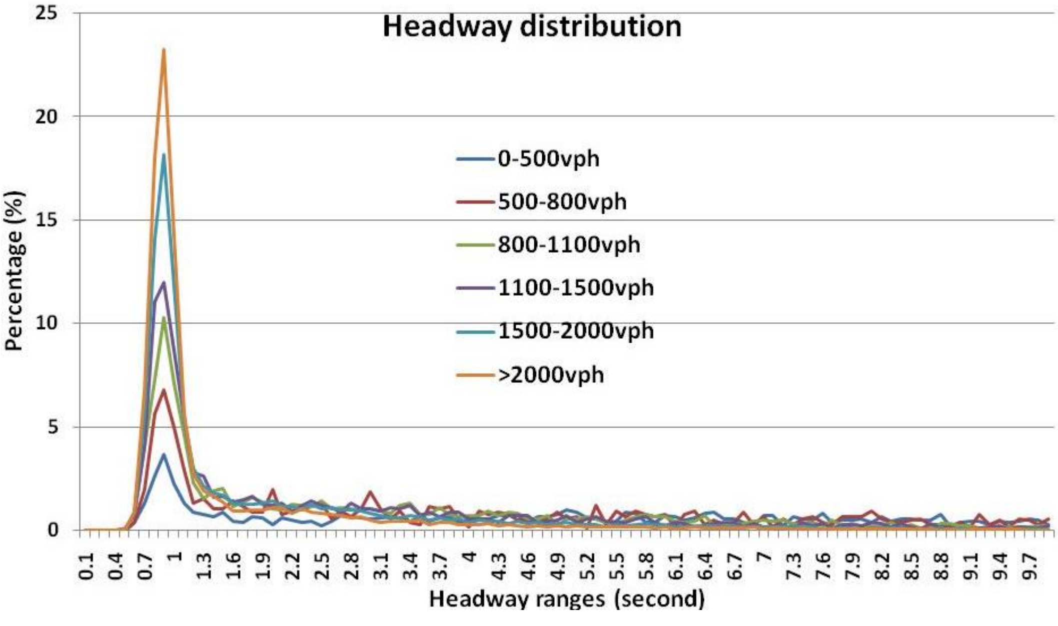


Fig 4.31 Headway distribution with simulation data

This can be caused by the fact that the minimum distance between vehicles has not been well distributed which concentrates vehicles' headway around the average value. To solve this problem, the standard deviation of the minimum distance between vehicles should be set higher and the wider outliners.

Fig 4.32 presents the headway distribution with the following parameters for the minimum distances between vehicles:

- Average: 1m
- Maximum distance : 2m

- Minimum distance: 0.5m
- Standard deviation stdev=0.5
- In Fig 4.32, headway distribution has been improved in following meanings:
- HWC4 are more closely approached.
- Highest points fall into headway interval from 1.0 to 1.1 seconds.

Despite the improvement, HWC2 and HWC4 are not satisfied. Further values for parameters of the minimum distance between vehicles are tested, there are no value set that can make the four criteria HWC1, HWC2, HWC3, and HWC4 satisfied. The value set that produces the results shown in Fig 4.32 is finally fixed.

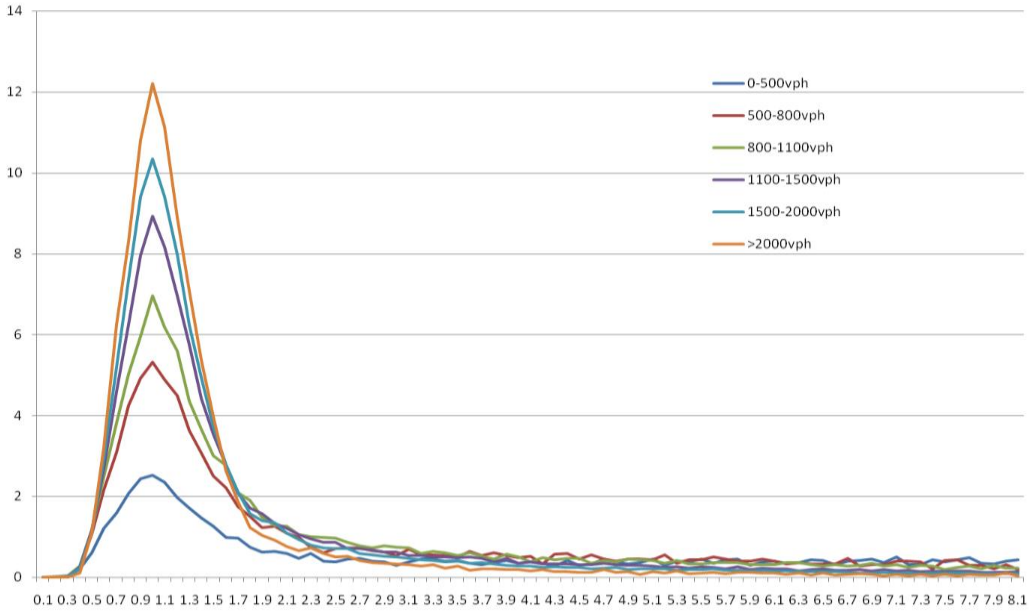


Fig 4.32 Headway distribution after changing standard deviation of the minimum distance between vehicles

Fig 4.33 summarizes the values set for parameters used. These are the value set that can reproduce the best the headway distribution compared to the reality.

Fig 4.33 Tuned parameters and their final values

Index	Parameters	Values
1	Reaction time	1.08 second
2	Reaction time at a stop	1.35 second
3	Minimum distance between vehicles	1m, max: 2m; min: 0.5m ; stdev=0.5
4	Simulation step	0. 4 seconds

Two risk indicators are used in this study: TTC and PBTR. These two risk indicators depend on the speed distribution and time gap distribution (i.e. headway distribution). As the speed distribution is good and the headway distribution cannot be further improved, there is no way to improve the distributions of the two risk indicators.

Fig 4.34 and Fig 4.35 introduce the TTC distribution with real data and with simulation data, respectively. The main difference between the two distributions is that the minimum TTC value in simulation is too high (about 2.5, 5.0, or 6.0 depending on the flow range) compared to the reality (about 1.0 second).

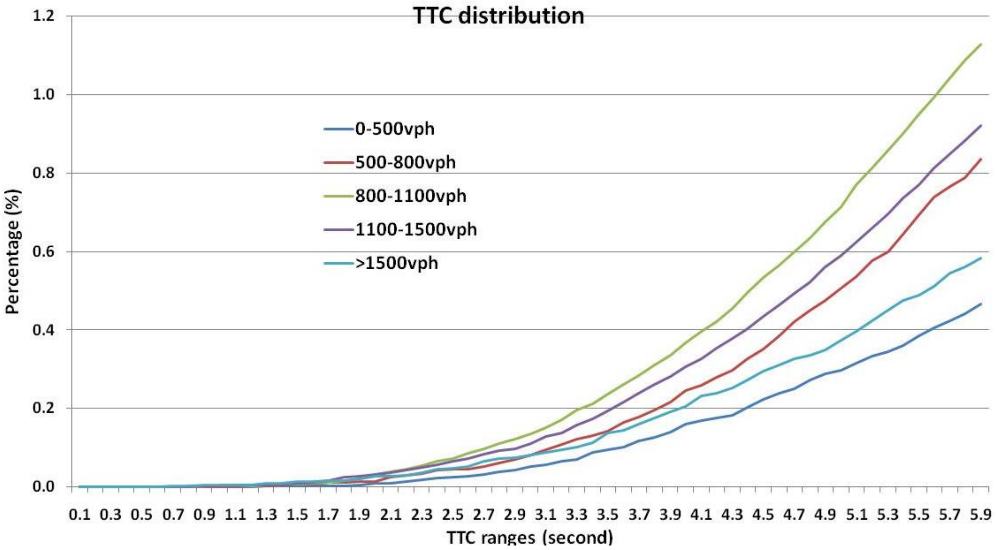


Fig 4.34 TTC distribution with observed data

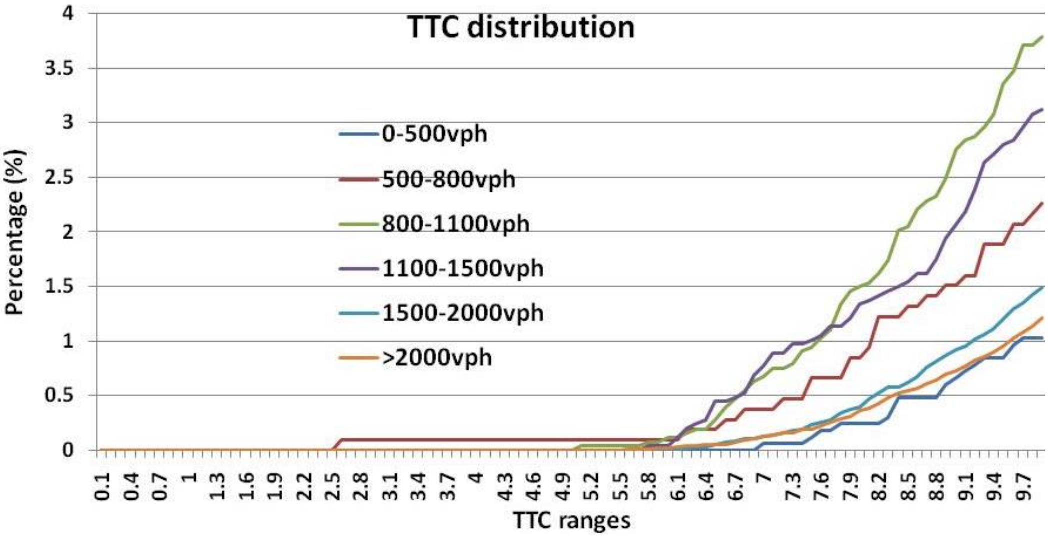


Fig 4.35 TTC distribution with simulation data

Fig 4.36 and Fig 4.37 also introduce the PBTR distributions with real data and simulation data, respectively. Once again, the two distributions are different. The main difference is that there are more vehicles in simulation having positive PBTR values than in reality.

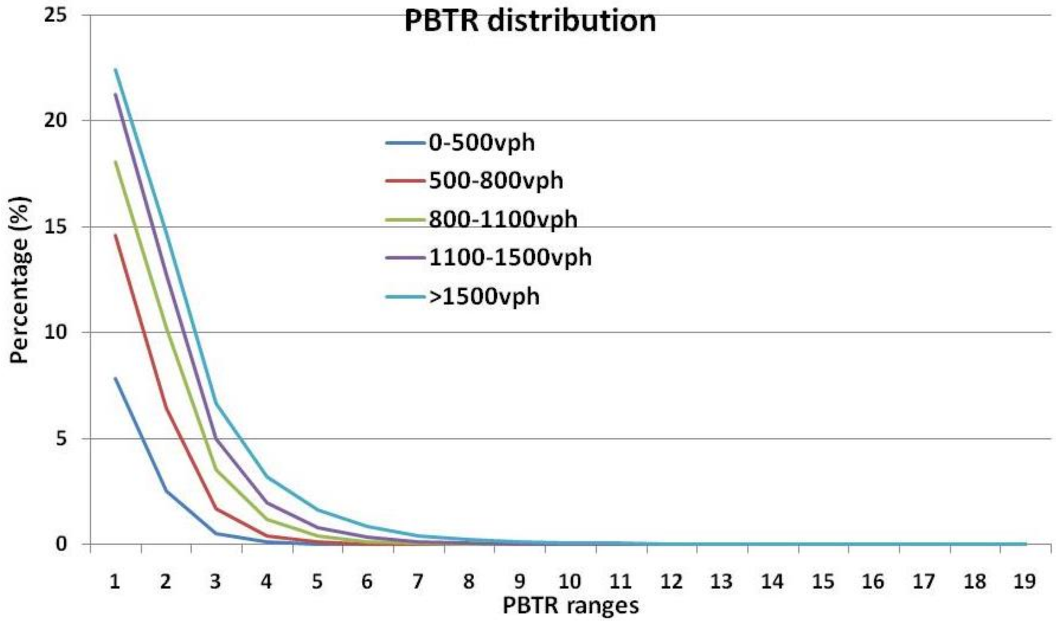


Fig 4.36 PBTR distribution with observed data

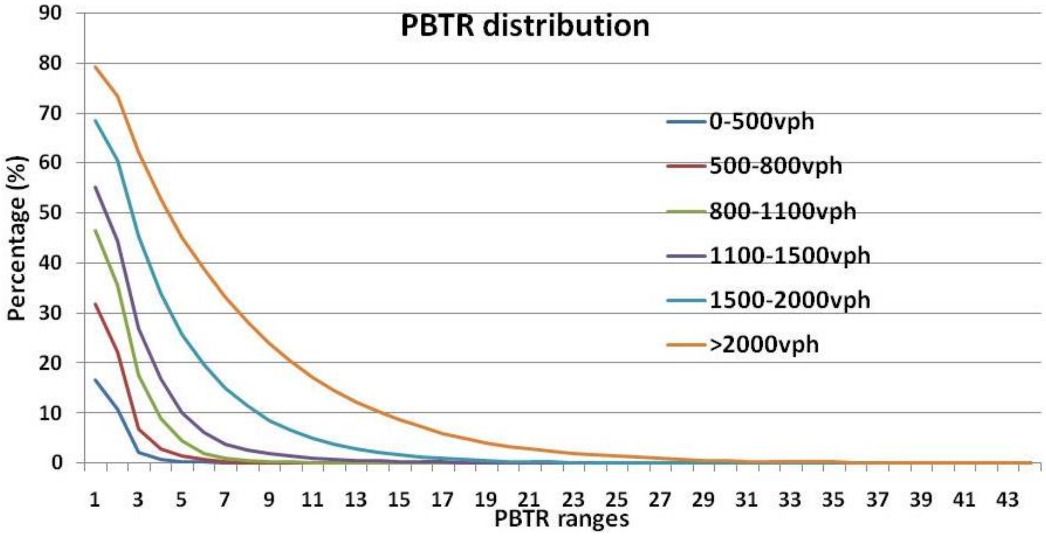


Fig 4.37 PBTR distribution with simulation data

The distributions of risk indicators are not calibrated but they will help defining the risk criteria in the simulation model based on the desired alarm rate.

4.3.7 Calibration issues

Model calibration is notoriously a tough task in traffic simulation. The calibration process depends strongly on the simulator used. Each simulator may provide a set of tunable parameters that allow its users to control the simulation model. In case of AIMSUN 6.0, tunable parameters are listed in Fig 4.9. The calibration process as presented from section 4.3.2 to 4.3.6 shows that there exist still calibration issues that need to be fixed for this simulator.

Firstly, the lane capacity is always relatively high compared to reality. For example, the lane capacity of the model is usually about 2500vph or more per lane. Although a list of parameters is tuned, it is difficult to reduce the lane capacity in the model. The lane capacity cannot be reduced down to 2300vph.

Lane distribution is also another issue of the model. If under high flow conditions, equal distribution of vehicles on two lanes is correct, the lane distribution under low flow ranges is far from the observation in reality. Furthermore, the adjustment of tunable parameters does not change much.

The calibration for the headway distribution is also unsatisfied. Headway depends on simulation step.

Unsatisfied headway distribution leads to a very bad TTC distribution. Minimum TTC value is about 6 seconds which is rather high compared to real TTC of smaller than 1 second observed with field data.

4.3.8 Risk indicators and risk criteria

Among the two risk indicators implemented in the simulation model, PBTR has its distribution closer to the observed PBTR distribution. TTC distribution is on the contrary very different from the observation, which is the gap between the reality and the simulation by AIMSUN 6.0.

To make the gap smoother, the risk criteria used in the simulation model are changed to adapt to the new distribution of risk indicators. However, the principle to develop the risk criteria for the observed data is still applied. It is based on the desired alarm rate to control the risk criteria.

Fig 4.38 Risk criteria used in the simulation model

Flow ranges	TTC				PBTR			
	Slow lane		Fast Lane		Slow lane		Fast Lane	
	Ind	Agg	Ind	Agg	Ind	Agg	Ind	Agg
0-500vph	≤6.0	0.6	≤6.0	0.7	≥3.5	0.3	≥3.5	0.4
500-800vph	≤6.0	1.2	≤6.0	1.1	≥3.5	1.0	≥3.5	1.2
800-1100vph	≤6.0	1.0	≤6.0	1.1	≥3.5	1.7	≥3.5	1.8
1100-1500vph	≤6.0	0.8	≤6.0	0.8	≥3.5	2.2	≥3.5	2.2
1500-2000vph	≤6.0	0.7	≤6.0	0.7	≥3.5	2.5	≥3.5	2.6
>2000vph	≤6.0	0.6	≤6.0	0.6	≥3.5	2.6	≥3.5	2.7

Fig 4.38 shows the risk criteria used in the simulation model. Due to the high values of TTC, the criteria for TTC are high. In reality, a vehicle having TTC smaller than 2 seconds can be considered as in a dangerous situation. In the simulation model, there is no vehicle having TTC smaller than 5 seconds. For this reason, to assure the desired alarm rate, the thresholds for the individual TTC have to be higher.

In Fig 4.38, for each risk indicator on each lane, there are two thresholds: the individual threshold (Ind) and the aggregate threshold (Agg). For example, if under a 5-minute period which is under 400vph flow, there is 0.7% vehicles having TTC smaller than 6.0 seconds, that 5-minute period will be considered as risky period and the risk criteria are considered as violated.

Although, only two risk indicators are implemented in this model, other risk indicators can also be used once their distributions are calibrated.

4.4 Preventive measures and their effects

4.4.1 Introduction and algorithm

Preventive measures are implemented whenever the risk criteria are violated. The algorithm for the preventive measure is listed below and illustrated in Fig 4.39.

- 1-For each detection step, consider the last M-minute interval.
- 2-If risk criteria are violated, do 3. Otherwise, do 4.
- 3-Take necessary actions.
- 4-If some actions are being taken, disable those actions. Otherwise, do nothing.

In this algorithm, the M-minute interval is a sliding time window, of which the last point is the current time. This means that from the current time, the traffic evolution during last M minutes is considered to determine if a preventive measure should be activated at the next detection step. In this study, M is equal to 5.

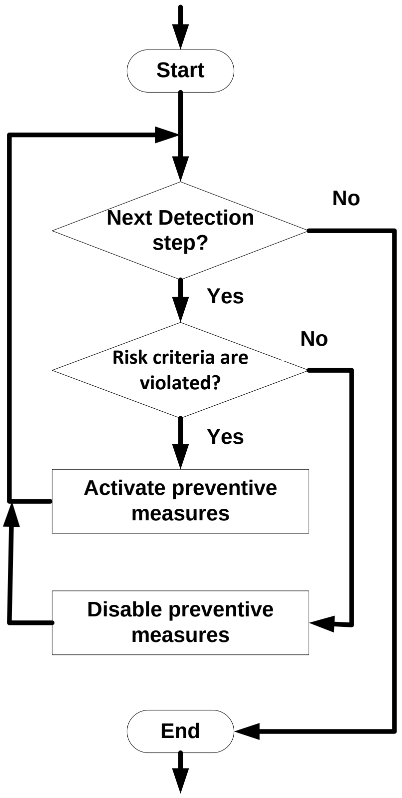


Fig 4.39 Algorithm for the preventive measure

If M is high, more traffic information in the past is used to decide if the risk criteria are violated. In that case, the recent effects of vehicles having risky behavior carry less weight. The algorithm can risk failing to capture new risky of driving behavior. If M is low, less information is needed. In this case, the risk criteria violation is usually decided by only some vehicles, which is not a collective trend in the traffic.

Another factor that can influence the performance of the measures is the detection step D. Decision is only given after each detection step. The sliding time window goes step by step so that small changes in the traffic evolution are recognized. In this study, a step of 30 seconds is applied.

There are many possible preventive actions that can be taken when the risk criteria are violated. In this study, one of the actions is implemented and its effectiveness is evaluated. That is to force vehicles reducing their speeds when the risk criteria are violated. There are many ways to implement these preventive measures in the simulation model, for example:

- Define new type of vehicles, change speed of these vehicles when the risk criteria are violated.
- Intervene directly involved vehicles that are in the alarmed road sections.
- Change speed limit of the involved road sections

In the first two implementation cases, only some of vehicles are intervened. The interpretation of these cases in the real world is that alarm information is directly sent to vehicles that can receive information, i.e. vehicles that have cooperative system installed on board. This means that the effectiveness of the implementations depends on the percentage of vehicles being able to receive information out of all vehicles and the percentage of vehicles that react after receiving the information.

For the third case, speed limits of road sections can be dynamically changed. As a consequence, all the vehicles passing through the involved road sections have to react to the dynamic speed limits.

From the simulation point of view, the third case is easier to be implemented. As an example, the third case is used as the preventive measure in this study.

In this section, the strategy for changing section speed limit is discussed bases on one detector. Six algorithms for changing speed limits are tested. Each algorithm is a case. The results of each case are evaluated and compared to the results of other cases. The evaluation includes estimating the improvement of safety and the cost paid for that improvement.

4.4.2 Implementation example: Dynamic section speed limit

Focus section
One focus section is selected for installing the preventive measures. Six cases are implemented bases on the focus section.

In the focus section, there are traffic detector 116 and three tunnels. The data from detector 116 is collected and processed to make the decision whether the risk criteria are violated or not. Fig 4.40 illustrates the position of the focus section in the overview of the simulation site.

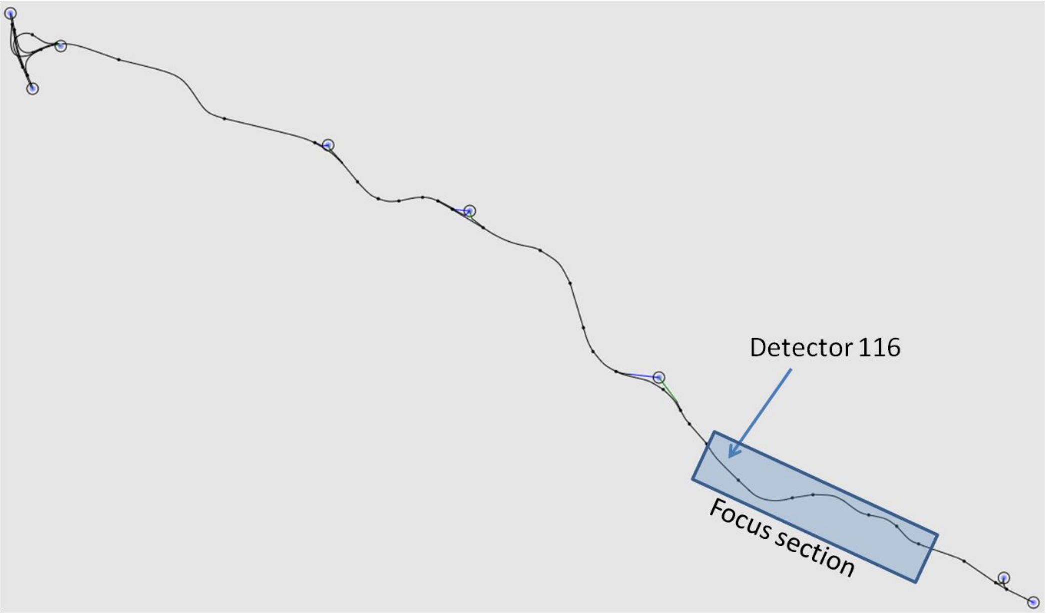


Fig 4.40 Focus section

Besides the detector 116, seven other traffic detectors, namely det116U1 to det116U7, are also installed at the upstream and one, 116D at the downstream of detector 116. These eight detectors can also capture traffic information necessary for the evaluation of the preventive measures. Small arrows in Fig 4.41 point to the positions of the eight traffic detectors.

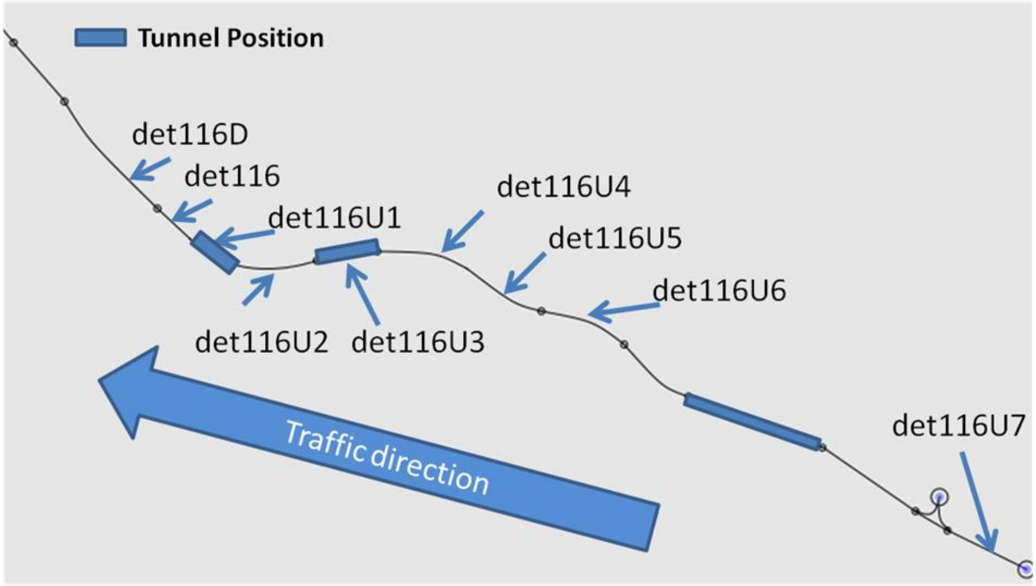


Fig 4.41 Installations inside the focus section

In Fig 4.42, smaller sections in the focus section are named SA, SB to SK from left to right. Two sections are separated by a dot. Among the eleven sections, three tunnel sections are SB, SD, and SH. Six implementation cases of preventive measures are described based on Fig 4.42.

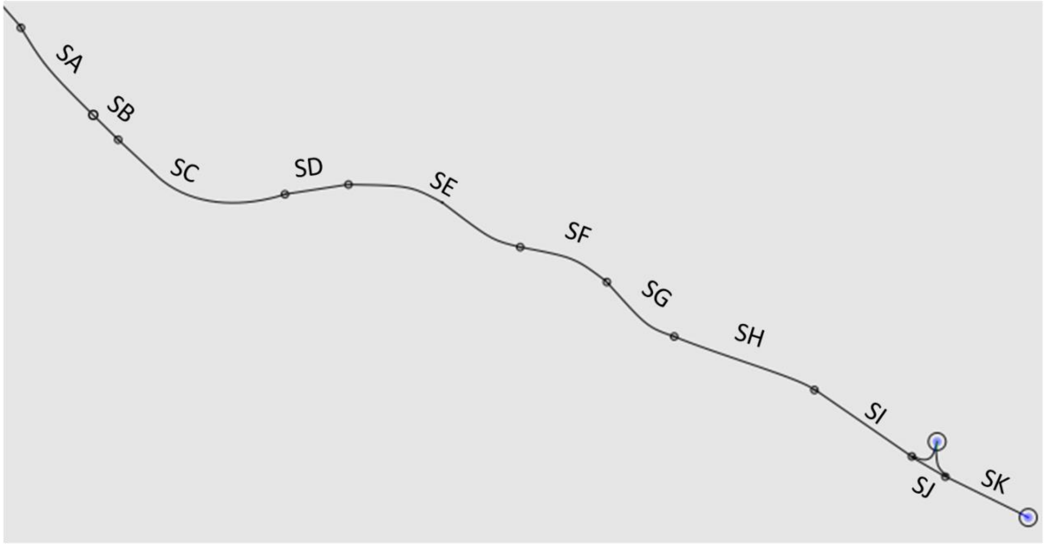


Fig 4.42 Smaller road sections in the focus section

Fig 4.43 presents the traffic detectors and their respective sections. Detector 116 is located in section SA. When an alarm is activated by detector 116, speed limits at the upstream sections of SA can be changed. In this study, control is applied to upstream sections only.

Fig 4.43 Locations of traffic detectors

Detectors	Sections	Speed limits (km/h)
Det116D	SA	120
Det116	SA	120
Det116U1	SB	100
Det116U2	SC	120
Det116U3	SD	100
Det116U4	SE	120
Det116U5	SF	120
Det116U6	SG	120
Det116U7	SK	120

Implementation cases

There are six cases implemented. Each case introduces a different setting for changing speed limit when the risk criteria are violated.

Under case 1, the speed limit of SA is reduced by a step of 10km/h until a speed limit of 80km/h when the alarm is activated. Right after the alarm disappears, the speed limit of SA is increased by steps of 10km/h until a speed limit of 120km/h.

For the case 2, the idea is that the risk alarm raised in one section could be caused by the traffic from the upstream section. This means that speed limit of SB is reduced by a step of 10km/h until a speed limit of 80km/h. After that, the speed limit is increased back to its normal limit of 100km/h when no more alarm is raised.

In case 3 and case 4, speed limits of four sections, namely SA, SB, SC, and SD, are involved when alarms are raised. When the risk criteria are violated, speed limits are reduced at SA. If the violation continues, the speed limit at SA is reduced further and the speed limit at SB starts to be reduced. If the risk criteria are still violated, the reduction of speed limits continues upstream up to SD. For each section, the speed limit is reduced until the minimum speed limit of 80km/h. For case 4, the trend is inversed: when the risk criteria are violated, the speed limit at SD is reduced first and the speed limit reduction continues downstream down to section SA.

The idea of case 5 comes from the fact that among four sections SA, SB, SC, and SD, there are two sections with speed limits of 100km/h (SB and SD) and two other with speed limits of 120km/h (SA and SC). It is possible that the irregularity of speed limits causes the risk alarms. When the risk criteria are violated, the speed limits of SA and SC are reduced first. After two steps, speed limits of SA and SC are now 100 km/h i.e. all four sections have the same speed limit. If the alarm continues, the speed limits on all the four sections are gradually reduced down to a minimum of 80km/h.

Fig 4.44 Section lengths

Section	Length (m)
SA	800
SB	200
SC	800
SD	220

The speed limit reductions for five implementation cases relate directly to the road sections SA, SB, SC, and SD. The lengths of these four sections are listed in Fig 4.44. The total length of the four road sections is 2.02km.

In case 6, a broader view is put on the problem by changing the speed limits of more road sections from SA to SH (see Fig 4.41). In Fig 4.41, there are three road sections with the speed limit of 100km/h, i.e. tunnel road sections while the speed limit on the other road sections is 120km/h. When the risk criteria are violated, speed limits are harmonized by reducing at step of 10 km/h from 120km/h to 100km/h for non-tunnel sections. This speed limit reduction makes speed limits of all sections equal. If the violation continues, further reduction of speed limit down to 80km/h is applied for all sections. The total length of road sections involved in Case 6 is 5.5km.

Results from all the implementation cases are compared to the normal case, i.e. the traffic conditions without any preventive measure implemented.

When to activate preventive measures?
The preventive measures are applied when the risk criteria are violated. The traffic situations are checked against the risk criteria every 30 seconds (see section 4.4.1). Traffic information during the last 30 seconds is added to the traffic information during 4minutes 30 seconds before the last 30 seconds. If the risk criteria start to be violated for the last 5 minutes, it is highly probable that the traffic evolution during the last 30 seconds causes the violation. Preventive measures can be applied right after the violation of risk criteria caused by the last 30 seconds.

However, it is arguable that the violation of risk criteria is just a fluctuation of the traffic during the last 30 seconds over the period of 5 minutes. It is then worth checking the next periods of 30 seconds to ensure that the traffic flow is risky. In this case, preventive measures are activated only after two or more periods of 30 seconds. The advantage is only persistence risks will raise the alarm. However, it may not be sensitive enough for risk management.

In this study, preventive measures are activated immediately when the risk criteria are violated.

When to disable the preventive measures?
When the traffic flow is risky, preventive measures are activated. Till a point, the traffic flow returns back to safe state, i.e. the risk criteria are no more violated. If during the last 30 seconds, risk criteria are not violated, should the preventive measures be disabled?

Similarly to their activation, preventive measures are disabled immediately when risk criteria are no more violated.

4.5 Outcome evaluation

4.5.1 Simulation experiment

The outcome evaluation is undertaken based on the same AIMSUN experiment. All simulation parameters are kept unchanged after the model calibration described in section 4.3.

Different implementation cases are coded in an API plug-in supported by AIMSUN. Speed limits for road sections are changed accordingly in the API code.

The same fifteen replications are executed for each implementation case. The results introduced in this section are summed over or averaged over fifteen replications.

4.5.2 Safety improvement measurement

The risk criteria are used as the evaluation of risk at each detection step, the number of times where the risk criteria are violated is used as an index of risk. This number is called violation count.

4.5.3 Cost estimation

Decreasing traffic risk may increase the cost of other metrics and therefore to evaluate the effectiveness of the preventive measures, other costs should be taken into account. There are many parameters that can be used to estimate that cost such as travel time, gas consumption, or CO2 emission, etc. In this study, the cost for implementing preventive measures is represented as the average travel time of per km.

4.5.4 Simulation issue

An issue detected relates to the number of times where the risk criteria are violated. If a traffic detector is installed at an extreme point of a road section, i.e. the starting point or the ending point, the violation count tends to be higher compared to the violation count by the same detector installed at the center of the section.

Two road sections SA and SF are chosen to check this hypothesis. Twenty-six traffic detectors are installed in SA and SF with the spacing of 15m and 10m, respectively, so that the detectors cover the whole road sections. Results in Fig 4.45 show that traffic detectors near the two extreme points of a road section have higher violation count.

To avoid this simulation issue, traffic detectors used for the outcome evaluation should be installed at the middle areas of road sections. Detectors listed Fig 4.43 match this requirement.

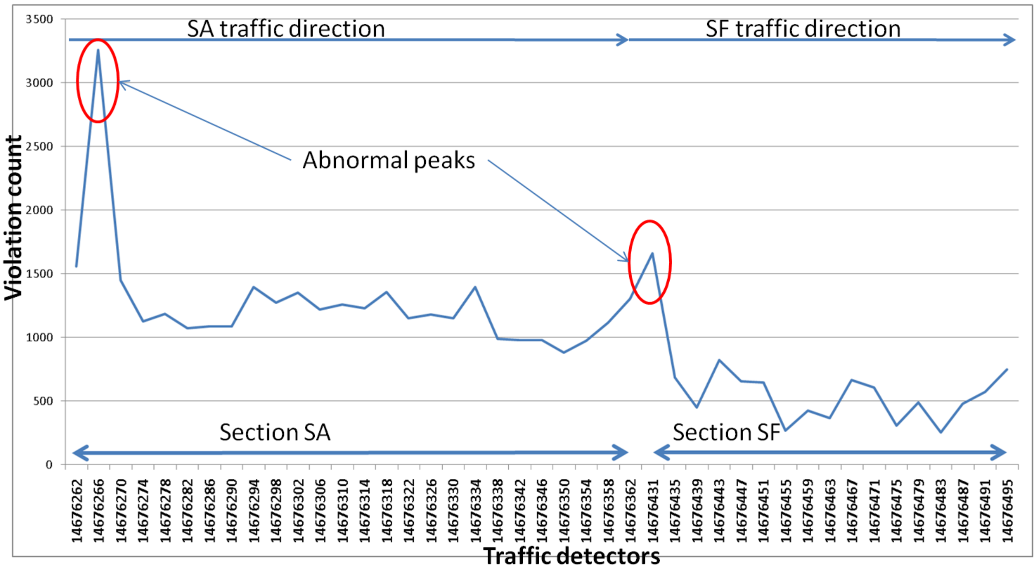


Fig 4.45 Violation count depends on positions of detectors in a section. This is under normal case.

4.5.5 Outcome evaluation

Fig 4.46 summarizes the outcome by all the implementation cases with the average number of alarms per day detected by the traffic detector det116 and the average travel time per kilometer.

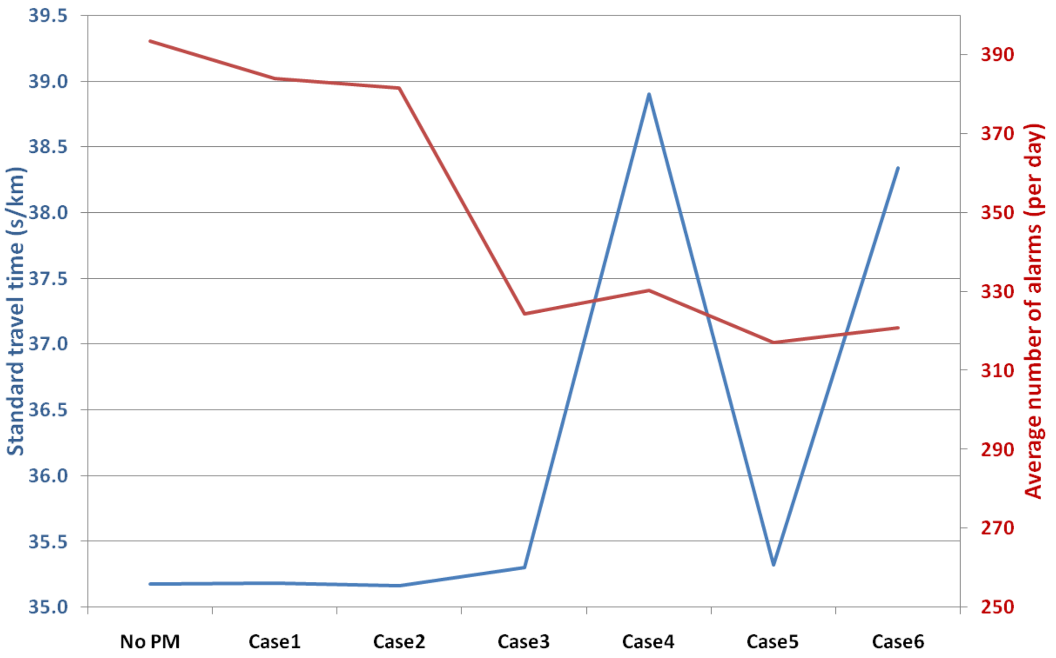


Fig 4.46 Outcomes by different implementation cases

As shown in Fig 4.46, the safety is generally improved when preventive measures are installed. In normal case (No PM), the violation count is the highest while the violation counts in other cases are lower. The violation counts are much lower in case 3, case 4, case 5, and case 6.

However, the cost paid for safety improvement is not always proportional to the reduction of violation count. Among cases where safety is improved the most, case 6 and case 4 require a higher costs compared to case 3 or case 5. For this reason, if only det116 is considered, case 3 and case 5 are the most cost effective solutions to improve safety, i.e. to reduce the crash risk with a small increase of travel time. Besides, case 6 where the speed limit intervention is on a long road section further investigated.

To assess the overall effectiveness of case 3, case 5, and case 6, other traffic detectors are installed at the upstream and downstream sections of detector det116. The considered detectors are listed in Fig 4.43.

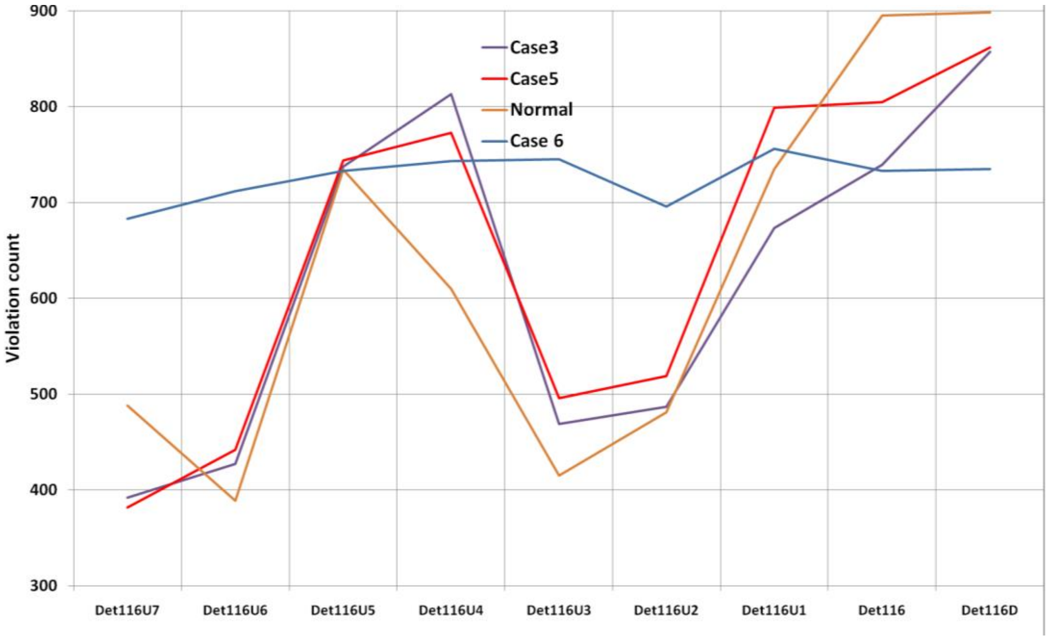


Fig 4.47 Overall performances of implementation cases

Fig 4.47 shows that the safety improvement recorded by det116 is not only paid by the increase of the standard travel time but also the increase of violation counts recorded by upstream detectors. The violation count is most increased at Det116U4 for the case 3 and case 5 compared to the normal case.

If the total violation count on the whole road section is considered, case 3 has the best performance thanks to its lowest violation count as illustrated in Fig 4.48. In the contrary, the implementation in case 5 has bad effects on traffic safety at upstream sections due to the high violation count caused by case 5 in comparison to the normal case. In case 6, the total violation count is even higher than in case 5.

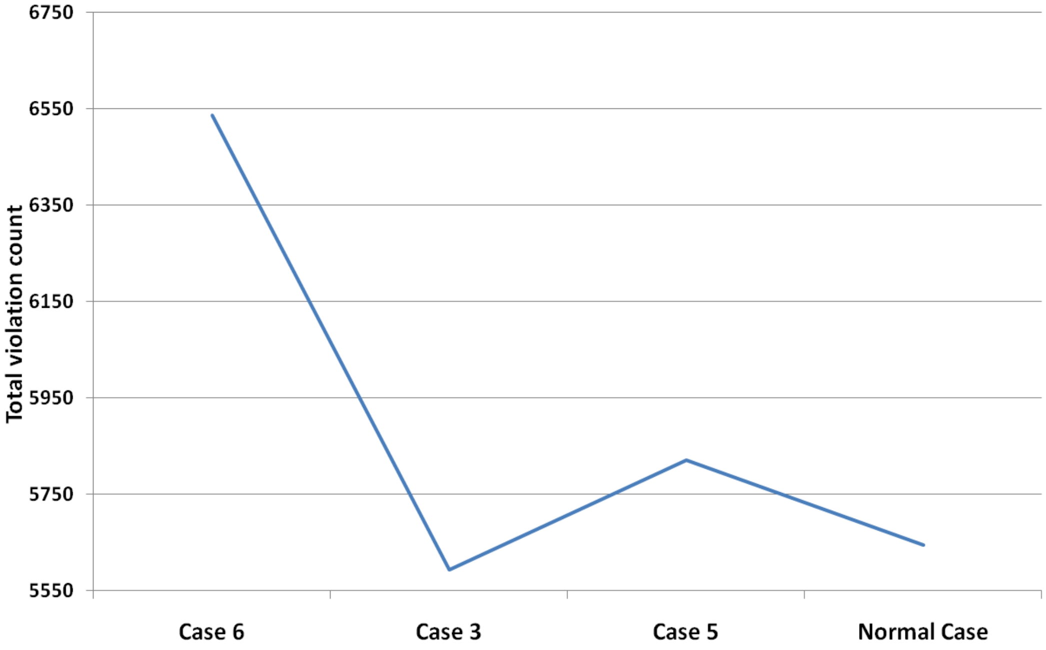


Fig 4.48 Total violation counts in three cases

4.6 Conclusions

Chapter 4 presents the application of a simulation model for implementing preventive measures that otherwise cannot be undertaken in the real world. The model is constructed based on the observed data collected from traffic detector 116 on the motorway A9 in Switzerland. Other input data necessary for the model originates from other project undertaken at EPFL. The model was calibrated before serving as a tool for traffic safety study, which is the objective of the simulation model.

The model calibration is a process of tuning model parameters so that the model matches with the real world and best represents the simulation road section from traffic safety point of view. The calibration process in this study deals with lane capacity, lane distribution, speed distribution, and headway distribution. During the calibration process, simulation issues for the study in traffic safety are identified and discussed in section 4.3.7. Taking into consideration the short comings of present simulation, the best set of values for model parameters are fixed for the next step of this study.

This chapter also discusses the different algorithms for implementing preventive measures when the risk criteria are violated. The risk criteria in this chapter are based on observed data and are introduced in section 3.5. Six cases (algorithms) for the implementation of speed limit reduction are proposed. The outcome of each case, including the improvement of traffic safety and the cost incurred for risk improvement is evaluated and compared against other cases. The cost that is increased travel time and the decreased in violation count at upstream are considered.

The results indicate high effectiveness for case 3 and case 5 if only det116 is used for risk evaluation. If other detectors upstream of det116 are considered, only case 3 perform better than the business as usual case. It can be concluded that the implementation of case 3 is most suitable for improving traffic safety.

Although case 5 presents negative effects at the upstream sections, it can still be applied when each of the upstream detectors is used for controlling the traffic crash risk. This is to say that when all traffic detectors function similarly to what det116 does (captures the traffic data, checks with the risk criteria, applies necessary actions), the violation count at each detector would be reduced, i.e. the total violation count would also be reduced.

Case 6 is a good example of a more global implementation of preventive measures with a long road section intervened when the risk criteria are violated. Although the average violation count is reduced at the control detector (the det116) in this case, the average travel time increased substantially. Furthermore, the total violation count from traffic detectors installed along the considered road sections also rises significantly. As shown in Fig 4.48, the violation counts by traffic detectors det116U6 to det116D are similar, i.e. the fluctuation of violation counts is lower compared to other cases. The violation count by det116U7 increases compared to other cases. This indicates that traffic risk might have shifted upstream of the road section with the preventive measures.

5 Conclusions

5.1 Summary

This report discusses about how to improve motorway traffic safety based on risk indicators and Swiss motorway conditions. The study is undertaken at the study site which includes motorway road sections circling Lausanne city, Vaud canton, Switzerland. This study includes two stages and proposes a framework to build up a pro-active crash prevention system. During the first stage, risk indicators are tested with real data. The understandings at the first stage help to build up a simulation model at the second stage.

Motorways are the safest road type by design. To promote high speed, motorways are designed to assure that crash risk is the lowest because crashes under high speed are more severe. The implementations such as direction separators, no intersection, etc aim at making the traffic flow fluent and reducing the collision possibility between vehicles. Therefore there is small chance for some crash types to occur while other crash types become more typical. As shown in Fig 2.8, three dominating types of crash on Swiss motorways are single vehicle crashes, rear-end crashes, and sideswipes crashes. Rear-end crashes can occur when vehicle speeds are not homogenous due to drivers’ disregard of the safe distance. Sideswipe crashes can occur when vehicles overtake or change lanes. Most of the single vehicle crashes are caused when the drivers try to avoid collision with other vehicles which would lead to rear-end or sideswipe crashes.

Most of the risk indicators used in this study relate to rear-end crashes. Five risk indicators are selected for testing their sensitivity and performance. The distributions of risk indicators provide value ranges of risk indicators while checking the evolution of risk indicators before crashes can certificate the performance of risk indicators.

The sensitivity analysis of risk indicators with real data results in a guideline to define the risk criteria as below:

- There should be two thresholds defined for a risk indicator: the individual threshold and the aggregate threshold.
- The trade-off between the two thresholds is presented in Fig 3.30.

Although the number of crashes that can be used for this study is limited, risk criteria are defined and reported in [Pham, M.-H. et al., 2008].

The guideline of risk criteria definition is the most important result from the first stage. According to this guideline, the risk criteria can be defined based on the desired alarm rate, the detection rate, and the false alarm rate.

The simulation model built at the second stage of the study is a platform to implement preventive measures when the traffic risk is identified by the risk criteria. The model is calibrated so that it best represents the real world. Then the implementation of preventive measures is undertaken and outcomes are evaluated.

The study site during the second stage is the sub-area of the study site introduced in chapter 2. This is because the lack of input data for the simulation model.

The calibration process aims at adjusting and tuning model parameters to assure the model represent as much as possible the real world from traffic safety point of view. The model calibration composes of several calibration steps such as capacity calibration, speed distribution calibration, lane distribution calibration, headway distribution calibration. The output of the calibration process includes the calibrated model and some issues relating simulation model calibration. The detected issues lie out of simulator’s capacity and the calibrated model is the best model that can be obtained.

There could be many possible preventive measures that could be implemented such as vehicle speed intervention, speed limit intervention, gap intervention, etc. This study demonstrates one example of preventives measures when the risk criteria are satisfied. However, to evaluate the outcomes of the preventive measures, different ways for implementing the preventive measures are introduced.

Changing speed limits of road sections is the preventive measure demonstrated in this study. There are six implementation cases for this preventive measure plus one normal case in which, no preventive measure is implemented. To evaluate the outcomes obtained from seven cases, two parameters are used:

- The number of alarms rose during the simulation.
- The average travel time by vehicles in the simulation.

The best implementation case should be able to improve the traffic safety, i.e. reduce the number of alarms while it should not increase too much the cost paid for that improvement, i.e. the average travel time.

Among the implementation cases, the best solution is to reduce the speed limit of the road section where the controlling traffic detector is installed and if the risk criteria are still violated, speed limits at the upstream of the detector are reduced.

An interesting result is that the cost paid for improving safety at the section of the controlling detector does not only include the increase of the average travel time but also the increase of alarms at the upstream sections. Therefore the outcomes of the different implementation cases should be counted in a more global area and not only at one traffic detector or local road section.

For some implementation cases, the number of alarm reduces at the controlling detector but increases at the upstream detectors. This means that the risk moves from the controlling section to upstream sections. If upstream detectors are also controlling detectors, the risk could be reduced.

5.2 Conclusions

The conclusions for this study include two parts: methodology for detecting risky traffic situations and strategy for implementing preventive measures.

For the methodology of risk traffic situation detection, the risk criteria can be developed based on detection rate, false alarm rate, and desired alarm rate. The test with available crashes shows that the risk indicators can bring potential improvement for traffic safety.

Through this study, we also find that simulation calibration for traffic safety is a difficult task as the models for simulation are themselves limited to safe traffic conditions. The calibration problem is illustrated by the fact that calibration targets failed for the distribution of several variables such as headway distribution, lane distribution, and all the safety indicators' distribution.

With the best tuned simulation parameters, the results with simulation show that:

- Many strategies for VSL can be implemented to improve traffic safety.
- VSL implementation has certain cost.
- A global strategy would reduce more cost than local-based strategy.
- Finally, the optimized implementation of VSL is dependent on the study site and the considered motorway. An implementation is optimal if traffic risk and the cost reduce together.

5.3 Perspectives

This study provides a framework of methodology for improving motorway traffic safety. Although the results are very encouraging, there are still lots of work to do to build a complete pro-active motorway crash prevention system.

There could be more effective risk indicators that need to be defined to identify crash-prone conditions. In this study, the used risk indicators are single detector-based due to the limited number of traffic detectors. If the detector spacing is small enough, multi-detector based risk indicators can be defined to identify crash risk inside the inter-detector road section.

The risk criteria in this study can also be improved if more traffic detectors are installed so that traffic situations before crashes are recognized. However, the determination of the exact crash time is challenging with the current crash database. If a traffic detector is not close enough to crash positions so that abnormal events can be identified, it is difficult to know the exact crash time. Without the exact time of crashes, there is no way to determine pre-crash conditions. Therefore, although the number of crashes is high, there are only some of them are useful for defining the risk criteria. If there are more traffic detectors, the chance to check the exact crash time is higher, which is more preferable.

The simulation model in this study is calibrated as close as possible to the real world from traffic safety point of view. However, there are limitations of the application of the simulation model such as calibration issues which are beyond the scope of this study.

The preventive measures can also be varied to address the right cause of alarm. In this study, the risk criteria implemented in the simulation model are based on two risk indicators which are functions of speed. Therefore the preventive measure implemented is to reduce the speed limits of road sections. However, the two risk indicators are also functions of other parameters such as headway, time gap, and speed difference between vehicles (speed homogeneity) and other preventive measures should also be investigated.

Finally, all the potential improvements discuss in this chapter can be undertaken in a perspective that they will be implemented in a global traffic management center.

6 Acknowledgement

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Abréviations

Concept	Signification
AFR	Average Flow Ratio
agghold	Aggregate threshold
API	Application Programming Interface
ASTRA	Bundesamt für Strassen
ATC	Automatic Traffic Count
cm	Centimeter
COST	European Cooperation in Science and Technology
CVF	coefficient of variation in flow
CVS	coefficient of variation in speed
DATEC	Dipartimento federale dell'ambiente, dei trasporti, dell'energia e delle comunicazioni
DETEC	Federal Department of the Environment, Transport, Energy and Communications
DTS	Deceleration to safety time
DUI	Driving Under the Influence of alcohols and drugs
EPFL	Ecole Polytechnique Fédérale de Lausanne
Fedro	Federal Road Office
Fig	Figure
HWC	Headway Criteria
IBTR	Individual Braking Time Risk
ISI	individual safety indicator
IVIS	In-Vehicle Information Systems
Km/h	Kilometers per hour
LAVOC	Laboratoire de voies de circulation
N1LB	the section A1 from Lausanne to Bern
N9LS	the section from Lausanne to Sion
NAPD	normalized average percentage difference
OAFR	Overall Average Flow Ratio
OD	Origin-Destination
OFROU	Office fédéral de routes
OFS	Office fédéral de Statistics
PBTR	Platoon Braking Time Risk
PET	Post-Encroachment-Time
PICUD	Potential Index for Collision with Urgent Deceleration
PM	Preventive Measure
SI	Safety Indicator
SER	Secretariat for Education and Research
Prof.	Professor
SOSL	Speed Over Speed Limit
SUNB	seemingly unrelated negative binomial
TET	Time-Exposed-TTC
TIT	Time-Integrated-TTC
TTC	Time-To-Collision
UD	Unsafety Density
UVEK	Eidgenössisches Departement für Umwelt, Verkehr, Energie und Kommunikation

Concept	Signification
vph	vehicles per hour
VSL	Variable Speed Limits
VSS	Schweizerischen Verband der Strassen- und Verkehrsfachleute
WP	Work Package

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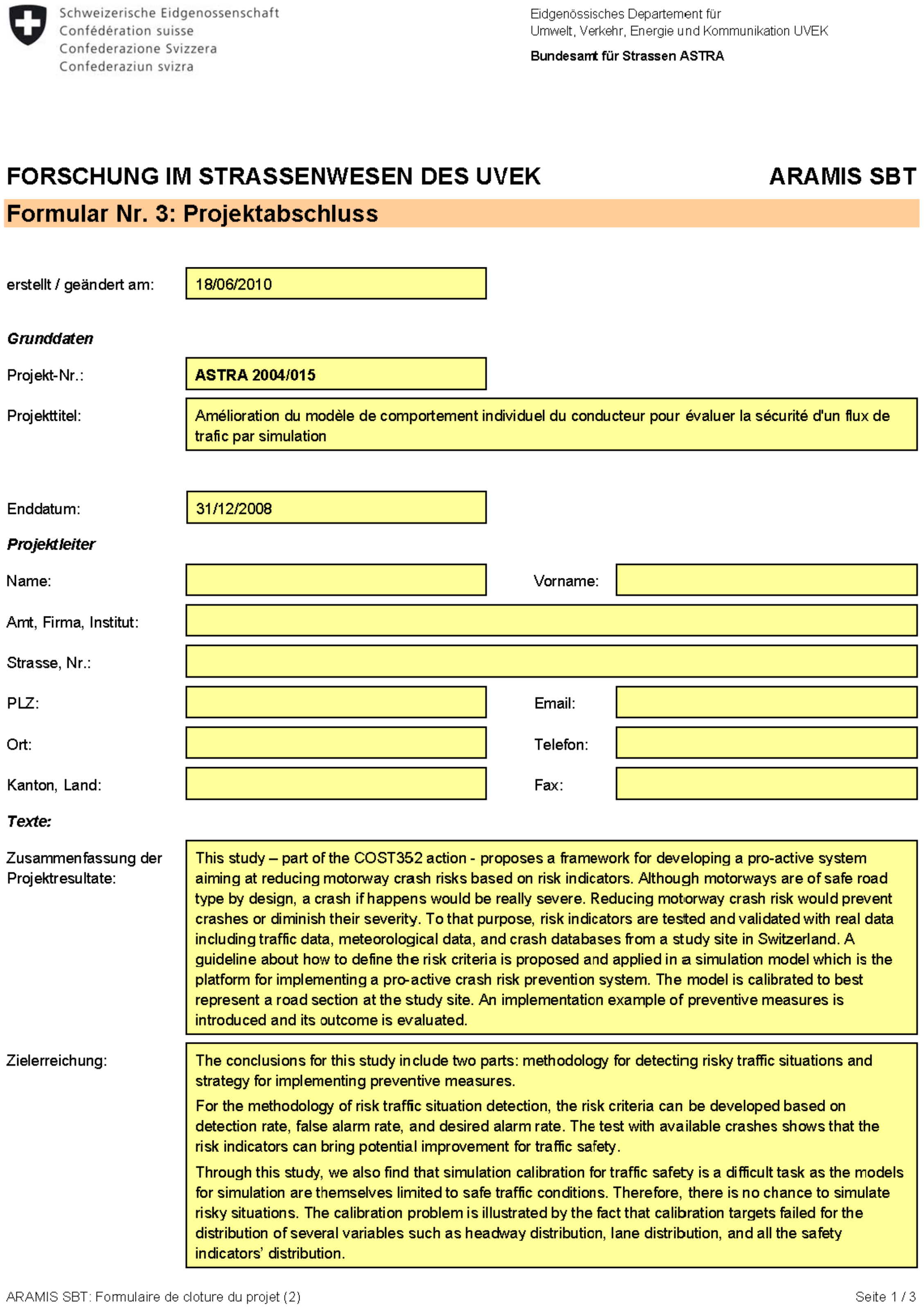
Clôture du projet



Schweizerische Eidgenossenschaft
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Eidgenössisches Departement für
Umwelt, Verkehr, Energie und Kommunikation UVEK
Bundesamt für Strassen ASTRA

FORSCHUNG IM STRASSENWESEN DES UVEK **ARAMIS SBT**

Formular Nr. 3: Projektabschluss

erstellt / geändert am: 18/06/2010

Grunddaten

Projekt-Nr.:	**ASTRA 2004/015**		
Projekttitel:	Amélioration du modèle de comportement individuel du conducteur pour évaluer la sécurité d'un flux de trafic par simulation		
Enddatum:	31/12/2008		
Projektleiter			
Name:		Vorname:	
Amt, Firma, Institut:			
Strasse, Nr.:			
PLZ:		Email:	
Ort:		Telefon:	
Kanton, Land:		Fax:	

Texte:

Zusammenfassung der Projektresultate:

This study – part of the COST352 action - proposes a framework for developing a pro-active system aiming at reducing motorway crash risks based on risk indicators. Although motorways are of safe road type by design, a crash if happens would be really severe. Reducing motorway crash risk would prevent crashes or diminish their severity. To that purpose, risk indicators are tested and validated with real data including traffic data, meteorological data, and crash databases from a study site in Switzerland. A guideline about how to define the risk criteria is proposed and applied in a simulation model which is the platform for implementing a pro-active crash risk prevention system. The model is calibrated to best represent a road section at the study site. An implementation example of preventive measures is introduced and its outcome is evaluated.

Zielerreichung:

The conclusions for this study include two parts: methodology for detecting risky traffic situations and strategy for implementing preventive measures.

For the methodology of risk traffic situation detection, the risk criteria can be developed based on detection rate, false alarm rate, and desired alarm rate. The test with available crashes shows that the risk indicators can bring potential improvement for traffic safety.

Through this study, we also find that simulation calibration for traffic safety is a difficult task as the models for simulation are themselves limited to safe traffic conditions. Therefore, there is no chance to simulate risky situations. The calibration problem is illustrated by the fact that calibration targets failed for the distribution of several variables such as headway distribution, lane distribution, and all the safety indicators' distribution.

ARAMIS SBT: Formulaire de cloture du projet (2) Seite 1 / 3

With the best tuned simulation parameters, the results with simulation show that:

- Many strategies for VSL can be implemented to improve traffic safety.
- VSL implementation has certain cost.
- A global strategy would reduce more cost than local-based strategy.
- Finally, the optimized implementation of VSL is dependant on the study site and the considered motorway. An implementation is optimal if traffic risk and the cost reduce together.

Folgerungen und Empfehlungen:

This study provides a framework of methodology for improving motorway traffic safety. Although the results are very encouraging, lots of work still remains to be done in order to build a complete pro-active motorway crash prevention system.

There could be more effective risk indicators that need to be defined to identify crash-prone conditions. In this study, the used risk indicators are single detector-based due to the limited number of traffic detectors. If the detector spacing is small enough, multi-detector based risk indicators can be defined to identify crash risk inside the inter-detector road section.

The risk criteria in this study can also be improved if more traffic detectors are installed so that traffic situations before crashes are recognized. However, the determination of the exact crash time is challenging with the current crash database. If a traffic detector is not close enough to crash positions so that abnormal events can be identified, it is difficult to know the exact crash time. Without the exact time of crashes, there is no way to determine pre-crash conditions. Therefore, although the number of crashes is high, there are only some of them are useful for defining the risk criteria. If there are more traffic detectors, the chance to check the exact crash time is higher, which is more preferable.

The simulation model in this study is calibrated as close as possible to the real world from traffic safety point of view. However, there are limitations of the application of the simulation model such as calibration issues which are beyond the scope of this study.

The preventive measures can also be varied to address the right cause of alarm. In this study, the risk criteria implemented in the simulation model are based on two risk indicators which are functions of speed. Therefore the preventive measure implemented is to reduce the speed limits of road sections. However, the two risk indicators are also functions of other parameters such as headway, time gap, and speed difference between vehicles (speed homogeneity) and other preventive measures should also be investigated.

Finally, all the potential improvements discuss in this chapter can be undertaken in a perspective that they will be implemented in a global traffic management center.

Publikationen:

M.-H. Pham and E. Chung. Risk indicators - Unsafe traffic conditions. In Cost Action 352 - The influence of in-vehicle information systems on driver behaviour and road safety. Final Report., pages 44-54. COST Office, Czech Republic, 2009.

M.-H. Pham, O. de Mouzon, E. Chung, and A.-G. Dumont. SENSITIVITY OF RISK INDICATORS UNDER MOTORWAY TRAFFIC REGIMES CLUSTERED BY SELF-ORGANIZING MAP. In 7th European Congress and Exhibition on ITS, Geneva, 2008.

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M.-H. Pham, E. Chung, O. De Mouzon, and A.-G. Dumont. Season effect on traffic: A case study in Switzerland. Seisan Kenkyu, 59(3):214-216, 2007.

Beurteilung der Begleitkommission:
Diese Beurteilung der Begleitkommission ersetzt die bisherige separate fachliche Auswertung.

Beurteilung:	Cette recherche s'inscrit dans un processus européen de recherche qui vise à regrouper et coordonner des thèmes de recherche abordés dans divers pays. Chaque pays peut développer, sur ses propres fonds, une problématique qui l'intéresse. Le Lavoc a ainsi développé une approche liée à la sécurité du trafic sur les autoroutes. Des études précédentes avaient montré la possibilité de donner des indicateurs du niveau de sécurité. Ici l'on est allé plus loin en prenant en compte les données de trafic, l'accidentologie et partiellement les effets de la météorologie. Une approche par simulation microscopique du trafic apporte des enseignements nouveaux. Le travail est de qualité et correspond globalement aux objectifs fixés. On peut regretter que le rapport sorte en retard et ne soit disponible qu'en anglais.
Umsetzung:	L'implémentation des résultats peut se faire dans des considérations de gestion du trafic en file sur autoroute. Comme le taux d'accidents de ce type reste élevé les gestionnaires de réseau et les opérateurs de centrale de gestion peuvent prendre connaissance de cette étude pour améliorer leur prise de décision en cas de situation à risque. Les bureaux d'ingénieurs du trafic peuvent s'inspirer de la démarche de microsimulation et surtout de calibration des modèles de trafic lors d'études attachées aux réseaux routiers.
weitergehender Forschungsbedarf:	Il n'a pas été explicitement détecté de besoin en recherche complémentaire. L'on peut simplement souhaiter que les chercheurs continuent de donner dans le futur une haute priorité aux problèmes de la sécurité routière.
Einfluss auf Normenwerk:	Les résultats présentés sont indépendants des normes existantes et n'ont pas d'effet direct sur la normalisation.

Präsident Begleitkommission:

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Unterschrift Präsident Begleitkommission:

Michael Bernhard

Index des rapports de recherche récente en matière de route

Bericht-Nr.	Projekt Nr	Datum	Tite
1323	VSS 2008/205	2011	Ereignisdetektion im Strassentunnel
1323	VSS 2008/205	2011	Ereignisdetektion im Strassentunnel Détection d'incidents dans les tunnels routiers (Seulement disponible en allemand)
1326	VSS 2006/207	2011	Erfolgskontrolle Fahrzeugrückhaltesysteme
1326	VSS 2006/207	2011	Erfolgskontrolle Fahrzeugrückhaltesysteme Contrôle de l'efficacité des dispositifs de retenue de véhicules (Seulement disponible en allemand)
1328	VSS 2005/302	2011	Grundlagen zur Quantifizierung der Auswirkungen von Sicherheitsdefiziten
1328	VSS 2005/302	2011	Grundlagen zur Quantifizierung der Auswirkungen von Sicherheitsdefiziten Principes pour la quantification des effets des déficits de la sécurité (Seulement disponible en allemand)
1331	VSS 2005/501	2011	Rückrechnung im Strassenbau Analyse inverse pour la construction routière (Seulement disponible en allemand)
1331	VSS 2005/501	2011	Rückrechnung im Strassenbau
1332	VSS 2006/905	2011	Standardisierte Verkehrsdaten für das verkehrsträgerübergreifende Verkehrsmanagement
1332	VSS 2006/905	2011	Standardisierte Verkehrsdaten für das verkehrsträgerübergreifende Verkehrsmanagement Standardisation des données de trafic pour gestion intermodale du trafic (Seulement disponible en version allemand)
1335	VSS 2007/502	2011	Stripping bei lärmmindernden Deckschichten unter Überrollbean-spruchung im Labormaßstab. Désenrobage des enrobés peu bruyants des couches de roulement sous sollicitation de roulement en laboratoire (Seulement disponible en version allemand)
1335	VSS 2007/502	2011	Stripping bei lärmmindernden Deckschichten unter Überrollbean-spruchung im Labormaßstab.
1343	VSS 2009/903	2011	Les technologies de base pour l'enregistrement automatique des usagers de moyens de transports (Seulement disponible en version allemand)
1343	VSS 2009/903	2011	Basistechnologien für die intermodale Nutzungserfassung im Personenverkehr
1344	VSS 2009/709	2011	Initialprojekt für das Forschungspaket "Nutzensteigerung für die Anwender des SIS"
1344	VSS 2009/709	2011	Projet initial pour le paquet de recherche "Augmentation de l'utilité pour les usagers du système d'information de la route" (Seulement disponible en version allemand)

1334	ASTRA 2009/009	2011	Was treibt uns an? Antriebe und Treibstoffe für die Mobilität von Morgen
1334	ASTRA 2009/009	2011	Was treibt uns an? Antriebe und Treibstoffe für die Mobilität von Morgen Transports de l'avenir ? Moteurs et carburants pour la mobilité de demain (seulement disponible en version allemand)