

**Traffic Safety Analysis for Urban Highway Ramps and
Lane-Change Bans Using Accident Data and Video-Based
Surrogate Safety Measures**

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Abstract

The purpose of this study was to evaluate the traffic safety of urban highway segments near exit and entrance ramps in the Montréal metropolitan area. The city's tight urban environment has resulted in the construction of sub-standard highway ramp merging sections (e.g. short merging lengths, inadequate visibility, influence zone overlap, etc.). In order to mitigate safety problems associated with these inadequately designed features, a special lane-change ban treatment (technical designation LCGV1) was implemented several years ago at various ramps. This study used accident data and video-based surrogate safety measures to evaluate the safety effectiveness of the treatment.

The cross-sectional accident analysis controlled for factors such as lane configuration, merge length, traffic flow and speed, area of influence overlap (inter-ramp distance), lane and shoulder widths, horizontal and vertical curves, and covered the presence of the treatment across 10 years of accident data at multiple sites along Montréal's busiest highways. The time-to-collision conflict measure obtained from automated video-based vehicle trajectory extrapolation was analyzed and used to identify microscopic behaviour patterns and conflicting interactions.

The study generally concludes that, across all sites, the presence of the treatment has led to no appreciable change in accident rate and that other contributing factors have played a greater role in observed accident rate, time-to-collision distribution, and lane changes. However, the study also indicates that there was significant variation between contributing factors across all analysis sites, leading to the conclusion that adopting a general policy of treating an entire urban region is a futile exercise. In addition, it was observed that the treatment has had a slight accident migration effect. These conclusions lead to the recommendation that the treatment should be applied on a case-by-case basis only, and otherwise that the default case (no treatment) should remain in effect so as not to hinder the normal navigation and operation of highway drivers.

Résumé

Le but de cette étude était d'évaluer la sécurité routière à la hauteur des entrées et des sorties des autoroutes urbaines dans la région métropolitaine de Montréal. Certains sites de cette région comportent des problèmes de conception au niveau de la sécurité routière (notamment une longueur de diffusion trop courte, une visibilité d'approche insuffisante, des zones d'influences se chevauchant, etc.). Afin d'atténuer les problèmes de sécurité associés à ces caractéristiques, un traitement spécial interdisant les changements de voie (désignation technique LCGV1) a été mis en place à ces bretelles. Cette étude vise à évaluer l'efficacité du traitement à partir des données d'accidents et d'une analyse du comportement des usagers à partir des trajectoires des véhicules telles que recueillies depuis des données vidéo.

Les facteurs de contrôle de l'analyse transversale comprennent la configuration des voies, la longueur de diffusion, le débit et la vitesse, le chevauchement des zones d'influences (distance des échangeurs), la largeur des voies et des accotements, les courbures horizontales et verticales, et la présence du traitement. L'analyse se fait avec dix ans de données d'accidents à plusieurs sites le long des autoroutes les plus fréquentées de Montréal. L'analyse du comportement des usagers étudie le *time-to-collision* (TTC) entre les trajectoires des usagers extrapolés depuis des données vidéo et elle identifie les interactions de conflit microscopiques.

L'étude conclut généralement que, pour tous les sites, la présence du traitement a conduit à aucun changement appréciable des taux d'accident et que d'autres facteurs ont un effet plus important sur le taux d'accident observé, la distribution du *time-to-collision*, et les changements de voie. Cependant, on a constaté qu'il y avait des variations importantes de facteurs de site en site, menant à la conclusion que l'adoption d'une politique générale de traitement n'est pas justifiée. En outre, on a remarqué que le traitement avait un léger effet de migration d'accidents. Ces conclusions ont conduit à la recommandation que le traitement soit appliqué au cas par cas seulement.

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Chapter 1: Introduction

1.1 Introduction

The objective of this study is to conduct a road safety study to investigate the safety effectiveness of the pavement marking treatment defined as the “ligne-de-délimitation-continue-à-gauche-de-la-voie-1” (technical designation: LCGV1). This treatment, a type of lane-change ban, bans lane changes from the inside lane(s) towards the first lane (the outermost lane, excluding merging and auxiliary lanes), but permits lane changes in the opposite direction. This treatment uses a painted double-line lane marking between the first and second lanes: the painted line closest to the second lane is continuous while the painted line closest to the first lane is dashed.

The treatment is designed to encourage drivers to merge into the first lane early, in anticipation of exits (although it should be noted that many exits are found to have a treatment extending well beyond the merging area), or to shield users entering highways from users on the inside lanes, although it does not prohibit drivers already in the first lane from occupying it. Diagrams of the treatment and associated design behaviour are depicted in Figure 1. A detailed discussion of the theorized mechanisms behind the treatment is presented in section 1.3.

This dossier goes back more than 20 years, probably to around the year 1987. The treatment was first proposed, evaluated, and used at a unique site: the exit ramp of Christophe-Colomb on Autoroute 40 in Montréal. It was implemented with the idea of helping road users to merge where merging time was limited due to a short exiting ramp length and poor ramp visibility. The treatment then proliferated to other short entrance and exit ramps on the elevated portion of the six-lane Autoroute 40, “Autoroute métropolitaine,” in Montréal. This was again in response to substandard merging and exiting conditions on this highway. Yet the treatment was still not standardized. Up to now, no studies have been done on the effectiveness of this pavement marking.

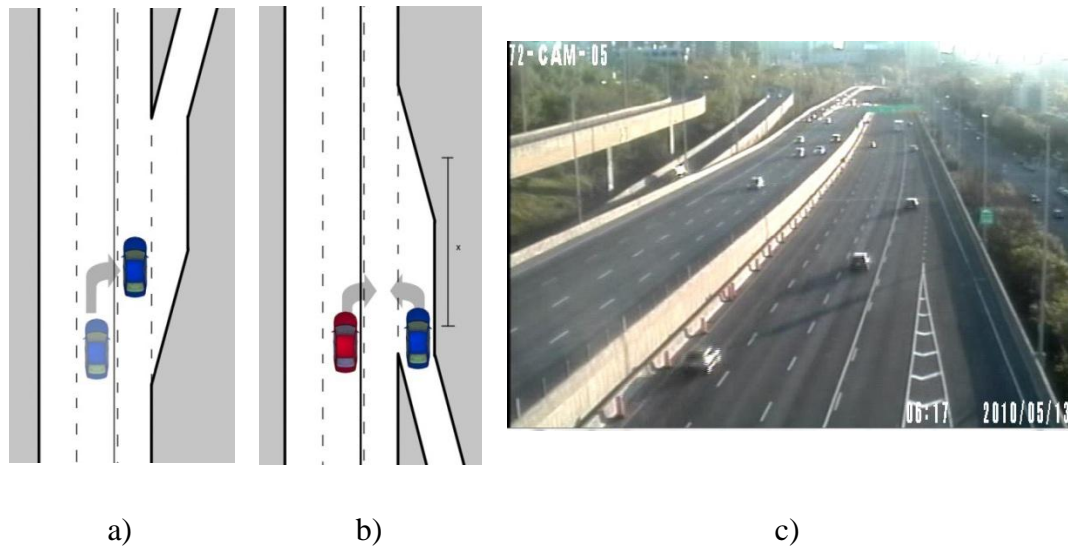


Figure 1 – Typical LCGV1 design. a) Treated highway exit, discouraging last-minute lane changes by preventing road users from exiting from the middle (second) or inside lanes. b) Treated highway entrance, encouraging merger into the first lane by ramp users only. c) Treated highway; Autoroute 720 “Ville-Marie Expressway” eastbound, entrance 3, “Avenue Green”, view from Cam720-05-eastbound.

In Québec, the treatment has since proliferated to other sites, including entrances. The problem is that the application of the treatment to other sites has never been formally evaluated for its safety merits. Furthermore, there is no standard design placement of the beginning or the end of the line, nor is there a standard formula or fixed distance defining the design distance of the treatment. It generally covers all the merging length for both entrances and exits and often extends well beyond the merging zone. In fact, some consecutive treatments are so long that they overlap; others leave less than 50 m of untreated length. Some highway sections have a majority of the road length covered by the LCGV1 treatment, leading some to criticize its usefulness and question whether restricting too many lane changes is detrimental to both traffic fluidity and road safety. Further questions have been raised as to whether the treatment is violated by road users, the practicality of its enforcement, and whether the treatment confuses drivers unfamiliar with the location.

Although each highway site is referenced by its associated interchange (ramp), the particular scope of study for each site is the section of highway

immediate the ramps and not the ramps proper, where the treatment is or would otherwise be located. This region is termed the ramp influence area, is generally 450 metres in length, and is already documented by the FHWA Highway Capacity Manual (HCM) 2000 (see section 1.3 for details).

The notation for highway ramps used in this study follows the format A20-E-E56-3, where the first letter denotes the highway system (A = autoroute, I = interstate, etc.), the following numbers denote the highway number, followed by a dash, one letter denoting direction (E = east, W, or O = west, N = north, S = south), another dash, one letter denoting the ramp type (E = entrance, S = exit), followed by numbers representing the highway stationing, and finally, an optional number representing the downstream order in which a ramp appears if a single interchange has more than one entrance or exit.

Section 1.2 will deal with the objectives of this study in response to the debate over the usefulness of the treatment, while section 1.4 will summarize previous research on the matter. Chapter 2 will cover accident, geometry, and flow data collection, while chapter 3 will use statistical methods to analyse it and provide results. Chapter 4 will cover video data collection methods and inventory, while chapter 5 will cover video data analysis methods and results. Chapter 6 will address design recommendations according to the results found in chapters 3 and 5.

1.2 Document objectives

The purpose of this document is to answer the questions formulated in the Ministère des Transports du Québec (MTQ) research project entitled “*Étude concernant l’utilisation d’une ligne continue de délimitation à gauche de la voie 1 (LCGV1) vis-à-vis les entrées et les sorties d’autoroutes R 678.1*”. These objectives are: (i) to study the LCGV1 treatment’s effectiveness from a traffic safety point of view, and (ii) determine whether it should be standardized as a general treatment for highway ramps in Québec.

The following hypotheses are formulated for testing:

- The treatment should reduce the overall accident probability inside the ramp influence area as measured by accidents-per-million-vehicle-kilometres after controlling for cross-sectional factors.
- The treatment should be obeyed by a certain measurable percentage of drivers: the simplest test is to observe compliance of road users. Although compliance with the treatment is not a direct measure of increased road safety, evidence that drivers do not comply with the treatment at all might lead to immediate conclusions about the usefulness of the treatment (testing the negative).
- The treatment might cause a migration of accident location as well as type: lane-change closures may prevent merging action along the treatment but, in so doing, may also concentrate the remaining merging action just before or after the treatment. Any increase in local conflict intensity might disproportionately lead to an increase in accident risk. Modifying interactions may also present an adverse effect on conflict types which may or may not pose a greater risk of accident.
- The treatment may have an equal effect for entrance and exit ramps alike.

The particular objectives of this study are:

1. To conduct a literature review related to highway safety at ramps, with a particular focus on pavement marking strategies.
2. Based on historical accident data, develop a cross-sectional accident frequency analysis in order to investigate the effect of the LCGV1 treatment on accident occurrence.
3. Based on video and conflict analysis, study the effect of LCGV1 treatment on driver behaviour and safety by comparing highway ramps with and without this treatment.

The study develops and uses two separate and complementary methods in order to answer these questions. Originally, the traditional historical analysis had been considered as the main avenue of research. However, due to a lack of historical data, notably, dates and design specifications for the original application

of each treatment as well as geocoded accident data, the historical analysis was replaced with cross-sectional and video analytics analysis methodologies, each with their advantages and disadvantages. A single site had some before-treatment and after-treatment video data available and was consequently studied in a before-after manner as well.

The study evaluates changes in traffic flow and behaviour only to the extent that it is needed to answer the question of traffic safety. While effects of the treatment on traffic flow may exist and may influence choice of design further; it is not explicitly explored in this document.

1.3 Treatment design and ramp elements

Before we begin analysing the LCGV1 treatment, it is important to have a grasp of the design principles and elements behind the design. Despite the lack of standardization, and a rather simple appearance on the surface, it is effectively a complicated road element, particularly vis-à-vis its placement with respect to other road elements.

Again, the LCGV1 was initially designed for A40-E-S73 (Cristophe-Colomb, Autoroute 40), a viaduct exit with a particularly short merging ramp and poor signage, in an attempt to prevent vehicles from merging into the outside lanes too late. Under pressure from various agencies, the treatment began being applied to other exits, as well as entrances, without thorough analysis. The issue lies in the fact that the treatment's design was never thoroughly evaluated for any other site, particularly entrances.

To understand the reasoning behind the original treatment design, we must study the behaviour of traffic inside the various regions of influence, as defined both by the HCM 1985 (design guide used when the treatment was first proposed) and the HCM 2000, and by the regions delimited by the treatment. The reader is

also invited to consult the original MTQ report outlining the debate regarding treatment's origin, design, and standardization (1).

Figure 2 illustrates the sub-regions of a treated ramp as delimited by changes in the ramp's geometry, including the treatment. Each treated ramp has a total of five zones. Untreated ramps are not distinguished between protected and unprotected areas and therefore have a total of three zones.

In the pre-exiting zone, upstream of exit ramps, vehicles must actively make navigation decisions and control operations to enter the first lane in preparation for exiting manoeuvres. The LCGV1 always begins in this zone if it is not continuous with a previous upstream ramp. According to the HCM 1985, the exit ramp influence area starts in this area, 762 metres upstream from the exit proper (although the centre point of the ramp is defined ambiguously). In the HCM 2000, this distance was shortened to 450 metres. In the exiting zone, exiting vehicles merge into the exiting ramp or auxiliary lane, which eventually leads to the exit of the highway itself. Some vehicles in the second or third lanes of the exiting zone may also attempt last-minute lane changes in an effort to reach the exiting ramp or auxiliary lane. This type of behaviour is generally discouraged as per the objective of the original treatment design. The post-exiting zone is characterized by a flow equal to or lower than the upstream zones, and the LCGV1 often extends uselessly well into this section, despite most exiting manoeuvres having ended. On occasion however, the painted chevrons are used by drivers to make last minute exits. The HCM 1985 extends the ramp influence area 152.4 metres downstream of the ramp into this area and might be a possible explanation for the original design decision to extend the treatment this far beyond the exit. The HCM 2000 eliminates this distance altogether.

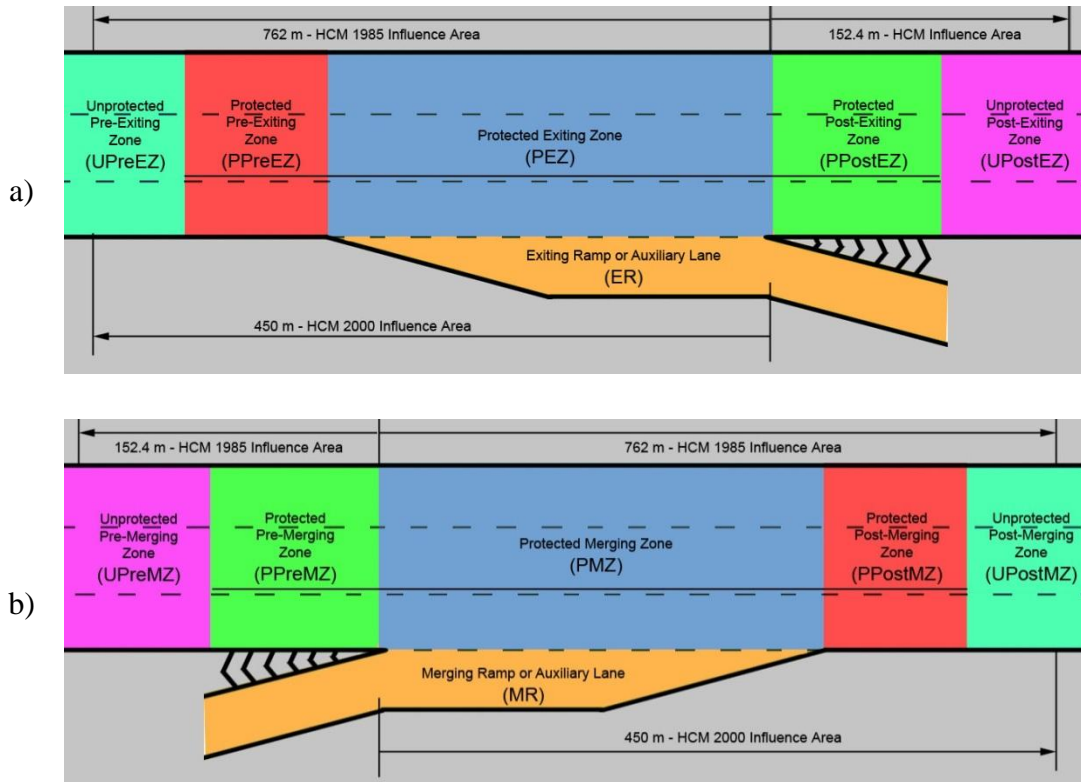


Figure 2 – Sub sections as delimited by ramp geometry and treatment: a) exit ramp; b) entrance ramp. In the case of untreated sites, there are no protected zones, with the Unprotected Exiting Zone (UEZ, not shown) and Unprotected Merging Zone (UMZ, not shown) corresponding to the PEZ and PMZ respectively.

Entrances are slightly more complicated. In the pre-merging zone, upstream of entrance ramps, vehicles make their approach to the ramp. In this area, drivers of vehicles in the first lane must start looking for potential conflicts with drivers attempting to merge. For this to happen, direct line-of-sight visibility is of great importance. Although, under Québec law, merging vehicles must yield to highway vehicles, it is common courtesy and safe driving procedure for highway vehicles to yield to or assist merging vehicles where necessary. The HCM 1985 begins the entrance's influence area 152.4 metres upstream of the entrance, while the HCM 2000 does not begin the influence area until the start of the entrance ramp. In the merging zone, merging vehicles must make use of the first lane in order to enter; it is therefore of interest to keep this lane clear and, particularly, to prevent drivers in the second or third lanes from merging into the first lane. The post-merging zone offers an area for drivers to settle into adequate following-

distances, and the HCM 1985 extends the ramp influence area by 762 metres into this area. The HCM 2000 uses a distance of 450 metres downstream of the entrance ramp instead. Of special note is the habit of some drivers to merge past the exiting zone from within the shoulder, possibly due to difficulty in finding an opening in traffic or due to late acceleration. It is also not infrequent to witness drivers merging very early, e.g. over the chevron region.

Current design equations assume adequate influence area. When the influence area of a ramp overlaps with another one, driver behaviours are expected to become quite complex, with flow patterns depending greatly on the nearness of the two ramps, relative flows, and ramp types. Regarding the treatment, this has been a source of much criticism, particularly since some highway segments have such a high interchange density that treatment lines extend along almost, if not all, the entire length of highway, banning all lane changes into the outside lanes over one or more kilometres. HCM 2000 only covers specific ramp orders through a set of volume predicting equations. Other than ramp distance and ramp merging length in the prediction of traffic volumes, few ramp design measures are published.

A thorough analysis of the geometric measurements and features of highway ramps is presented in section 2.2.

1.4 Literature review on traditional versus surrogate safety analysis

The two most popular safety analysis approaches are the traditional approach, based on accident analysis, and the surrogate approach, based on conflicts analysis. These two approaches are explored and then applied as complementary analyses for the study. The strengths and weaknesses of each are discussed below.

The traditional safety approach, based on the statistical analysis of historical accident data, has a long history. In fact, most of the current literature covering

highway safety analysis, including ramp and pavement marking safety studies, has been conducted using traditional methods. These include the classical before-after observation study and the cross-sectional study based on regression techniques. Despite the robustness of this established technique, some disadvantages relevant to this project should be mentioned (2) (3), the primary one being accuracy problems stemming from incomplete and inconsistent accident reports. This includes missing or estimated geographical locations of accident reports and human reinterpretation during database compilation. For instance, until recently, police reports in Québec did not provide geographic coordinates of accident locations. Furthermore, despite a uniform reporting form, many reports are completed with varying levels of detail or may omit important information in retrospect: any resulting holes must be reinterpreted by data compilation staff. Other issues include under-reporting of accidents (particularly minor ones) and limitations in the design of the original accident reporting form (insufficient detail in areas such as environmental, geometric, and weather conditions).

Although surrogate safety analysis is less common, it is not a new subject of research. Many papers in road safety have argued for and against the use of conflict analysis as a reliable safety measure, from the standpoint of both collision severity and collision frequency. The reader is invited to consult (2), (4) and (5) for a detailed summary of the results of conflict studies. A recurrent argument against conflict studies in transportation safety is the difficulty in obtaining quantitatively defined and objectively measured data and the fact that the application of the methodology is often too broadly defined.

Lately, however, obtaining objective data for surrogate safety analysis is becoming more feasible with advances in video tracking algorithms, increased access to more affordable processing power, increased data management, and emerging transportation information technology systems. The use of video analysis for transportation studies is rising dramatically. Many traffic behaviour studies have been conducted around data collected from cameras, e.g. Sarvi et al.

(6); video data has been used for simulation calibration and traffic flow theory, e.g. the NGSIM program (7); and some companies now offer automated traffic counting solutions using video detection. Automated video analysis for conflict analysis has been developed and used extensively by Sayed, Saunier and Ismail primarily for road safety analysis at intersections, including vehicle conflicts (8) and pedestrian-vehicle conflicts (3) (9) amongst others.

There is a growing interest in computer vision for automated traffic video analysis, a key element in the development of accurate traffic measurements, which allows for the acquisition of multiple traffic data along road sections. Versavel lists volume, speed, density, headway, and location as the primary traffic data; and counts, speed (acceleration), vehicle length, class, type and position as the individual vehicle data (10). The reader is invited to consult (11), (12) and (13) for more detailed information on the specifics of feature-based vehicle tracking.

Supported by previous studies (8) (3) (9), the main benefits of automated video analysis for safety are two-fold: firstly, it offers a convenient, low-cost method of calibrating driver behaviour parameters for specific roadway types and regions (without the need to install intrusive monitoring equipment), and secondly, it provides a flexible tool for complex driving behaviour analysis, particularly useful for microscopic road safety analysis using surrogate safety approaches.

However, video analytics is not without limitations such as the complexity of computer vision algorithms, the sensitivity to field of view and visibility, and individual vehicle tracking problems in high-density flows. Measurement accuracy is highly dependent on the quality of the camera installation as well as flow conditions; to this end, mobile hardware for video data collection operations is still under development, in particular for data collection at freeways. Weather conditions (i.e. visibility), obstacles (i.e. road signs, posts and overpasses), camera field of view and angle, curved roadway sections, and occlusions from dense traffic and large vehicles (i.e. trucks) are all potential sources of tracking

problems. Higher accuracies can be achieved by limiting these line-of-sight issues. Of course, many of these limitations are not specific to any particular trajectory-tracking technology, nor to human observers. Given ideal conditions, the practical rated accuracy of traffic detection by means of automated video analysis is in the 95-99 % range for basic recognition (10). Performance measures for tracking algorithms, however, are less clearly defined and make results from different systems difficult to compare. Automated analysis also has the significant advantage of having no loss of attention or error in judgement, and the ability to consistently measure velocity and position. Finally, video analysis may be used as an assisting tool, where large amounts of video data are filtered automatically to be reviewed by traffic and safety experts.

Despite important developments in the field of surrogate safety analysis in the last few years including a stronger push for practical guidelines (14), some gaps in the literature still persist. Among these we can mention the fact that few before-after studies have been done using video analytics and conflicts analysis. Moreover, many studies in recent years have concentrated mostly on conflict analysis at urban intersections and pedestrian safety.

The origin of the algorithms used for conflict analysis is quite elaborate and will be further developed in chapter 5.

Chapter 2: Site selection and data acquisition

Chapter 2 presents the site selection and data acquisition methods and considerations. The data constraints and limitations of this approach are also highlighted. For the selected sites, a data inventory is collected containing the potential contributing factors affecting the safety status of the road elements under analysis to control for in the cross-sectional analysis. This is done according to the highway safety literature, and typical contributing factors associated with accident occurrence including geometry characteristics and traffic conditions. Historical accident data is then used to investigate the contributing factors associated with accident occurrence, in particular, the effect of the LCGV1 treatment. The historical accident data comes from two different sources, and each data set presents advantages and disadvantages to be considered.

2.1 Site selection for accident data analysis

The selection of appropriate and representative sites is governed by a set of constraints. The sites (ramps) should be located on the Island of Montréal, given that this is the environment of study. Moreover, by selecting sites from the same city, we ensure that the cultural or regional differences in road design and driving behaviour are controlled. Furthermore, obtaining comparable data outside the main study area is logistically challenging. For the selection of sites, the following criteria are considered:

- *Environment*: Urban/semi-urban environment on the Island of Montréal.
- *Geometric configuration*: A lane configuration of three primary lanes and one merging lane, auxiliaries to be avoided, minor junctions only.
- *Major highways*: 80,000+ AADT/direction, 60-100 km/h min-max posted FFS, regular a.m./p.m. peak flow.
- *Traffic flow and accident data*: The sites considered in this project are those with available microscopic flow data and accident data.

- *Sample balance:* Equal amount of entrances and exits where both before-treatment and after-treatment data can be collected. Otherwise, a random selection to minimize the effect of selection bias.

As noted in the last point, it is desirable to avoid selection bias through random selection. Of course, the pool is not perfectly representative of every site, as sites with missing data cannot be included in the pool. It is therefore assumed that sites eliminated due to missing data are eliminated purely out of logistical requirements and do not bias the remaining sites in terms of factors. Sites eliminated by the geometric consistency criterion are not subject to the outcome of the study in any case, and so can be safely ignored. Figure 3 crudely illustrates constraints.

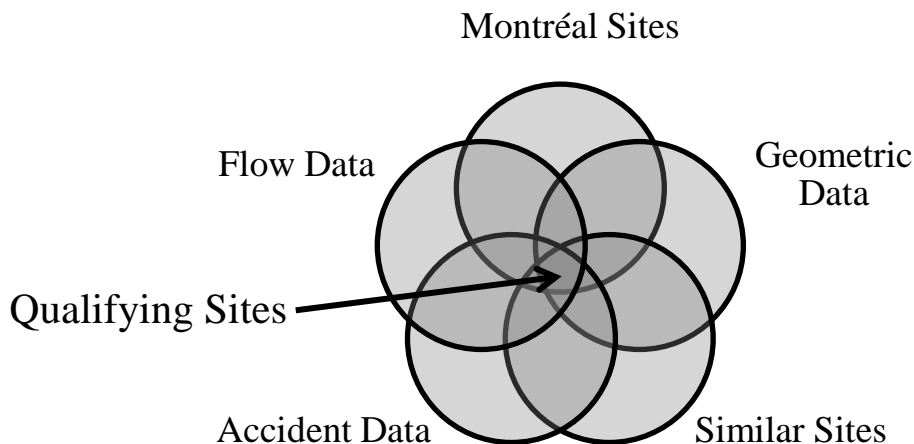


Figure 3 – Illustration of site and data availability. Selected sites are randomly and equally distributed among qualifying sites for selection.

The sources of data for road geometry, traffic, and accident history are:

- *Geometric data:* This is publicly available through services such as Navurb.com, Google Maps, Google StreetView, or stock satellite imagery. Measurement errors range from 10-400mm. See section 2.2.

- *Traffic flow data*: For some sites, traffic data was available from loop-detector. For another set of sites, data was manually collected using videos or field visits (with observers). The loop-detector or manual counts were expanded using nearby known highway-section-specific (no more than 2-3 kilometres away) expansion factors to produce AADT. For each ramp, as explained later, the upstream flow, downstream flow, and ramp flow was estimated. See section 2.3.
- *Accident history data*: This comes from provincial police reports or paramedic services; supplied by the SAAQ and Urgences-santé respectively. See section 2.4.

Table 1 lists ramp counts of all limited-access highways on the Island of Montréal. Ramps (minor junctions) are distinguished from major junctions in that the number of lanes upstream and downstream of ramps remains the same, whereas the number of lanes upstream and downstream of major junctions does not stay the same. Left-hand entrances are special-case entrance ramps located on the left (inside) portion of the highway segment. See section 2.2 for more details. It should be noted: although Autoroute 40 is listed as having the lowest average density, it does have the highest variability in density, with very low ramp densities in the east and west-ends of Montréal, and the highest ramp density on the elevated “Métropolitaine” section.

Table 1 – Montréal ramp totals, by highway stretch and ramp type.

| Autoroute | Ramps | Major Junctions | Left-hand entrances | Total | % All | Density |
|------------------|---------------|------------------------|----------------------------|---------------|--------------|----------------|
| 20, 720 | 65 | 14 | 1 | 80 | 25.32% | 1.09/km |
| 40 | 87 | 18 | 0 | 105 | 33.32% | 1.00/km |
| 520 | 23 | 0 | 0 | 23 | 7.28% | 1.58/km |
| 13, (138) | 25 | 5 | 0 | 30 | 9.49% | 1.31/km |
| 10, 15, (117) | 39 | 11 | 2 | 52 | 16.46% | 1.21/km |
| 25 | 13 | 12 | 1 | 26 | 8.23% | 1.55/km |
| Total | 252 | 60 | 4 | 316 | | |
| Mean | 42 | 10 | 0.667 | 52.667 | | 1.29/km |
| % All | 79.75% | 18.99% | 1.27% | | | |

Table 2 lists the population and subpopulation of sites by category as each criterion is applied. There are currently 252 limited-access highway ramps (minor junctions) on the Island of Montréal. Of these sites, 247 have easily accessible geometric data (tunnels hide underground highway segments from satellite imagery view). Of these sites, 185 are removed as they have consistency issues or other data accessibility issues (flow volume and polarization, geometric outliers, etc.), leaving 62 sites. Of these sites, 47 have detailed flow data and are kept. Of these sites, 23 have accident data from the SAAQ (mediocre location data but detailed reports) and 47 have accident data from Urgences-santé (reliable location data but fewer details).

Table 2 – Data availability after intersection.

| Criterion | Intersection with | Number of available sites |
|--------------------|--------------------------|----------------------------------|
| Montréal | - | 252 |
| Geometry data | Montréal | 247 |
| Site similarity | Geometry data | 62 |
| Flow data | Site similarity | 47 |
| SAAQ accident data | Flow data | 23 |
| Urgences-santé | Flow data | 47 |

2.2 Geometric design variables

Figure 4 illustrates typical geometric design elements for highway merging sections. These elements are chosen as part of the geometric design inventory used in this study and according to highway flow theory, highway safety theory (HCM 2000), and observations made in (1). minor junctions keep a constant number of primary lanes (numbered in ascending order from outside inwards) upstream and downstream of any ramp, while major junctions result in fewer lanes downstream of exit ramps and additional lanes downstream of entrances. In the case of major exits, the upstream outside primary lanes turn into exit ramps and, in the case of major entrances, the entrance ramps become primary outside downstream lanes. These types of lanes do not merge at all. All minor junctions employ a merging lane which starts at b) for exit ramps or ends at e) for entrance

ramps. Some major junctions (exits mostly) might employ the use of additional merging lanes to accommodate ramp queuing in order to prevent any queues from spilling over onto the highway. Some designs also use short auxiliary lanes which connect an entrance to a nearby downstream exit, causing a temporary increase in the number of lanes of the highway. Auxiliary lanes differ from unmerged exiting or entering lanes at major junctions in that they are never occupied by users who are not involved in highway entering or exiting manoeuvres (HCM 2000).

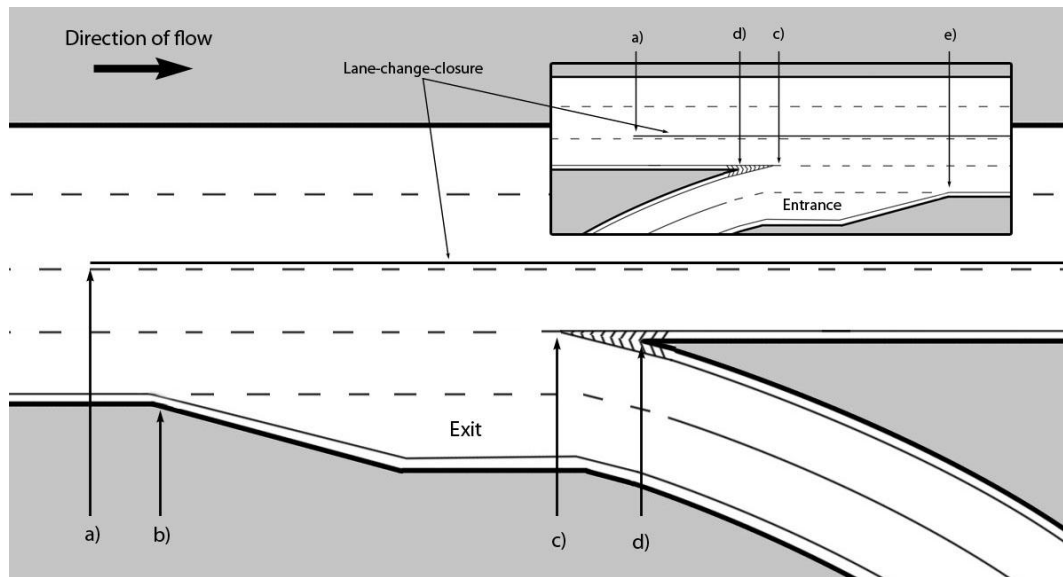


Figure 4 – Diagram depicting elements present in a typical exit or entrance ramp. a) lane-change closure start, b) merge-lane start (exit ramp only), c) painted pierhead, d) physical pierhead, e) merge-lane end (entrance ramp only).

All studied sites involve minor junctions with right-hand on-ramps. There are a few cases of left-hand on-ramps in Montréal, but they are mostly untreated (the treatment design would differ appreciably in any case). Left-hand on-ramp design is further covered in design literature (HCM 2000).

In this study, point c) of Figure 4 is used as the geometric centre of any ramp, and all subsequent distances are offset from this location. For design and accident analysis purposes, offset distances are measured in parallel and along with the road alignment/curvature, with negative distances representing an upstream (counter-flow) direction and positive distances representing a downstream (with-flow) direction. For geographic purposes, distances are given

in the Cartesian coordinate system after UTM zone-8 projection. Horizontal curves are defined by their radii. Positive horizontal radii define curves turning left from the driver's perspective. Vertical curves are defined by their approximate slope in degrees with positive horizontal slopes representing upwards sloping.

A non-exhaustive list of points of interest of ramps is illustrated in Figure 4. This includes a) lane-change closure start, b) merge-lane start (exit ramp only), c) painted pierhead, d) physical pierhead, e) merge-lane end (entrance ramp only). Design distances between these points constitute a portion of the geometric design variables that were generated for this study and are listed in Table 3 and illustrated in Figure 5.

Table 3 – Geometric design variables present in a typical exit or entrance ramp.

| Variable Name | Description | HCM 2000 Equivalent (25-1) |
|---------------------------|--|-----------------------------------|
| <i>l_diffuseur</i> | Merge length | L_A/L_D |
| <i>l_visibilite</i> | Chevron-concrete pierhead distance | - |
| <i>d_bret_amont</i> | Upstream ramp distance | L_{up} |
| <i>d_bret_aval</i> | Downstream ramp distance | L_{down} |
| <i>b_voie1</i> | Primary lane width | - |
| <i>b_voie_bretelle</i> | Merge/auxiliary lane width | - |
| <i>b_accot_ext</i> | Outside shoulder width | - |
| <i>b_accot_bret_int</i> | Inside shoulder width, ramp | - |
| <i>b_accot_bret_ext</i> | Outside shoulder width, ramp | - |
| <i>no_voies_autoroute</i> | Number of primary lanes | Only considers lanes 1 and 2 |
| <i>no_voies_bretelle</i> | Number of ramp lanes (unattached) | Special case |
| <i>no_voies_non-dif</i> | Number of non-merged lanes (auxiliary or major junction lanes) | Exhibits 25-6 and 25-13 |
| <i>sh_merge_lane</i> | Number of auxiliary lanes (explicit) | Exhibits 25-6 and 25-13 |
| <i>r_v_curve</i> | Vertical curve grade (highway) | Descriptive |
| <i>r_h_curve</i> | Horizontal curve radius (highway) | Descriptive |
| <i>r_v_bretelle</i> | Vertical curve grade (ramp) | - |
| <i>r_h_bretelle</i> | Horizontal curve radius (ramp) | - |
| <i>d_treat</i> | Treatment length | - |

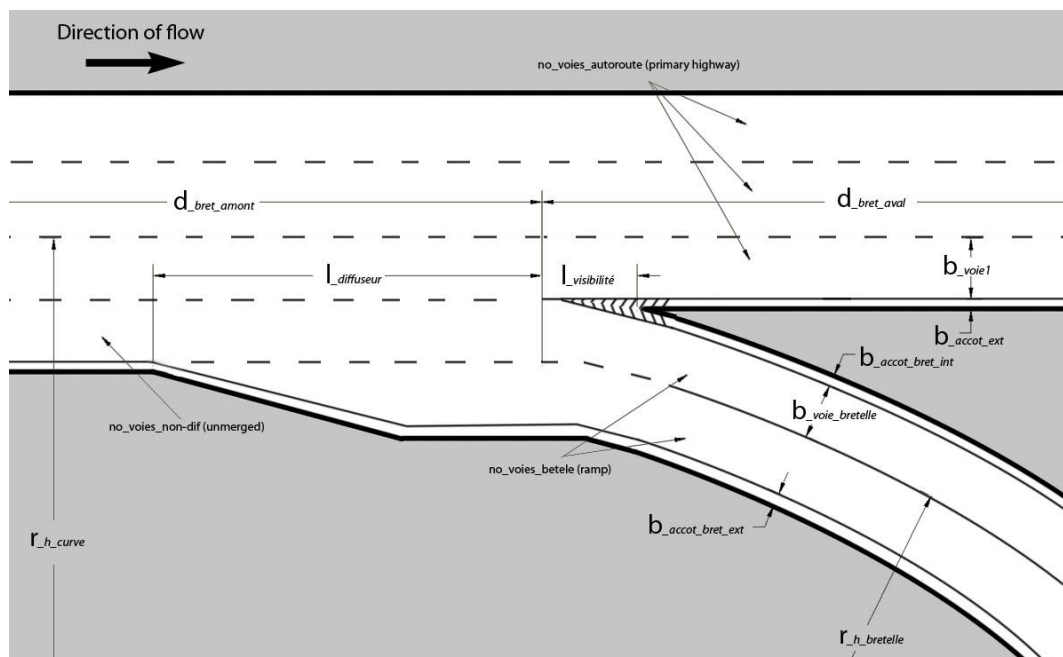


Figure 5 – Diagram depicting typical design measures for any type of junction. This illustration depicts primarily an exit, but can be flipped horizontally to represent the geometry of an entrance.

The merge length distance ($l_{diffuseur}$) is the road length drivers have for merging out of a primary lane for the first time (exit ramps) or the road length drivers have for merging into a primary lane for the first time. It is equal to the length of the auxiliary lane, if one exists. This variable is important, as it represents the distance, and thus the time according to speed, any entering or exiting vehicle has for a smooth merge manoeuvre. The chevron-to-concrete pierhead distance ($l_{visibilite}$) is the distance between the tip of the painted chevron pierhead and the tip of the concrete pierhead. In the case of exits, this area of road surface can be abused by drivers who make dangerous last-minute exiting manoeuvres. In the case of entrances, this area provides guaranteed visibility for entering vehicles looking for merging gaps, and for highway users looking to yield to or accommodate entering vehicles.

The upstream ramp distance (d_{bret_amont}) and downstream ramp distance (d_{bret_aval}) represent, from geometric centre to geometric centre, the distance from a particular ramp to the next upstream and downstream ramp, respectively. These variables, along with entrance/exit rate (see section 2.3), appear to strongly

influence lane changing density and, in the case of the upstream distance particularly, might also be an important factor in problems associated with spillover queues from downstream exit ramps.

It is important to discuss the typical ramp configuration found on Montréal highways with respect to ramp proximity associated with the city's more urban, dense interchange environment, as opposed to rural highway ramps. The average interchange distance (over/under-pass to over/under-pass) on Montréal highways is much smaller than that of rural highways (see Table 1), so much so that the inter-ramp distance becomes much smaller relative to the interchange distance, because most of the interchange distance is used up by the length of the ramps themselves. A few exceptions aside, the typical interchange configuration present on Québec highways follows the classic diamond configuration: corresponding exits are situated upstream of interchanges, corresponding entrances are situated downstream of interchanges. This configuration is used with overpasses and underpasses, viaducts, at-grade and sunken highways, and can be connected to the grid through assorted designs (e.g. service road, single-point diamond, roundabout). Because the dimensions of ramps are static and interchange density generally depends on local urbanization, distances between ramps on the Island of Montréal (particularly between ramps facing each other) shrink significantly in comparison to distances between ramps off the Island of Montréal, as well as the overall scale of the highway.

Figure 6 illustrates two near end-to-end diamond interchanges with relevant driving distances between ramp elements. Ramp lengths being relatively inflexible, the closer two interchanges are to each other, the smaller ramp-to-ramp inside and outside distances must be. As the ramp-to-ramp inside and outside distances shorten, design influence zones begin to overlap: in the typical case, merging behaviour overlaps with advanced exiting behaviour. For example, interchange distances of 1 km or less (consider four of the seven interchange distances of the elevated portion of Autoroute 40 "Metropolitaine" are below 1000 m with a combined average distance of 1182 m) with ramps 300-500 m in

length, including grid connections, leave very little room for ramp-to-ramp outside distances (sometimes as little as 300 m).

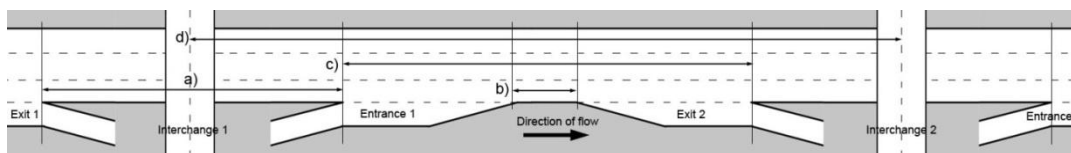


Figure 6 – Interchange and inter-ramp distances for any typical pair of interchanges using the diamond configuration: a) interchange inside ramp distance, b) ramp-to-ramp outside distance, c) ramp-to-ramp inside distance, d) interchange distance.

Given a sufficiently small ramp-to-ramp outside distance, most designs implement a continuous auxiliary lane combining entrances and exits facing each other. Though rather infrequent, this design treatment has been used on some portions of the elevated portion of Autoroute 40 as well as other areas of the Island of Montréal (example: A40-E-E75 followed by A40-E-S76). Nevertheless, this still does not change the fact that entering and exiting manoeuvres mix. Auxiliary lanes used in such a fashion are not to be confused with auxiliary lanes specifically designed as part of cloverleaf (or half cloverleaf) interchanges (example: A20-W-S50). It should be further noted that auxiliary ramps are not used exclusively in these roles.

Some basic widths are taken for the first primary lane (b_{voie1}), the average merging and/or auxiliary lane ($b_{voie_bretelle}$), the outside shoulder downstream of exits or upstream of entrances (b_{accot_ext}), the inside ($b_{accot_bret_int}$) and outside ($b_{accot_bret_ext}$) ramp shoulder. It should be noted that these widths, particularly those of the shoulders, are often quite irregular at a single site and not particularly practical to document or compare in great detail.

The number of highway lanes ($no_{voies_autoroute}$), ramp lanes ($no_{voies_bretelle}$), and auxiliary or major junction lanes ($no_{voies_non-dif}$) are recorded. It is expected that the number of lanes adds a certain degree of complexity to lane change activity simply because drivers have more choices to make. Generally, a combination of three highway lanes, one ramp lane, and no auxiliary lane on a minor junction is preferred, as this is the most consistent and

frequent combination of lanes and the one most closely associated with the original treated site.



Figure 7 – Autoroute 40 “Autoroute métropolitaine” eastbound, entrance 73, “Christophe-Colomb”, view from Cam40-11-westbound, with LCGV1 treatment. Horizontal and vertical curves can affect user visibility, particularly for entrance merging manoeuvres.

Horizontal ($r_h_curve/r_h_bretelle$) and vertical ($r_v_curve/r_v_bretelle$) curvature measures are suggested, as these variables might be associated with factors such as stopping sight distance (15) and visibility for any type of merging manoeuvres, and in particular, differentials in vertical curves between ramps and highways. Visibility is explored as an important factor in merging behaviour, both with respect to navigation (particularly for drivers unfamiliar with an area) and for manoeuvring in highway entrances. Ramp visibility for the purposes of navigation might also be affected by such factors as vertical signage and, in the case of the lane-change closures, horizontal signage (lane markings). Visibility measures have limited practical use, however, due to their complex and subjective nature. Figure 7 illustrates a large differential in vertical curvature between the ramp and the highway resulting in reduced merging visibility between the ramp and the

highway. This visibility should also depend on the type of vehicle and the skill or ability of the driver.

Highway speed is not explicitly tested. This is because coarse measures of speed (e.g. section speed as measured by the loop detector) already correlate directly with traffic flow, while average annual fine measures of speed (e.g. per lane, ramp acceleration/deceleration dynamics, spot mean speed) are not available. In any case, speed appears to correlate more with accident severity (see section 3.1).

2.3 Traffic flow at ramps

The basic principle of conservation of flow applies to the highways in this study. Obviously, the difference between the summation of input flows and the summation of output flows should be equal to the change in volume of the system. As an example, we cut the highway ramp geometry in such a way that we obtain a highway segment the size of the ramp's area of influence with a fixed volume equal to the summation of all vehicles inside this segment for any time t . All upstream flows are necessarily input flows and all downstream flows are necessarily output flows. The rate of flow for input and output traffic is typically counted over an hour comprising 15-minute segments. Furthermore, the change in system volume over $t > 15min$ is negligible (consider the LOS B average density in a 500 m long system to be 30 veh/km, representing 2% of the total volume in the span of 15 minutes; consider the LOS F average density in a 500 m long system to be 250 veh/km, representing 2.5% of the total volume in the span of 15 minutes). Therefore, the conservation of flow relationship effectively becomes an equilibrium between all input and output flow summations.

$$\sum q_{in} = \sum q_{out} \quad (1)$$

As shown in Figure 8, entrance ramps necessarily have two input flows ($q_{entrance}$, $q_{upstream}$) and one output flow ($q_{downstream}$). Similarly, exit ramps necessarily have one input flow ($q_{upstream}$) and two output flows (q_{exit} , $q_{downstream}$). The separation of flows between the ramp flow and highway flow in that direction is critical because this relationship is necessarily the primary indicator of site activity, unless a significantly more important site exists nearby, such that an important amount of activity spills over to the first site. The ratio between the two flows is called the **flow ratio** and is given by equations 2 and 3.

$$q_{\% \text{ exiting}} = \frac{q_{exit}}{q_{downstream} + q_{exit}} \quad (2)$$

$$q_{\% \text{ entering}} = \frac{q_{entrance}}{q_{upstream} + q_{entrance}} \quad (3)$$

In either case, the greater the ratio, the more merging activity a site must have, assuming a constant proportion of entering/exiting vehicles. This also means that for every ramp there are exactly three flows, and at least two flow-field measurements are required given that the third can be calculated from the law of conservation of flow. Furthermore, because there are no additional input or output flows between ramps, given any ramp A of any type, any ramp B, also of any type and immediately downstream of ramp A, must necessarily have an equal upstream flow as the downstream flow of ramp A.

For easy comparison between sites, and statistical analysis, all flows are converted from hourly counts to a single average annual daily traffic (AADT) measure using known expansion factors for nearby highway segments. Counts are also made per lane but usually combined afterwards into a single measure of flow per input and output; these counts do not offer much insight into lane change activity unless they are performed at regular intervals at standardized, regularly spaced locations (e.g. about 5 metres) along the highway, which they are not. As such, lane change data is only processed during video analysis.

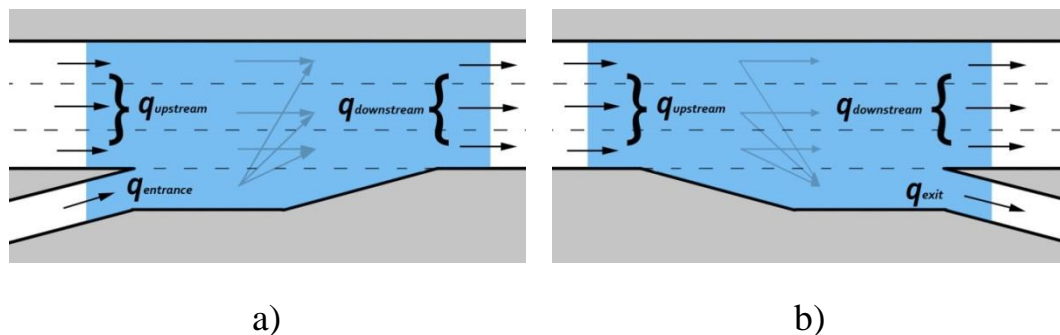


Figure 8 – Ramp flow arrangement diagrams: a) flows measured at an entrance, b) flows measured at an exit.

HCM 2000 primarily uses flows in the first and second lanes, as well as the ramp. It is, however, unknown whether the presence of the treatment affects the distribution of flows across all lanes (this is, after all, the desired immediate effect of the treatment) and so flows on all lanes must be considered.

2.4 Accident data: sources and spatial distribution

There are two historical accident data sources made available to the study.

The first source of historical accident data is the one managed by the SAAQ (Société de l'assurance automobile du Québec), supplied by the MTQ for the purpose of this project. It is built from accident reports completed by police officers responding to highway accidents and submitted to the SAAQ. The major problem with this data source is that police officers do not geocode their reports, meaning that locations of accidents must be guessed from often incomplete or illegible cross-street fields (highways do not have traditional cross-streets to begin with). The end result is that the database is not very reliable for the purpose of studying detailed geometric elements. In addition, data for many sites is simply not available, estimated accident location has an approximate error of about +/- 50-100 metres, and buffer zones are not explicitly defined. Furthermore, despite having exactly three consecutive years of accident data for each site, absolute reporting years are not the same for every site (3-year-reporting periods could be anywhere in-between 2002 to 2009). The advantage of this dataset is that it has a wealth of additional data such as collision type, road conditions, and severity, which may be useful on a macroscopic level. Some exploratory analysis is therefore conducted using the SAAQ data to help get a better profile of Montreal accidents in general.

The second source of accident data comes from Urgences-santé, the Québec agency that oversees paramedic services, and is also available for all highways of Montréal. This data set has complete coverage of all sites on the island with geocoding precision to about +/- 10 metres. This data set includes highway motor-vehicle accidents with 911 calls made from 1999 to 2008 inclusively (the caller's address is automatically sent to Urgences-santé and the location of the victim is validated over the phone; this location is instantly mapped in a GIS to dispatch and guide an ambulance) (16). Note that this dataset may not be very representative of accidents involving no ambulance dispatch or 911 call (e.g. property-damage-only). Despite this, as opposed to the SAAQ data, Urgences-

santé data seems to have less underreporting for accidents with injuries and misallocation problems.

In general, all historical accident data sources which rely on emergency services completing reporting forms share the common issue of subjectivity: it is safe to assume that emergency personnel aren't always equally concerned with making the same types of observations as accident reconstruction specialists or road safety engineers. Furthermore, different individuals from the same agency may provide varying levels of detail and focus when completing an accident report form.

Chapter 3: Accident analysis and results

This section presents the analysis and results of the traditional safety analysis based on the historical SAAQ and Urgence-santé data defined in section 2.4. Section 3.1 provides an exploratory analysis including potential accident migration according to relative distances from each ramp using both SAAQ and Urgences-santé data and section 3.2 covers a statistical analysis of accident frequency.

While the ramps themselves can be considered nodes, the ramps' area of influence can extend up to several hundred metres up- or downstream from entrances and exits (and arguably much further than that depending on the area of influence design guide used, e.g. HCM). Therefore, the accidents assigned to a given site (ramp) are those that occur within a chosen buffer zone (e.g., 200 or 400 metres) using the accident data collected by Urgences-Santé. Note that the SAAQ data was compiled externally and no explicit information regarding buffer size was provided. Furthermore, the lane at which the accident occurred is not reported in either data set. Therefore, a one-dimensional accident distribution was built for each site according to the alignment distance from the geometric centre of every ramp.

It is important to note that the directionality and distance to the ramp is very important. Because the geometry of a highway segment at a ramp can vary considerably over short distances, the cause of accidents located along different sub-sections can be attributed to different factors. Furthermore, shifts in distributions may provide evidence of any accident migration effects.

3.1 Exploratory Analysis

Accident data before-treatment is sparse and, in any case, no records indicating the date of application of the treatment exist. Therefore the problem has to be tackled by means of a cross-sectional analysis looking at all potential

accident factors. Figure 9 naively presents total accidents over a recent (2002-2009) three year period for selected sites for which SAAQ accident data is available. It is immediately clear that treated sites have more accidents than non-treated sites. The problem with this observation is that it is not possible to attribute accident causes to the presence of the treatment alone, even after controlling for flow volumes. In fact, it is very likely that the opposite is true: sites with the greatest amount of naturally-occurring accidents are the first sites to be selected for treatment by officials.

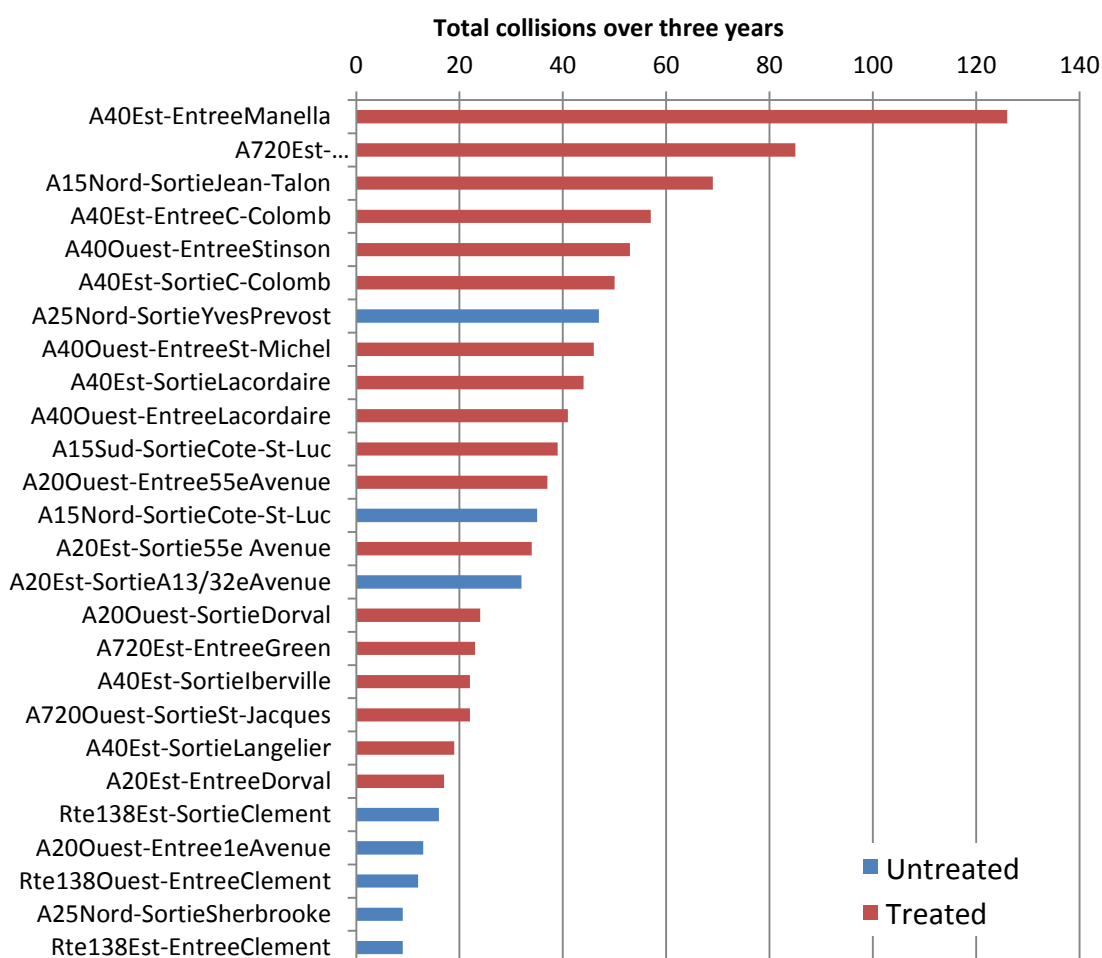


Figure 9 – Accidents over a three year period at selected sites. Data source: SAAQ.

Accident severity is briefly explored (major injuries, minor injuries, and property damage only) for both treated and non-treated sites. Figure 10 illustrates accident share by severity and treatment type according to accident data obtained

from the SAAQ reports. From this, one can see that accident severity levels do not appear to simply correlate with the presence of the LCGV1 treatment. Property-damage-only accidents of a value greater than \$1,000 represent a clear majority, while light and property-damage-only accidents under \$1,000 contribute minor and near-equal shares. Serious accidents (including deaths) account for a very small number of overall accidents.

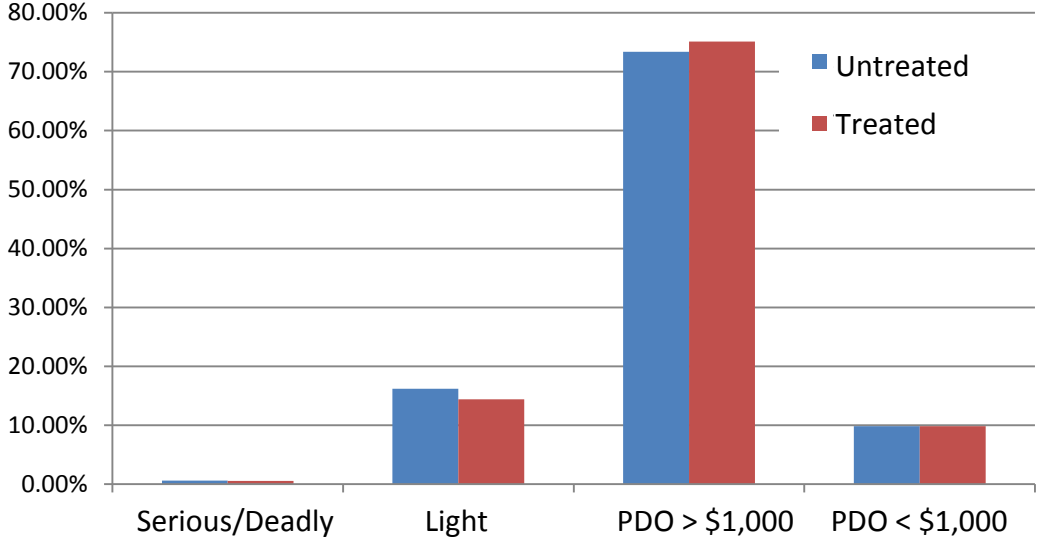


Figure 10 – Accident share by severity and treatment type. Data source: SAAQ.

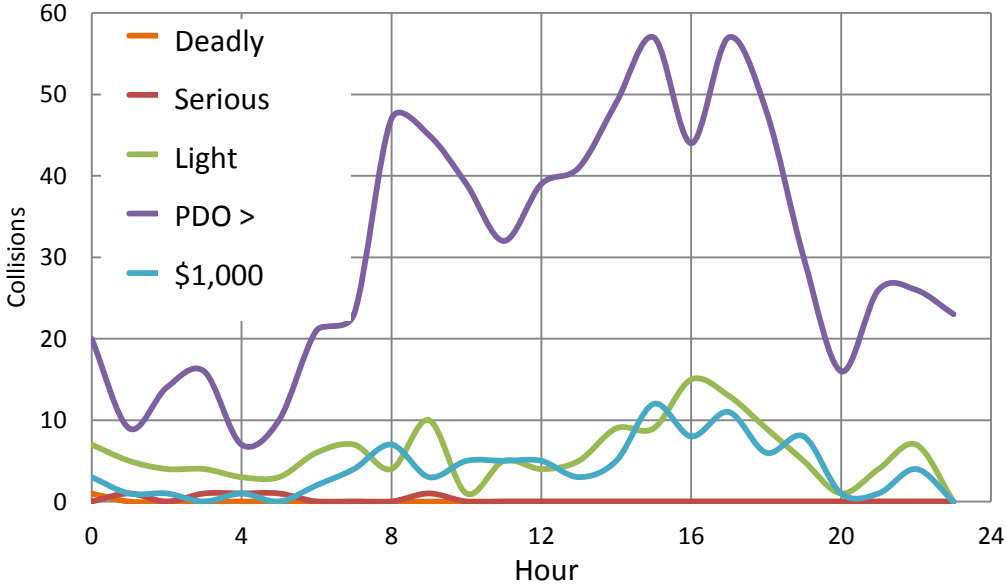


Figure 11 – Daily accident by hour according to severity. Data source: SAAQ.

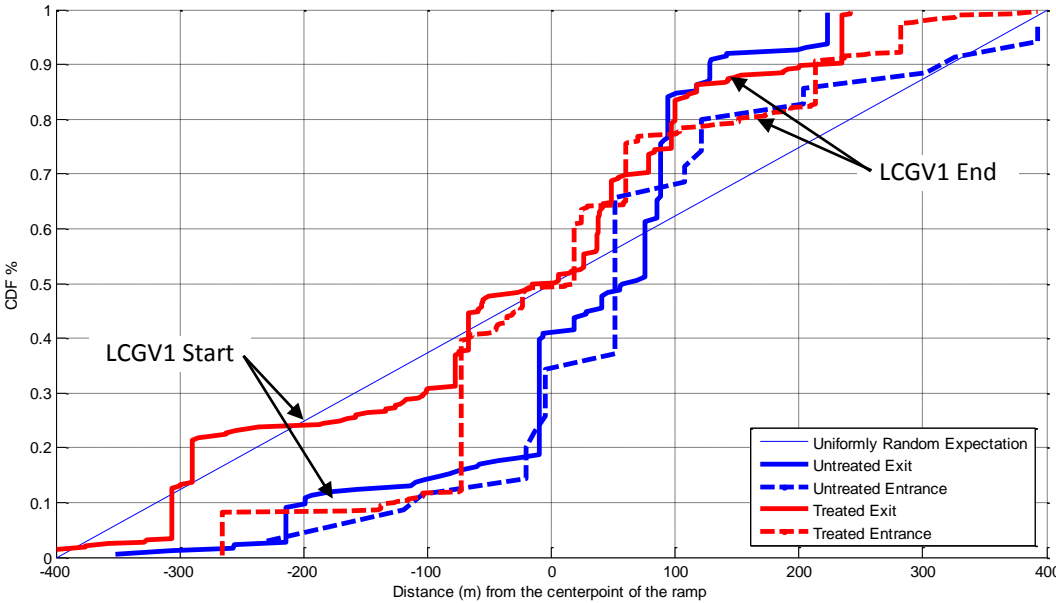


Figure 12 – Cumulative accident rate by distance from ramp center point (- upstream, + downstream) for highway entrance and exit ramps. The average start and end locations of the LCGV1 treatment are highlighted. Data source: SAAQ.

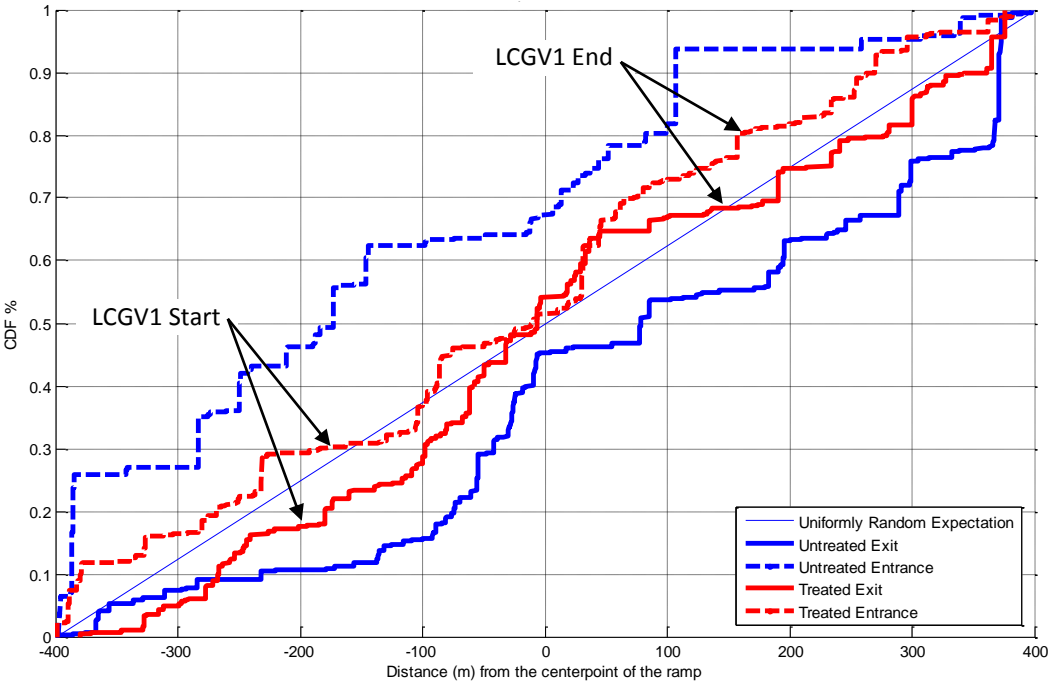


Figure 13 – Cumulative accident rate by distance from ramp center point (- upstream, + downstream) for highway entrance and exit ramps. The average start and end locations of the LCGV1 treatment are highlighted. Data source: Urgences-santé.

Figure 11 presents the temporal (hourly variation) distribution of accidents. It appears that most property-damage-only accidents, of either category, occur mostly during peak hours, while most severe and deadly accidents occur off peak hours. Knowing that the greatest flows and slowest speeds occur during peak hours, the aforementioned observation holds with the commonly accepted view that accident severity is strongly linked with speed: collisions at greater speeds have more kinetic energy to dissipate and cause physical damage.

Figure 12 and Figure 13 explore one-dimensional accident distribution as a function of distance from the center point of highway ramps for SAAQ and Urgences-santé accident data respectively. A uniform accident rate is expected to have a linear accumulation; accident clustering, on the other hand, is identified via steep accumulation. According to the SAAQ-collected data, there is some evidence to suggest that accident clusters form just upstream of the treatment at treated exits, at the pierhead for treated entrances, inside the merge length of untreated and treated entrance ramps, and along the chevrons and near the pierhead of treated and untreated exits. Overall, treated sites tend to have accidents shifted further upstream than untreated sites suggesting some type of accident migration effect. Nevertheless, accidents are generally more predominant downstream of the treatment than upstream, across all site types. This may be somewhat explained by the fact that the final locations of accidents are generally located downstream of the originating conflict, although this should not affect the uniformity of accident distribution unless geometry has an impact on how far apart accident initiating and ending locations are. According to the Urgences-santé-collected data, there is less evidence to suggest migration with respect to specific geometric features as accident distributions are somewhat more uniform, although overall trends still exist: for exits, there is still a downstream migration effect occurring, particularly near the start of the LCGV1 treatment. Interestingly and in contrast with the SAAQ accident data, evidence suggests that treated entrances shift accidents further upstream than untreated entrances. Also, accidents at untreated sites of both types are generally clustered at the ramp

center, where as accidents at treated sites of both types are generally clustered on the periphery of the ramp.

Differences in the two data sources can simply be explained by a smaller SAAQ data sample size (particularly for entrances) as well as less ge positioning precision. Overall, we can conclude that some strong evidence exists to suggest that the treatment has an upstream migration effect on accidents at exits. Evidence of accident migration at entrances is not significant enough to make a conclusion.

Figure 14 demonstrates Urgences-santé-collected, continuous accident density for the studied highways in Montréal, including Autoroute 20 and Autoroute 720 from Dorval to De la Montagne Street, Autoroute 15 from Autoroute 20 to Autoroute 40, Autoroute 40 from Autoroute 15 to Autoroute 25, Autoroute 25 from Autoroute 40 to the Louis-Hippolyte Lafontaine Bridge-Tunnel, and Autoroute 138, from Autoroute 20 to the Honoré Mercier Bridge. The greatest concentrations of accidents are located on the Métropolitaine section of Autoroute 40, particularly near the Christophe-Colomb interchange, and Autoroute 15 particularly near Autoroute 40 and de la Savane. Both highway segments are notoriously busy corridors and, incidentally, highly “underdesigned.”

Visual inspection between Figure 14 and maps of geometric features (see Appendix) appears to show hints of correlation between accidents and sites with “underdesigned” elements. The sites with the highest accident rates tend to correspond geographically with the sites with the highest flows and smallest geometric features including: painted chevron length ($l_{visibilité}$), ramp merge length ($l_{diffuseur}$), and downstream ramp distance (d_{bret_amont}) (section 2.2). This is formally tested in the next section based on a statistical analysis.

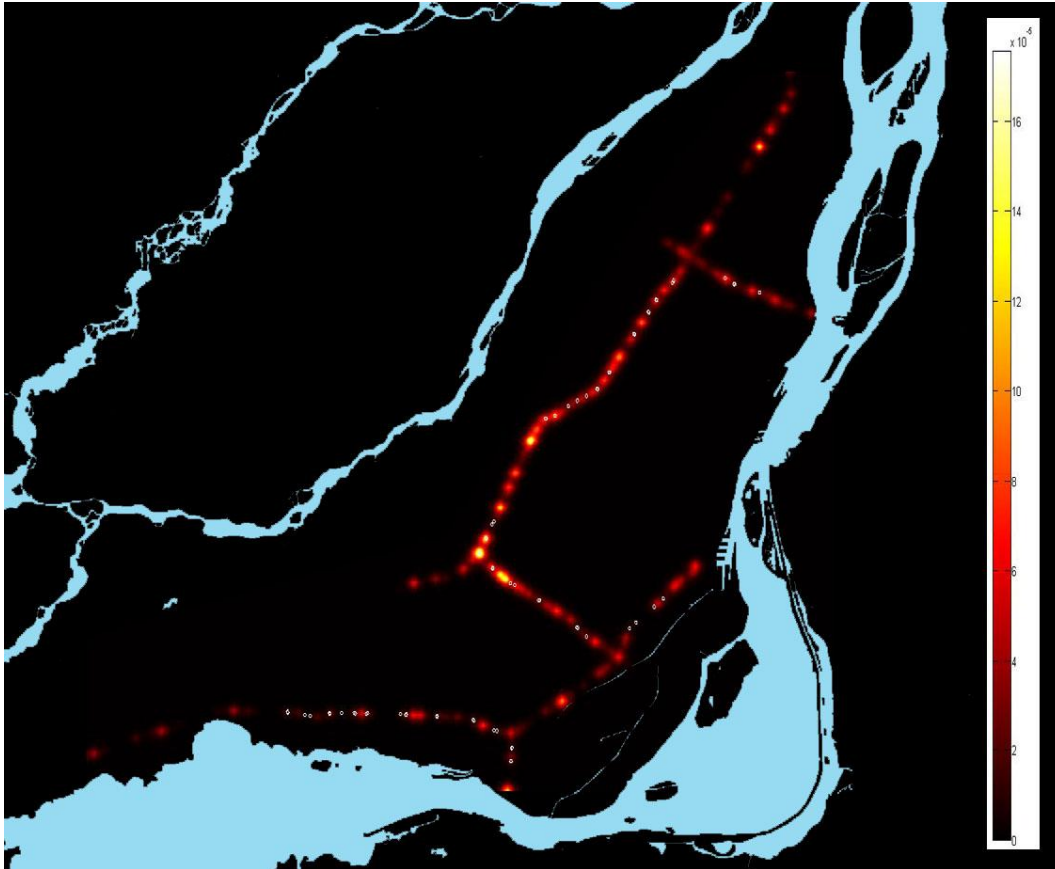


Figure 14 – Montréal highway accident density for studied highway segments with white dots representing study sites. Data source: Urgences-Santé, period: 2000-2010.

3.2 Statistical analysis

This section presents an in-depth accident data analysis based on the Urgences-santé data. This data is used as it is geo-coded at the source. Coordinates are recorded by means of a GPS system every time an ambulance is dispatched to the lieu of an accident. Moreover, the data exists for several years (1999-2008) and all locations under analysis. Note that SAAQ data is only available for a few sites and, as mentioned before, not geo-coded.

The accident analysis is presented in three parts: variables generation, exploratory analysis, and regression model analysis.

3.2.1 Accident variable generation and exploratory analysis

The accident rates for each site were defined according to buffers at each site. Buffers are generated from the geometric centre of every ramp (as defined in section 2.2) with the maximum size arbitrarily defined at 400 metres (HCM 2000 influence zone set at 450 metres, 85th percentile of ramp distances at approximately 400 metres). A second set of buffers of half the size, at 200 metres, are also used to check for significant spatial variation (recall Figure 12 and Figure 13. Figure 13 showed some degree of spatial variation). The buffers generated include the following:

- Total 200 m and 400 m buffers
- Upstream 200 m and 400 m buffers
- Downstream 200 m and 400 m buffers

Once the number of accidents for each buffer size was identified, the accident outcomes were then incorporated into the geometric and traffic flow inventory. A statistical summary of the list of variables generated for this study is provided in Table 4.

From these outcomes, one can observe that:

- The average number of accidents is larger than those reported in our previous analysis using the police report (SAAQ) data. This might be because of the different buffer sizes used to generate accident outcomes, the underreporting of minor accidents, or accidents that could not be properly located given the lack of spatial data in the SAAQ data.
- The mean number of accidents is relatively higher in exit ramps than in entrance ramps. However, accident rates (taking into account traffic conditions) appear to be more comparable.
- At exit ramps, the upstream traffic zone has, on average, more accidents than the downstream zone. The contrary is true at entrance ramps, where the concentration of accidents is higher in the downstream zone.

Table 4 – Average accident frequency at entrance and exit ramps. Data source: Urgences-santé, period: 2000-2010.

| Ramp type | Variable | Obs. | Mean | Std. Dev. | Min | Max |
|-----------|---------------------------------|------|----------|-----------|----------|-----------|
| Entrance | Total rates (400 m buffer) | 29 | 40.9 | 47.2 | 0.0 | 152.0 |
| | Total rates (200 m buffer) | 29 | 20.1 | 21.9 | 0.0 | 75.0 |
| | Upstream rates (200 m buffer) | 29 | 8.3 | 14.0 | 0.0 | 68.0 |
| | Downstream rates (200 m buffer) | 29 | 11.8 | 15.1 | 0.0 | 51.0 |
| | Upstream AADT | 24 | 53,887.1 | 18,051.6 | 24,632.1 | 103,389.8 |
| | Downstream AADT | 24 | 63,986.2 | 21,901.0 | 27,460.4 | 114,299.1 |
| | Ramp AADT | 24 | 10,099.1 | 6,105.0 | 771.0 | 22,778.0 |
| | Entrance ratio | 24 | 0.1 | 0.1 | 0.0 | 0.3 |
| Exit | Total rates (400 m buffer) | 31 | 53.4 | 38.7 | 0.0 | 147.0 |
| | Total rates (200 m buffer) | 31 | 29.6 | 28.4 | 0.0 | 141.0 |
| | Upstream rates (200 m buffer) | 31 | 18.5 | 17.6 | 0.0 | 75.0 |
| | Downstream rates (200 m buffer) | 31 | 11.1 | 17.3 | 0.0 | 66.0 |
| | Upstream AADT | 20 | 55,330.8 | 20,011.5 | 21,451.0 | 86,737.9 |
| | Downstream AADT | 20 | 44,335.2 | 19,286.6 | 15,609.0 | 77,577.1 |
| | Ramp AADT | 20 | 10,995.6 | 8,608.7 | 2,080.3 | 41,905.0 |
| | Exit ratio | 20 | 0.2 | 0.1 | 0.1 | 0.7 |

In addition to the raw average number of accidents as an exploratory analysis, the accident rate was estimated for each site (treated and non-treated) as follows:

$$R_i = \frac{y_i \times 10^6}{365 (T \times AADT_i)} \quad (4)$$

R_i represents accident rate of site i (accidents per million-vehicles), y_i represents accident frequency at site i during the period of analysis T (in years), and $AADT_i$ represents average annual daily traffic of site i . Note that for exit ramps, $AADT_i$ uses the upstream flow and for entrance ramps, $AADT_i$ uses the downstream flow.

Table 5 presents the average accident rates for the different buffer sizes. The rates are presented separately for both entrances and exits, as well as treated and non-treated ramps. From this table, it can be seen that, overall, the average rate of

accidents for treated entrance ramps is higher than the average rate for non-treated sites. However, the rates for exits are greater for non-treated sites. The differences between treated and non-treated sites seem to be marginal in most cases. This is tested in the following analysis using regression analysis and formal statistical tests.

Table 5 – Accident rates in treated and non-treated sites. Data source: Urgences-santé, period: 2000-2010.

| Ramp type | Treated (no and yes) | Variable | Obs. | Average Rate | Std. Dev. | Min | Max |
|------------------|-----------------------------|---------------------------------|-------------|---------------------|------------------|------------|------------|
| Entrance | Non-treated | Total rates (200 m buffer) | 10 | 0.17 | 0.20 | 0.00 | 0.68 |
| | | Total rates (400 m buffer) | 10 | 0.17 | 0.20 | 0.00 | 0.68 |
| | | Downstream rates (200 m buffer) | 10 | 0.04 | 0.06 | 0.00 | 0.21 |
| | | Upstream rates (200 m buffer) | 10 | 0.12 | 0.13 | 0.00 | 0.43 |
| | Treated | Total rates (200 m buffer) | 14 | 0.22 | 0.20 | 0.00 | 0.72 |
| | | Total rates (400 m buffer) | 14 | 0.22 | 0.20 | 0.00 | 0.72 |
| | | Downstream rates (200 m buffer) | 14 | 0.10 | 0.08 | 0.00 | 0.20 |
| | | Upstream rates (200 m buffer) | 14 | 0.11 | 0.16 | 0.00 | 0.54 |
| Exit | Non-treated | Total rates (200 m buffer) | 11 | 0.17 | 0.13 | 0.04 | 0.41 |
| | | Total rates (400 m buffer) | 11 | 0.44 | 0.30 | 0.06 | 1.05 |
| | | Downstream rates (200 m buffer) | 11 | 0.04 | 0.08 | 0.00 | 0.26 |
| | | Upstream rates (200 m buffer) | 11 | 0.11 | 0.08 | 0.03 | 0.30 |
| | Treated | Total rates (200 m buffer) | 9 | 0.15 | 0.10 | 0.02 | 0.32 |
| | | Total rates (400 m buffer) | 9 | 0.24 | 0.12 | 0.08 | 0.43 |
| | | Downstream rates (200 m buffer) | 9 | 0.04 | 0.04 | 0.00 | 0.11 |
| | | Upstream rates (200 m buffer) | 9 | 0.09 | 0.07 | 0.01 | 0.20 |

3.2.2 Regression analysis

Here the statistical analysis is extended in order to provide additional evidence of the impact of the treatment using a formal regression accident frequency analysis. For this purpose, we use the most commonly used statistical model in highway safety analysis, the Negative Binomial (NB) regression model.

This model can be written as, $Y_i \sim \text{Poisson}(T_i \theta_i)$, where Y_i represents the number of accidents (accident occurrence) at a given location i ($i=1, \dots, n$). Moreover, θ_i represents the mean accident frequency and is mathematically defined as $\theta_i = \mu_i \exp(\varepsilon_i)$, where $\mu_i = f(F_{i1}, F_{i2}, X_{ik})$ is the mean number of accidents, also known as safety performance function, and $\exp(\varepsilon_i)$ is the model error assumed to follow a Gamma distribution with a shape and inverse scale parameter α , also referred to as the dispersion parameter. Moreover, F_{i1} and F_{i2} represent traffic conditions and, X_{ik} , site-specific geometric attributes.

As part of the analysis, various safety performance functions are defined and tested. The general form used is the following one:

$$\mu_i = AADT_i^{\beta_1} (q_{\%e})^{\beta_2} \exp(\beta_0 + \beta_3 X_{i3} + \dots + \beta_k X_{ik}) \quad (5)$$

where $AADT_i$ represents the upstream AADT for entrance ramps and downstream AADT for exits. $q_{\%e}$ is the flow ratio as defined in section 2.3. Again, the upstream flow is used for entrances, and vice versa for exits. Instead of the flow ratio, the ramp AADT alone can be used. $X_{i1} \dots X_{ik}$ represents geometric characteristics for a given site.

To identify the geometric variables potentially associated with the mean number of accidents (μ_i), a correlation matrix is first calculated. From this, it is clear that, in addition to traffic volumes, visibility, and distance to the previous (upstream) ramp, there are also factors further correlated with number of accidents. Different models with one, two, and three geometric factors are fitted to the data. The best models are identified based on the level of significance for each individual parameter and the overall goodness of fit. The resulting models are presented in Table 6.

Table 6 – Models for entrance ramps. * For entrances, this is the upstream AADT, ** in 100s, * in 1000s, numbers in bold indicate statistical significance at 10% or less.**

| Variables | Total accident model (400 m buffer) | | Total accident model (200 m buffer) | | Upstream accident model (200 m buffer) | | Downstream accident model (200 m buffer) | |
|---------------------------|-------------------------------------|-------------|-------------------------------------|-------------|--|-------------|--|---------|
| | Coef. | p-value | Coef. | p-value | Coef. | p-value | Coef. | p-value |
| AADT* | 0.93* | 0.07 | 1.00 | 0.07 | 2.00 | 0.00 | 0.44 | 0.67 |
| Flow ratio | 0.75 | 0.02 | 0.28 | 0.46 | 0.97 | 0.02 | 0.16 | 0.83 |
| Visibility** | -0.78 | 0.10 | -1.22 | 0.05 | -1.30 | 0.04 | -1.05 | 0.35 |
| Upstream ramp distance*** | -0.47 | 0.40 | -0.46 | 0.46 | -2.66 | 0.00 | 0.84 | 0.45 |
| Treatment | 0.26 | 0.49 | 0.37 | 0.38 | -0.14 | 0.77 | 0.48 | 0.42 |
| Constant | -4.41 | 0.44 | -6.41 | 0.31 | - | 0.03 | -2.57 | 0.83 |
| alpha | 0.53 | | 0.61 | | 0.50 | | 1.34 | |

Table 7 – Models for exit ramps. * For exits, this is the downstream AADT, ** in 100s, * in 1000s, numbers in bold indicate statistical significance at 10% or less.**

| Variables | Total accident model - (400 m buffer) | | Total accident model (200 m buffer) | | Upstream accident model (200 m buffer) | | Downstream accident model – (200 m buffer) | |
|---------------------------|---------------------------------------|-------------|-------------------------------------|-------------|--|-------------|--|-------------|
| | Coef. | p-value | Coef. | p-value | Coef. | p-value | Coef. | p-value |
| AADT* | 0.57 | 0.07 | 0.15 | 0.60 | 0.52 | 0.12 | 0.01 | 0.99 |
| Flow ratio | 0.41 | 0.08 | 0.24 | 0.29 | 0.18 | 0.36 | 0.61 | 0.25 |
| Visibility** | -1.18 | 0.12 | -2.32 | 0.00 | -1.97 | 0.00 | -3.97 | 0.01 |
| Upstream ramp distance*** | -0.20 | 0.58 | -0.90 | 0.01 | -1.16 | 0.00 | 0.66 | 0.42 |
| Treatment | -0.63 | 0.07 | -0.11 | 0.71 | -0.20 | 0.49 | -0.19 | 0.82 |
| Constant | -0.08 | 0.98 | 4.33 | 0.14 | -0.08 | 0.98 | 4.68 | 0.54 |
| alpha | 0.33 | | 0.24 | | 0.21 | | 1.07 | |

The regression modeling results are in accordance with the patterns observed in the exploratory analysis. After controlling for traffic and geometric conditions, the treatment has a non-statistically significant positive effect on entrance ramps in most of the buffers (total 400 m, total 200 m, and downstream 200 m). However, for exits, the effect is negative but also not statistically significant in most cases, with the exception of the model using accidents in a 400 m buffer.

Traffic conditions and geometric designs seem to be, in most cases, the most significant factors with an error of less than 10%. Again, these results confirm the

important role that geometry and traffic conditions can play in collision occurrence. Moreover, the effect of the treatment seems to be marginal after taking into account other geometric and traffic conditions.

It is important to keep in mind that this analysis has important limitations. First, the dataset used in the analysis has a very small sample size. Some of the AADT data was generated based on manual and video counts and expanded using known expansion factors. In many cases, the treated sites have different geometric conditions which can introduce a selection bias to the analysis.

Chapter 4: Video data collection

4.1 MTQ sources of video data and limitations

Before the automated video analysis methodology was proposed, some early videos had been taken by the Ministère des Transports du Québec and provided for the purpose of visual evidence of lane changes. It was concluded that some of the videos were useable for a more systematic and automated conflict analysis, therefore more videos were obtained. The first batch of videos, taken in late 2009 and early 2010 focused primarily on the original “underdesigned” sites along Autoroute 40, including Christoph-Colomb, and then some moderately comparable sites along Autoroute 20 and Autoroute 720.

Table 8 – Summary of video data available from MTQ highway cameras. *Videos for Cam20-16-Dorval with a view in the direction of the second entrance (2) were available for both before and after treatment, nearly one year apart.

| Camera ID | Highway | Entrance/Exit | Treatment | Visual Quality |
|----------------------|---------|---------------|-----------|----------------------------------|
| | | November 2009 | | |
| Cam40-11-CC | 40-W | Entrance | Treated | Moderate: Some traffic, angle |
| Cam40-13-d'Iberville | 40-W | Exit | Treated | Poor: Out of range |
| | | May 2010 | | |
| Cam20-16-Dorval* | 20-W | Entrance (2) | Untreated | Good |
| Cam20-14-32e_avenue | 20-W | Exit | Treated | Poor: Sign and pole obstructions |
| Cam20-11-1ere_avenue | 20-E | Entrance | Untreated | Poor: Heavy traffic, angle |
| Cam138-03-Clément | 138-E | Entrance | Untreated | Good |
| Cam720-05-Green | 720-W | Entrance | Treated | Good |
| Cam720-04-St-Jaques | 720-W | Exit | Treated | Poor: Out of range, shakiness |
| | | November 2010 | | |
| Cam40-17-Langelier | 40-W | Entrance | Treated | Poor: Out of range, rain |
| Cam40-17-Langelier | 40-W | Exit | Treated | Poor: Out of range, rain |
| Cam40-18-G.Anjou | 40-W | Interchange | Untreated | Poor: Heavy traffic, rain |
| Cam25-02-Anjou | 25-S | Interchange | Untreated | Poor: Heavy traffic, rain |
| Cam20-16-Dorval | 20-W | Entrance (1) | Untreated | Poor: overpass visibility issues |
| | | May 2011 | | |
| Cam20-16-Dorval * | 20-W | Entrance (2) | Treated | Good |

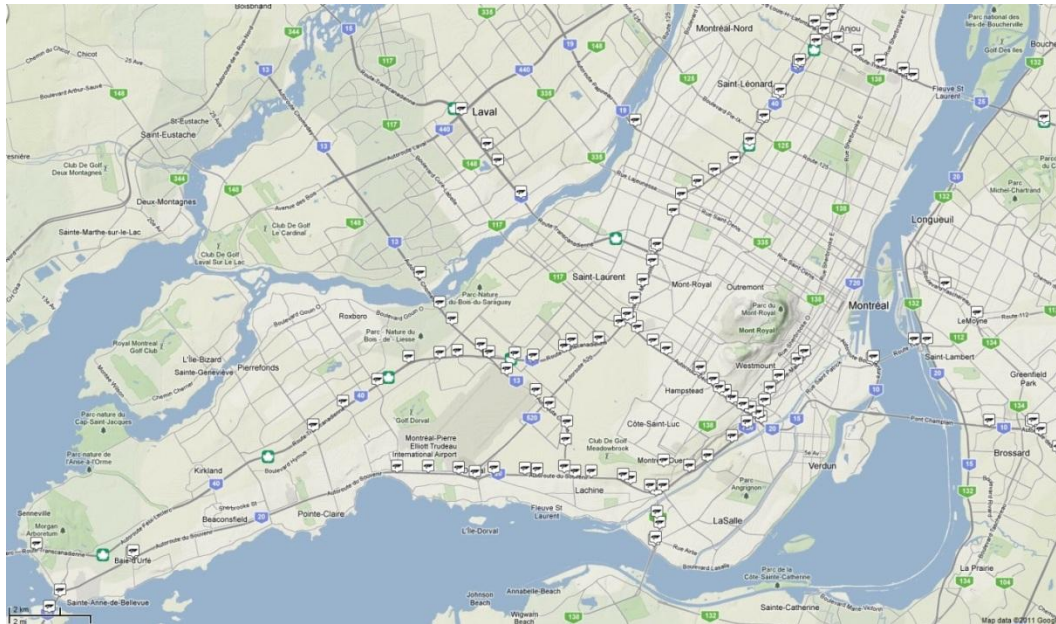


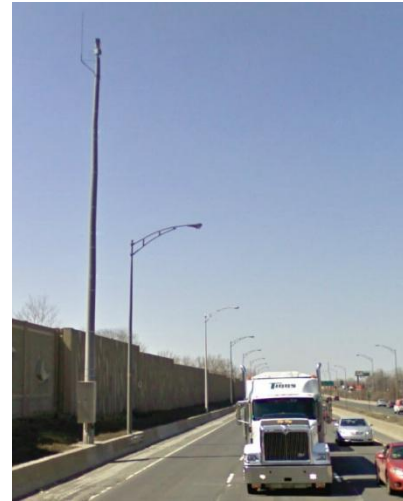
Figure 15 – Map indicating location of installed MTQ traffic cameras as of September 2011. Source: Québec 511.

Figure 15 maps MTQ highway cameras along Montréal highways. Some sample shots of these cameras are provided in Figure 1, Figure 7, and Figure 16. While the views in Figure 1 and Figure 7 are good and are further used in the video analysis (see below), the view from Figure 16 is not particularly useful, as most of the view is blocked by large super-structure signage and lamp posts.

Furthermore, as vehicles approach the horizon, their pixelated projection on the image sensor becomes smaller, meaning pixels begin representing larger and larger features of each object. This phenomenon results in a loss of tracking accuracy the further away the object moves from the camera. With medium and low-resolution cameras, such as those used on highways, tracking becomes increasingly difficult at further distances. The practical viewing distance in clear weather of a 60-foot 720x480 pixel camera is roughly 500 metres, whereas the practical tracking distance is between 50-100 metres; furthermore, tracking generally stops after 200-300 metres.



a)



b)

Figure 16 – a) Autoroute 20 “Autoroute Jean-Lesage” eastbound, exit 60, “32e Avenue/A-13”, view from Cam20-14-eastbound; b) Cam20-14 installation (large pole on the left).

4.2 Construction of a video data collection tool

Given the important limitations and the lack of video data from the MTQ cameras, a video data collection tool was designed. During the conception of this tool, the following characteristics of the equipment were considered:

- **Mobile:** The primary requirement of a mobile camera system is that it be capable of being installed at locations and cover angles that fixed security cameras cannot reach. Mobility is advantageous for short and numerous installations.
- **Tall:** The higher the camera is positioned relative to the road, the steeper the viewing angle and the larger the viewing zone. A steeper viewing angle further reduces lane parallax error. A minimum of 7 metres in height is strongly suggested.
- **Weather proof:** The electronic components must be shielded from rain, severe wind, and direct sunlight. The camera lens should also have a visor to reduce sun glare.
- **Remote:** Outlets and internet ports are not available in the field. All electrical systems must be completely independent and capable of operating autonomously in the field.
 - **Power:** The unit requires electricity to operate. Therefore, every unit is to be supplied with a battery pack to last at least 24 hours continuously, including filming and stand-by.
 - **Data:** Video data consumes a lot of disk space, and otherwise requires very expensive wireless broadband for live upload to a remote server. Every unit is to have detachable recording media to last the unit 8-16 hours of continuous recording.
- **Stable:** A certain amount of video shakiness can be corrected with video stabilization software; however this action degrades the quality of the video and adds additional processing time. Furthermore, stabilization software cannot correct excessive shaking. The two primary sources of shakiness present in the highway environment include vibrations from vehicles

(particularly trucks) and high winds (see section 4.4). Finally, it should be noted that the equipment must be structurally sound with failsafe redundancy in order to prevent equipment from falling onto the highway.

- Vandal proof: Vandalism of public property, and particularly that of cameras and other monitoring equipment, is not unheard of, even in the middle of highways. Constant supervision is not a very practical solution and so the camera must be protected against theft, equipment tampering (particularly cable cutting), and especially from inadvertently falling on the highway as a result of vandalism.
- Discrete: The presence of obvious surveillance equipment and other activity in general on the highway could lead to driver distraction and pose a safety risk. Additionally, the prominent presence of a camera might externally affect driver behaviour, by distraction or dissuasion.
- Easy installation: A limited budget, live highway environment challenges, and frequent short installations require the camera to be easily and quickly installed. No cranes, “bucket boom” trucks, heavy ladders, or power tools should be needed.
- Budget: The project budget is very small. Depending on the design choices, there is only room for the construction and operation of a single camera unit.
- Emergency identification: Although not a constraining factor, it is still a requirement to have every module labeled with ownership and emergency contact information.

No single solution exists for the design, as some constraints are in direct competition with each other. For example, to make a camera fully vandal proof, it must be permanently affixed to the infrastructure or placed high up out of reach, involving complicated installation work. Another solution might be to position the camera on a vehicle, on an immobile trailer, or to actively supervise it, although all these options come with problems of discreetness and mobility. Mobility also tends to come at the expense of stability and height. Different sites and

applications will benefit from different configurations and a modular camera system would provide the flexibility needed for different sites.

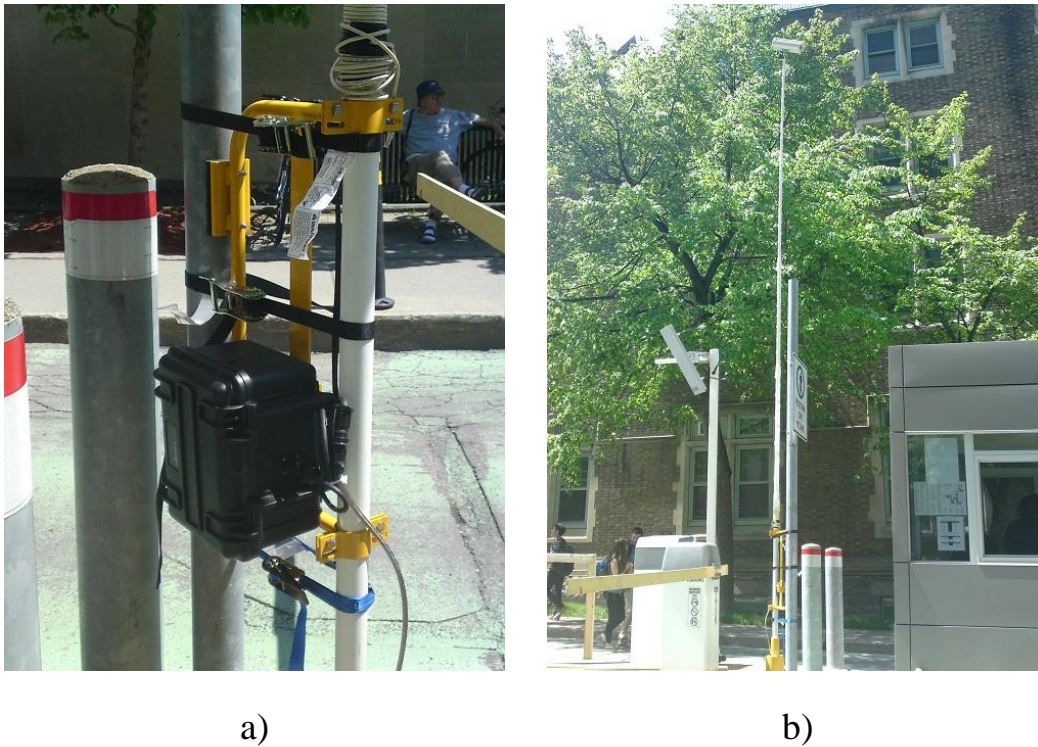


Figure 17 – Short-pole test installation of the recycled MioVision hardware, version 1.0: a) base installation, b) complete view.

Figure 17 demonstrates the first selected design—essentially a completely rewired and reinforced recycled MioVision pole VCU—being tested. The design choice of this unit compromises some stability for a very light, easy to install, discrete pole. The unit attaches itself behind other poles for stability, discretion, and security, although this limits the deployment capabilities somewhat. The basic installation procedure is depicted in Figure 18, including winch straps, chains and some rope.

Figure 19 shows the camera head in a free-standing deployment. A standard gutted camera housing protects the camera from the elements and sun glare

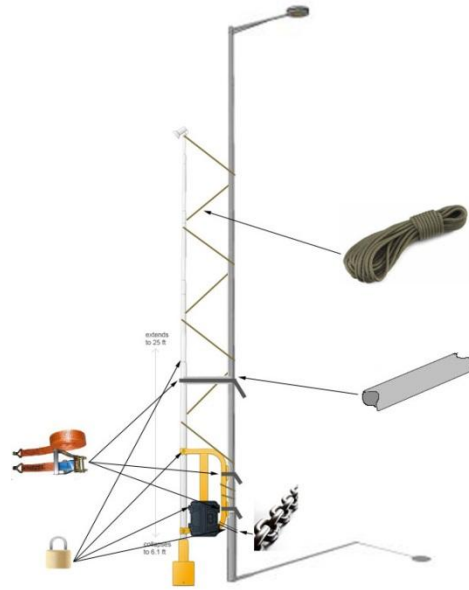


Figure 18 – Schematic illustrating the basic installation components and procedure.



Figure 19 – Free-standing camera housing deployed to a height of 8 metres.

4.3 Video data collection: site selection and issues

The mobile video data collection unit designed and presented in section 4.2 was installed at various highway sites during the course of the summer of 2011. The choice of site was governed primarily by accessibility and availability of infrastructure to install against and under the supervision of an MTQ official.

Viaducts could not be accessed because of logistical difficulties, for they generally do not incorporate large shoulders and/or accessible adjacent facilities. These locations would require partial or full lane closure(s) for installation work. Facilities located at the centre of median-separated highways (as opposed to ditch-separated highways) are also generally inaccessible without a full-scale partial or full lane closure operation. Unfortunately, these sites represent a significant portion of Montréal highways, particularly the busiest, most “underdesigned” sections, including Autoroute 40 and Autoroute 720. Nevertheless, these sites do not constitute “typical” sites.



Figure 20 – Base of pole installation for MTG-Cam20-E-55e-Sortie.

Sites selected for equipment installation were those with large shoulders and/or 2 metres or more of provincial land accessible by foot or vehicle. Camera installations at such sites are depicted in Figure 20 and Figure 21. The majority of these sites have some type of outside infrastructure next to or along merging ramps, including lampposts and superstructures. Autoroute 20, Autoroute 13, and Autoroute 25 presented many of these features. Virtually all rural highways are easily accessible in such a manner. Otherwise, sites were randomly picked to present an equal mix of entrances and exits, treated and untreated sites.



a)

b)

Figure 21 – Superstructure installation at a) MTG-Cam25-N-Sherbrooke-Sortie, b) MTG-Cam25-S-Sherbrooke-Sortie.

Table 9 lists the sites filmed using the mobile video data collection unit. Virtually all installations presented a superior visual quality over the stationary MTQ security cameras, as all installations were made in accordance with trajectory analysis requirements. The primary limiting factor in all of these installations' visual qualities was the viewing angle. Camera height was limited

according to unit stability and physical size of the unit. These limitations however were largely offset by a closer shot, a much higher capturing resolution and more framing control.

Table 9 – Summary of video data collected by the research team.

| Camera ID | Highway | Entrance/Exit | Treatment | Visual Quality |
|-----------------------------------|---------|---------------|-----------|---------------------|
| June 2011 | | | | |
| MTG-Cam20-Dorval | 20-W | Entrance | Treated | Moderate: shakiness |
| MTG-Cam20-E-55e- Sortie | 20-W | Exit | Treated | Good |
| MTG-Cam20-E-55e- Entree | 20-W | Entrance | Treated | Good |
| MTG-Cam20-1ere | 20-E | Entrance | Untreated | Good |
| MTG-Cam25-S- Sherbrooke-Sortie | 25-S | Exit | Untreated | Moderate: Angle |
| MTG-Cam25-N- Sherbrooke-Sortie | 25-N | Exit | Treated | Moderate: Angle |
| MTG-Cam13-520E- Sortie | 13-N | Exit | Untreated | Good |

4.4 Video data pre-processing

Before video data can be processed, some pre-processing must be performed if the video data has stability or format issues. Video stability is generally an issue with mobile equipment and the video will need to be stabilized. Stationary cameras often have proprietary formatting which must be corrected to make the video data compatible with the video processing software. In either case, a choice in compression is required, affecting video quality and video size; this generally means a trade-off between trajectory processing accuracy and data management complexity. A higher quality video file will yield more accurate trajectory extrapolations, at the cost of a larger video size leading to penalties on processing time and storage (a single hour at a single site can use anywhere from 1 GB to 20 GB of data, overhead and extrapolation data included).

Furthermore, it is important to note that any video compression operation introduces new artefacts into the video, impacting the video's quality in the same manner that the quality of a paper document decreases every time it is photocopied. This includes any compression performed natively by the camera. Uncompressed video is, with the current technology, impractical to store and transmit, however.

The targeted video format has the following characteristics: digital, at least 800 pixels wide, 600 pixels tall, a square pixel ratio, deinterlaced (progressive scan), 25 frames per second or more (29.97 NTSC standardized), and no audio. Frames must be consistently timed in order to obtain realistic and consistent speeds. The greater the number of pixels, the higher the accuracy, particularly for areas closest to the horizon, although the gain in distance from increased pixels offers diminishing returns, particularly for low-angled views.

The compression algorithm of choice is H.264 at a bit-rate of 2.5Mbit/s using the .avi container. Of special note is the fact that H.264 has compatibility issues with the .avi container and was primarily designed for use with the .mp4 container. The practice of using H.264 is discouraged but is adopted here due to

limitations intrinsic to the video analysis libraries used. The compression engine used is the open source x264, which has convenient support for .avi with the ffdshow codec pack.

Under normal operation, two major sources of shake exist: vehicle vibrations and wind. Camera shake is manifested in the camera view from tilting, panning, and rotation of the view. The magnitude and frequency of this shakiness is highly dependent on camera installation. Stabilization is the process of removing any global tilt, pan, or rotation in the camera view. This is generally achieved by tracking the movement (local colour changes) of multiple pixels and calculating an overall movement pattern. By applying the inverse of the movement to the entire image, motion compensation is effectively achieved.

Chapter 5: Video data analysis

Recent developments in semi-automated conflicts analysis from traffic videos are put to use in this study to address some of the shortcomings of the historical analysis method. Videos are semi-automatically analysed via computer tracking to produce vehicle trajectories in time and space. Various measures of driver behaviour (based on observed spatial relationships between vehicles and road geometry) are introduced and reported. These measures include the conflict measure *time-to-collision* (TTC), lane change rate, trajectory density, and flow ratios. Every site analysed in this way is done so over a highway segment (between 50-75 metres in length) at a particular location with respect to the ramp.

5.1 Methodology

This study makes use of the video analysis tool developed at the University of British-Columbia to track vehicles from video data (11) (17). Individual pixels are tracked and followed over the course of many frames and recorded as features trajectories.

The positional analysis of vehicles requires the accurate projection of pixel coordinates in image space to real-world coordinates that lie on a reference surface with a known model (pavement surface). When video data is collected by a third party, access to the camera is not possible and therefore all camera parameters must to be inferred from video observations and an orthographic (aerial) image of the intersection. This is done using a robust calibration method relying on various features such as the shape, position, and length of remarkable objects in both image and world spaces (3). Additional issues are caused by slight camera orientation drift over time, which was dealt with automatically by tracking the stationary portion of the field of view. Time is measured in frames: a datapoint (position per object per frame) is collected for each new video frame and there are 29.96 frames per second. A displacement of 1 metre from one datapoint to the next (1 m/f) represents an object traveling at a speed of 29.96 m/s or 107.86 km/h. This high polling rate produces very large datasets of small increments.

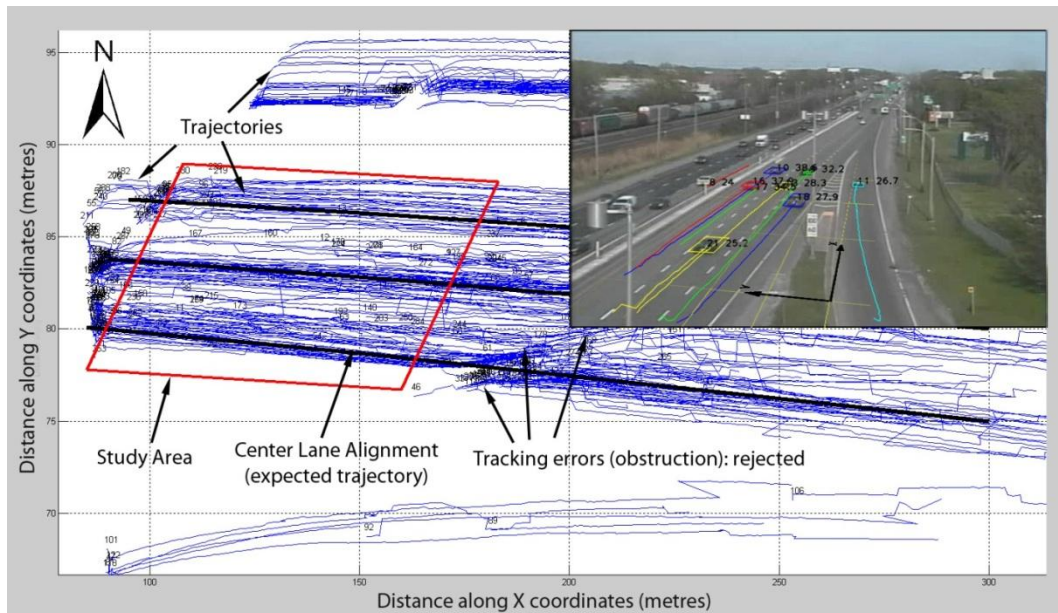


Figure 22 – Sample X,Y data for spatial analysis of entrance 56 (Bouchard), Autoroute 20 eastbound, Dorval, Montréal. Datapoints are filtered to include only the study area (50 m long by 10 m wide).

A second phase of data filtering was developed specifically for this study to optimize the tracking reliability under the constraints of highway flow and for the type of camera angles used to record the video footage. This phase includes edge and warm-up truncation, expected trajectories coordinate transformation, noise reduction, and tracking error flagging (such as duplicate objects, multiple vehicles per object, split objects, etc.) for manual review. These filtering routines were empirically validated.

Figure 22 shows sample trajectories being extrapolated and the selection of a study area to remove unreliable trajectories. Vehicles were assigned a lane and a set of transformed coordinates for rear-end calculations based on an expected trajectory representing the average path of trajectory clusters associated with each lane.

Conflict measures have been broadly defined in many publications, but in general they tend to be derived from three variables: the positions x and y of two or more vehicles as a function of time t . In a 2003 report (18), Gettman and Head compiled a list of the major and recurring surrogate measures of safety used in the

literature. The report identified seven major measures: gap time (GT), encroachment time (ET), deceleration rate (DR), proportion of stopping distance (PSD), post-encroachment time (PET), initially attempted post-encroachment time (IAPT), and time to collision (TTC). The FHWA report defines these measures primarily as indicators of probability of collision. It should be noted, however, that the exact relationship between conflicts and collisions has yet to be clearly defined.

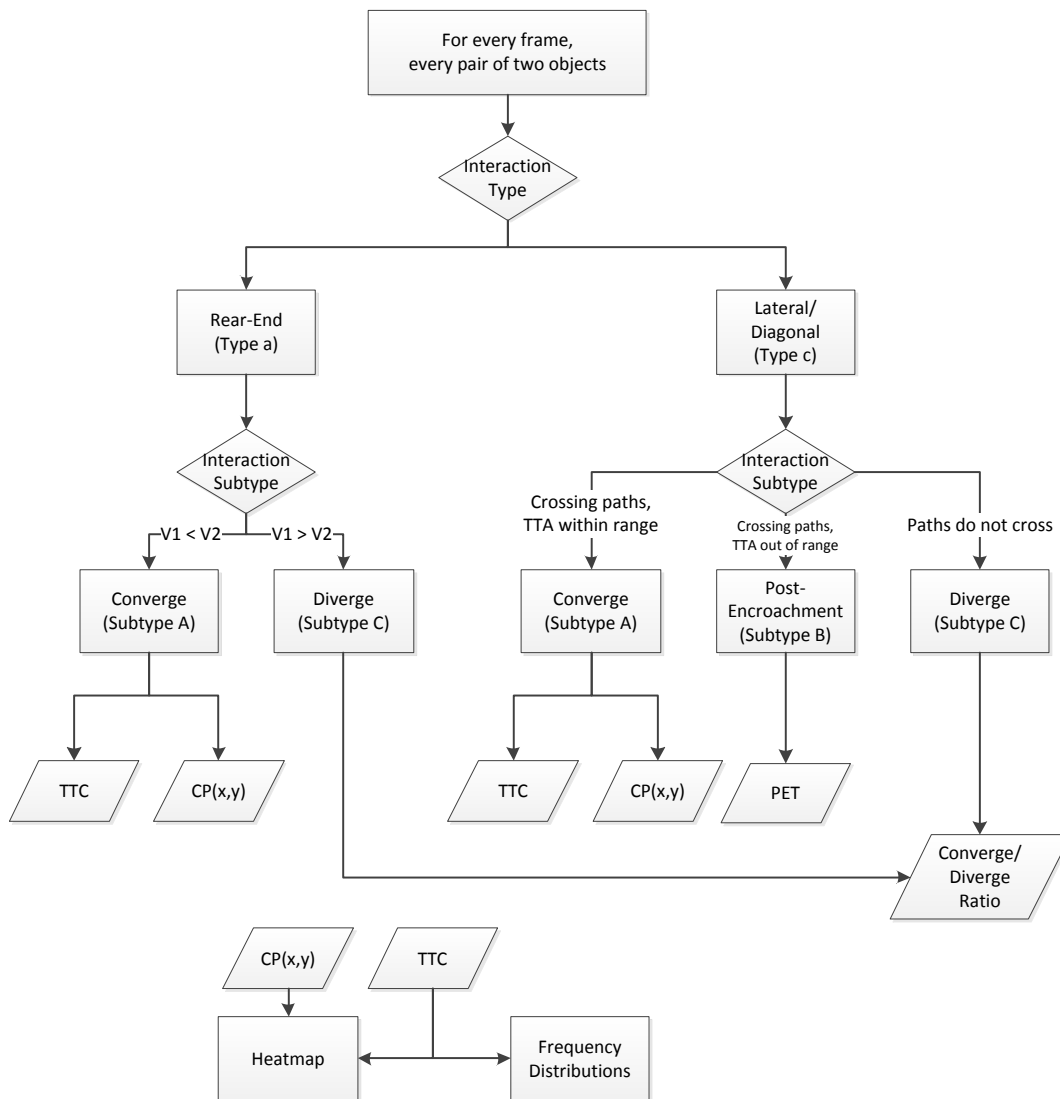


Figure 23 – Interaction classification and data extraction flowchart. Time-to-Arrival (TTA), Conflict Point (CP), Leading Car Speed (V1), Following Car Speed (V2).

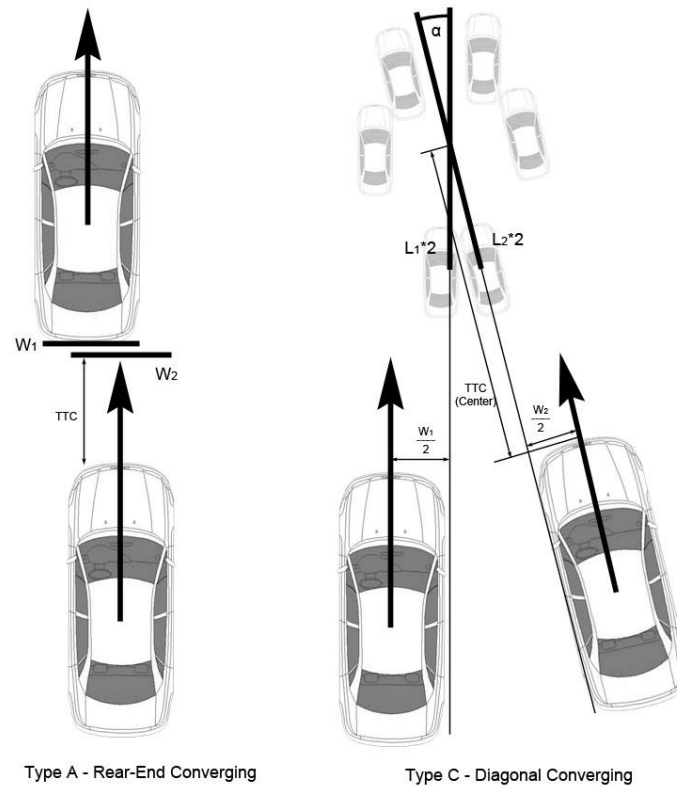


Figure 24 – Path prediction types. TTC is calculated from speed, position, width W , length L , and angle of conflict α for converging interactions.

For the purpose of freeway conflict analysis, where we assume no head-on or perpendicular-lateral conflict situations and a certain amount of constraint-of-direction, we focus primarily on two types of major interactions: rear-end (type A), and lateral-diagonal (type C). Each of these can be observed as either converging or diverging (see Figure 23 and Figure 24 for the classification). Out of all the measures mentioned previously, TTC is found to be the most reliably measured, tends to be already collinear with other measures (e.g. GT, DR, PSD), and is the most frequently observed in a highway environment (e.g. the other major conflict type (PET) was measured less than 0.1 % of the time) and so is chosen as the primary measure of comparison (8). Additionally, it is already a popular choice in the literature.

The TTC measurement can be defined as the time until two objects, whose predicted paths defined by constant velocity at that point in time intersect, meet and collide (19). A straight line TTC path and collision prediction method was

deemed sufficient for the requirements of this paper as all studied examples are in straight highway sections. The algorithm used to make TTC measurements is based off of the work by Laureshyn et al.(6). In situations where the vehicles are not predicted to collide (no collision course) and TTC does not exist, but paths converge, a PET measurement is recorded instead. Additionally, the position (x, y) of each predicted collision point CP is recorded. See Figure 24 for an illustration of the path prediction and measuring algorithm used. We leave the discussion of other methods such as “expected trajectory” (8) and path-probability prediction for future work.

In order to make the TTC measure absolutely useful in the context of road safety, it is important to understand its relationship with collision probability, if it reliably exists. Unfortunately, a formal relationship between the two still requires much research (both empirically and theoretically from driver behaviour literature). TTC is a measure of the remaining time, at any time t , before two vehicles are expected to collide given no driver reaction over the time span of TTC. Thus the observed outcome of such an event, on average, is proposed as a method of empirically measuring the probability of collision of an interaction, at time t , over a time step Δt , given a TTC and other factors (such as driver reaction time, visibility, vehicle performance and impairment). Accordingly, the following relationship is given by:

$$PC(t) = f(TTC(t), factors) \quad (6)$$

Where $PC(t)$ is the probability of collision which depends of $TTC(t)$ and other factors θ . By definition of time-to-collision, the probability of a collision for $TTC = 0$ is 1:

$$PC(TTC(t) = 0, factors) = 1 \quad (7)$$

Because the probability that a driver reacts to a potential collision likely decreases as TTC shortens (an effect likely compounded by the reaction lag associated with

the static action-reaction cycle of driver behaviour), we can presume that the general relationship between probability of collision and TTC is exponentially decaying, given otherwise identical factors. The following formula is then proposed for illustration purposes to weigh an interaction with a given TTC at instant t :

$$WC(TTC(t)) = \frac{1}{e^{TTC(t)}} \quad (8)$$

where $WC(TTC(t))$ is the weighted collision density function used to compare weighted collision density maps (“heat map”). This type of theoretical accident probability weighing shares similarities with previous discussions of accident probability, e.g. (20). One proposed approach would be to calibrate $WC(TTC(t))$ with a comparable observed $PC(t)$ for sets of identical factors, although it is not yet clear what types of research applications could take advantage of such information. For this study, it is assumed that factors leading to changes in TTC distributions are significant whereas factors leading to changes in $WC(t)$ relationships are not significant on the grounds that the LCGV1 does not introduce new interaction types and that the LCGV1 produces no physical change to the environment.

Weighted collision density maps are a means of examining the spatial distribution of conflicts according to “heat”. These maps are produced by plotting the 2-dimensional histogram of all observed $WC(TTC(t))$ according to $CP(x,y)$ using a Gaussian kernel size of 50 mm by 50 mm. Areas with a greater density of conflicts and as well as conflicts of smaller TTC are highlighted in this way. Although, from a practical standpoint, $WC(TTC(t))$ does not influence density across lanes, as virtually all trajectories are parallel or near parallel with the highway, future work will attempt to turn this weighing function into a properly calibrated probability of collision function; until then weighted collision density maps are used primarily for cross-sectional and before-after comparisons only.

Figure 25 shows a sample conflict analysis report and labels various elements. Elements 1) through 5) and 8) provide metadata on the site analysed. It

should be noted that the presence of the treatment in some satellite imagery 4) conflicts with what the camera view presents 5). Conditions as seen in the camera view take precedence. Element 6) is the weighted conflict density map or “heatmap” which plots conflict density by location in (x,y) space and in weight according to equation (8): “hotter” colours indicate a greater density of small TTC measures suggesting a greater risk of accident at this location (in this case in lanes 2 and 3, with lane three having a slightly longer density trail). Element 7) is the analysis area which delimits where interactions are analysed. It should be noted that heatmap trails often extend outside of this region. These conflicts are projected out of the movements confined to the analysis area and will likely overlap with any conflicts measured in any analysis area further downstream. Similarly, any conflicts analysed in an analysis area upstream of this area will also overlap. Heatmaps should therefore only be read as extrapolated conflicts resulting of behaviour confined to the analysis area. Element 8) provides visual reference to existing road geometry in relationship to the measured location of conflicts. This and all other heatmap elements, including conflict density, are georeferenced, although axes are not equally scaled in order to fit information on the graph. Element 9) provides a normalized colour ramp used to highlight density differences within sites only. Element 10) provides heatmaps for particular conflict types. In this example, Type A (rear-end) conflicts bear a similarity to the master heatmap, suggesting that it is the dominant conflict type. Type C (side-swipe) conflicts have the greatest weighted density in the second lane, with an otherwise equal, if not noisy, spread. Elements 11) through 13) provide a TTC (measured in seconds) versus observation rate (as a percentage per 0.5 seconds) conflict distribution graph. As the density of conflicts shifts towards 0, the probability of accidents is expected to increase. The graph also numerically compares distributions between Type A and Type C conflicts as well as conflicts summarized by all interactions (average over time) or according to the minimum observation for each unique interaction pair (the latter is generally noisier and sensitive to tracking errors, but more true to the original definition and observation of a conflict). Finally, it should be mentioned that all distributions

have a distribution tail that converges to 0 as $TTC \rightarrow \infty$. For all distributions, it is observed that the area bounded by distributions after 40 seconds represents approximately 5% of the distribution area and is largely composed of error. Conflicts greater than 40 seconds (or even arguably 10 seconds) have little practical significance in predicting accidents in any case, given that the average driver perception time is approximately 1.5 seconds (15).

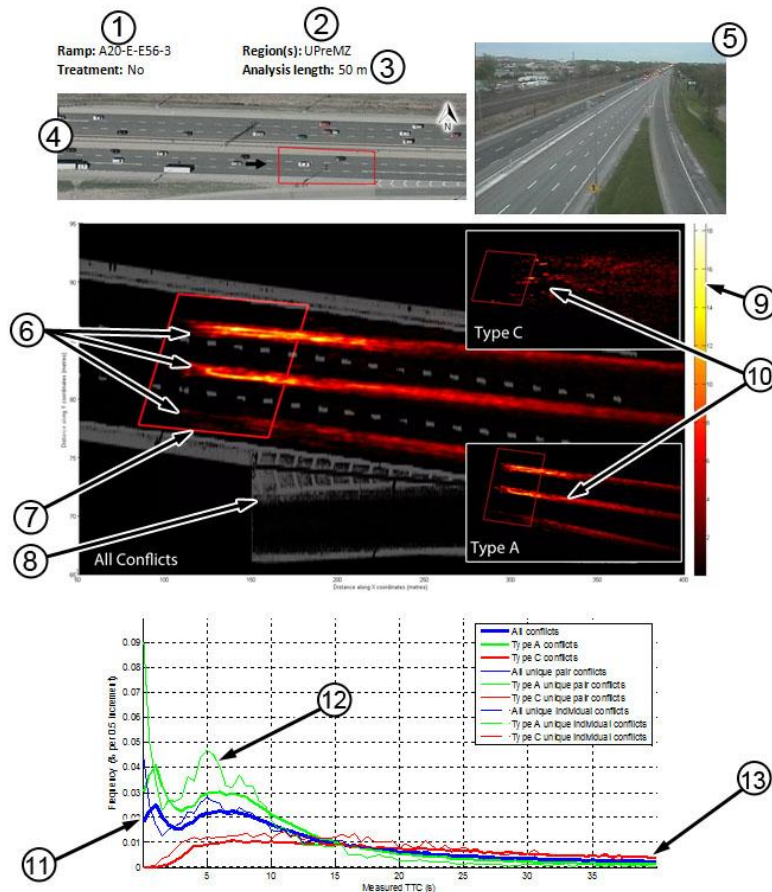


Figure 25 – Reading a conflict analysis report: 1) Site index, see section 1.1 for notation; 2) Corresponding ramp sub-sections as defined in section 1.3; 3) alignment length of the analysis area; 4) analysis area superimposed over satellite imagery used for calibration, flow from left to right; 5) sample view from site camera; 6) conflict heatmap with visible trails; 7) analysis area box; 8) approximate superimposed geometry for visualization, axis scales are not proportional; 9) normalized colour ramp indicating “heat” or weighted density level; 10) heatmaps according to conflict types; 11) conflict distribution, TTC in seconds versus observed rate, where blue = all conflicts, green = Type A conflicts, red = Type C conflicts; 12) distributions vary slightly by summary method; 13) conflict distributions converge towards 0 observations after a TTC of 40 seconds.

Figure 26 presents typical speed distribution as measured by the tracking algorithm. Speeds are generally normally distributed. The average site is found to have a median speed of 90 to 110 km/h, with a standard deviation of 10 to 15 km/h (see Table 10 and Table 12 of sections 5.2.1 and 0 respectively for full results). For some sites, nearly 70% of drivers are recorded as driving above the posted speed limit (100 km/h), although some of these can be attributed to projection/calibration errors (mostly measures above 140 km/h). Ramp speeds

were normally distributed around 75 km/h with a larger standard deviation of about 25 km/h (acceleration/deceleration). Ramp speeds should be considered speculative only; ramp measurements were not conducted systematically and the dynamics of ramp acceleration and deceleration dictate that normal speeds are appreciably different at different locations along the ramp.

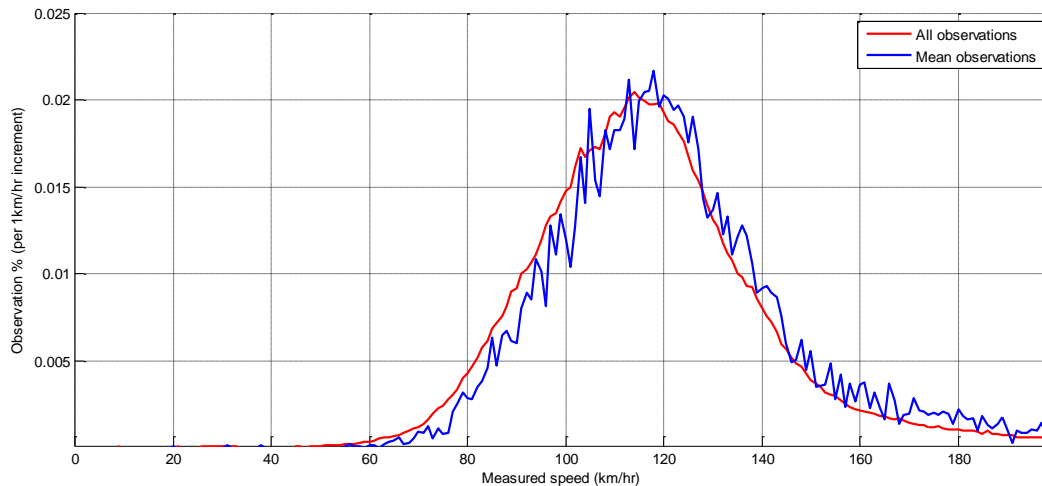


Figure 26 – Typical speed distribution profile (site A13-N-S3-1). In red: all speed observations of all objects; in blue: average speed of each unique object.

5.2 Conflict analysis results

A conflict analysis report is prepared for each video data set for a total of eight different sites, compared with one another as part of the control-case study (sites being picked for roughly identical geometric and flow conditions other than test variables). Furthermore, one of the sites has before and after treatment video data available and the reports for each are compared for a brief before-after study.

5.2.1 Control-case study

For the control-case study, eight sites with available video data are analysed. Table 10 lists details of all camera data extrapolated and analysed. Most filming and all conflict analysis is conducted for LOS A, B, or C conditions. There are a few reasons for this:

1. All, current vehicle tracking algorithms break down under heavy flow conditions. This is largely due to dense traffic resulting in closely-spaced vehicles, moving at the same slow speed and making it difficult for tracking algorithms to differentiate individual vehicles. Furthermore, stationary vehicles are not currently tracked by the video analysis tool.
2. The conflict method is not well adapted to stop-and-go traffic conditions where large fluctuations in TTC measures are frequent yet benign. This results in very noisy measures of little significance. Furthermore, low-flow collisions generally yield less-severe accidents (see section 3.1).
3. MTQ personnel agree that, during heavy traffic, queuing behaviour becomes predominant. This results in more lane-change-closure violations, as users begin to fight for room.
4. In any case, the treatment is originally designed to guide vehicles traveling at high speeds, with little time and opportunity to merge safely.

Furthermore, for equal sites of equal flow (total flow as well as exiting/entering ratio), the number of analysed objects will depend on whether the analysis area contains the ramp flow or not.

Table 10 – Control-case video data inventory.

| Site | Treatment | Objects per hour | μ Speed | σ Speed | Analysis s length | Analysis time |
|-------------|------------------|-----------------------------|-------------------------------|----------------------------------|------------------------------|--------------------------|
| A20-E-E56-3 | No | 2515 | 95 km/h | 11 km/h | 50 m | 5 h |
| A20-W-E62 | No | 2946 | 88 km/h | 16 km/h | 80 m | 3.6 h |
| A20-E-E58 | No | 2497 | 111 km/h | 15 km/h | 100 m | 5.3 h |
| A720-E-E3 | Yes | 2193 | 60 km/h | 10 km/h | 75 m | 6 h |
| A13-N-S3-1 | No | 2643 | 110 km/h | 14 km/h | 60 m | 3.6 h |
| A25-S-S5 | No | 3012 | 88 km/h | 10 km/h | 50 m | 4 h |
| A20-E-S58 | Yes | 2146 | 110 km/h | 11 km/h | 70 m | 4.6h |
| A25-N-S5 | Yes | 2388 | 90 km/h | 9 km/h | 50 m | 3.2h |

The following pages contain the conflict reports for each site. Following the reports is a discussion of each as well as a comparative discussion and general analysis conclusions.

Ramp: A20-E-E56-3
Treatment: No

Region(s): UPreMZ
Analysis length: 50 m

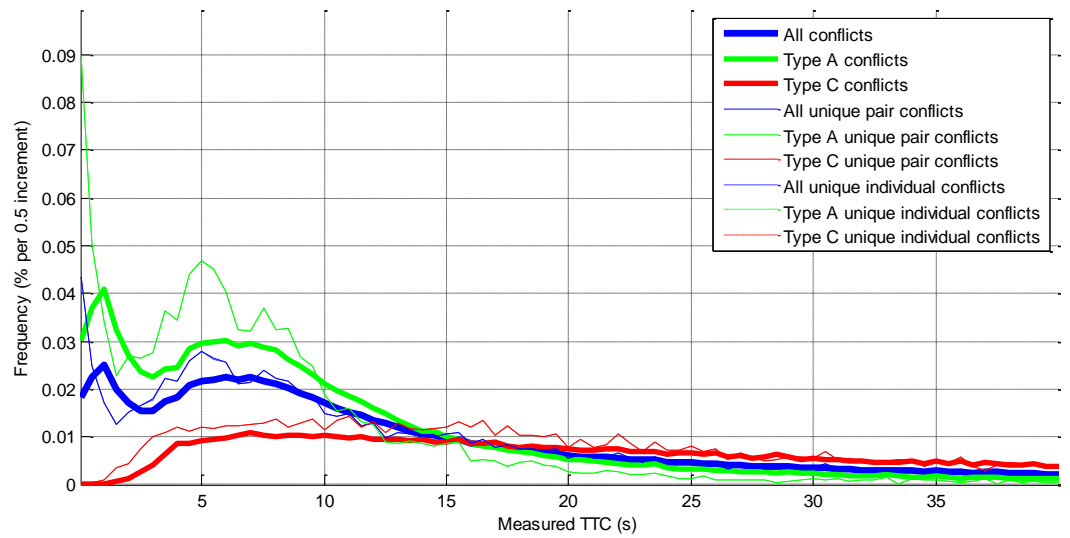
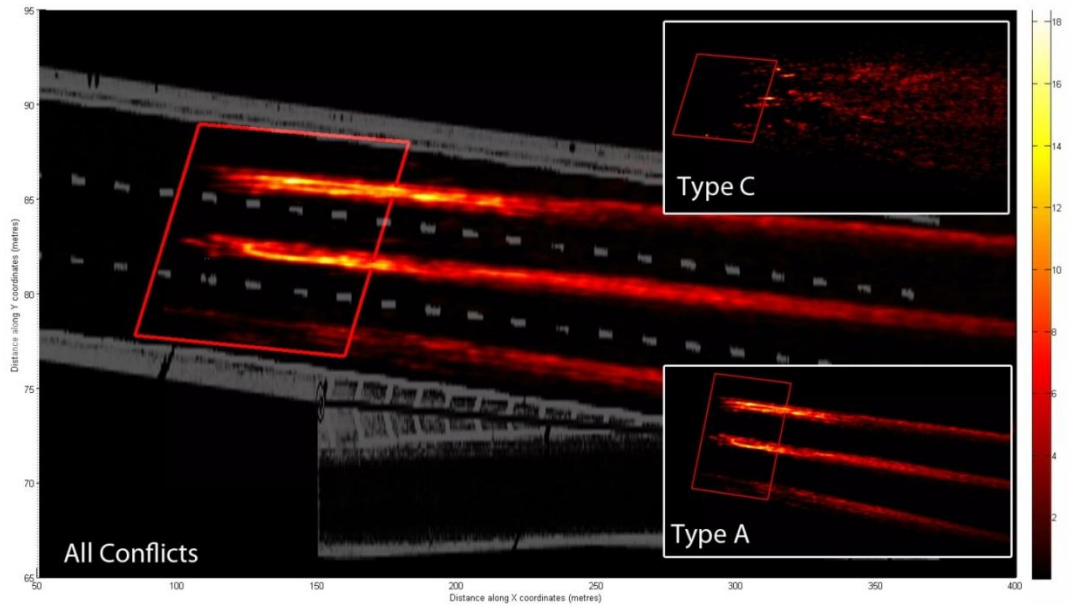
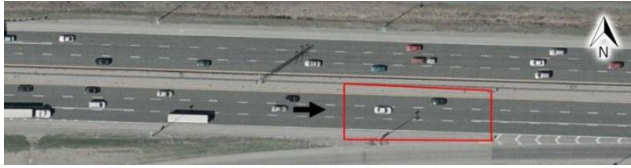


Figure 27 – Conflict analysis Cam20-16-Dorval (Untreated).

Ramp: A20-W-E62
Treatment: No

Region(s): UPreMZ
Analysis length: 80m

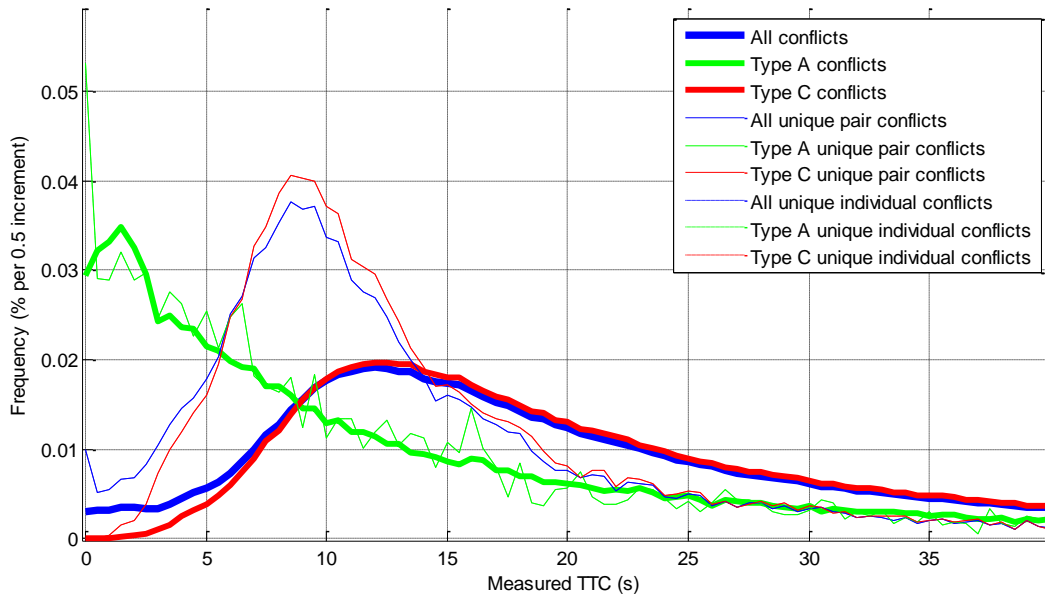
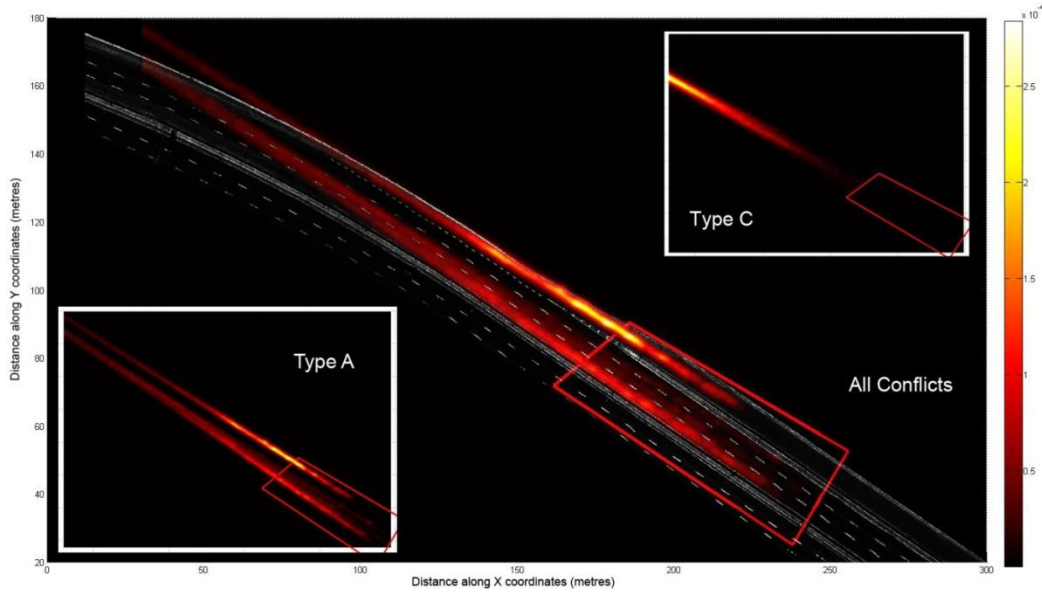


Figure 28 – Conflict analysis MTG-Cam20-1ere (untreated).

Ramp: A20-E-E58

Region(s): UPreMZ

Treatment: No

Analysis length: 100 m

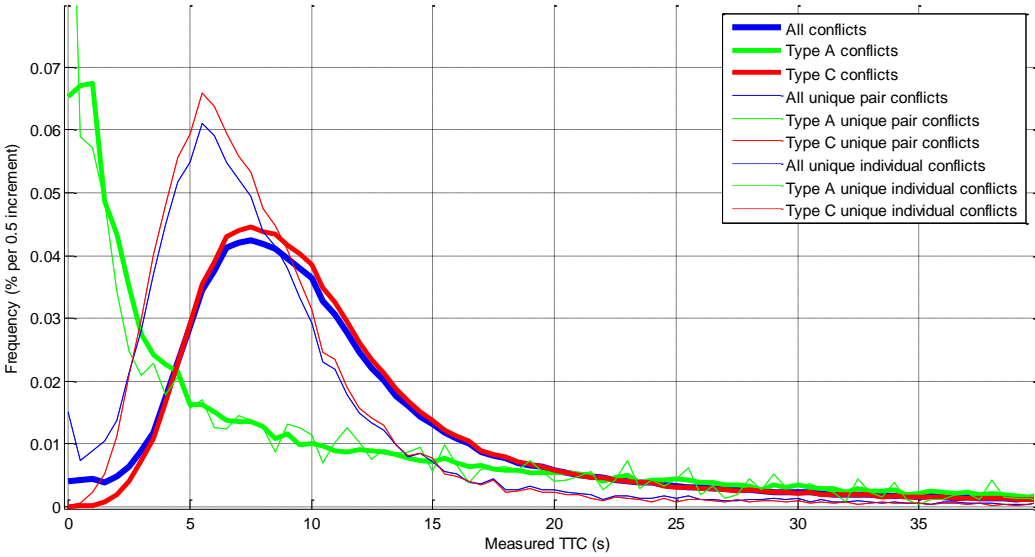
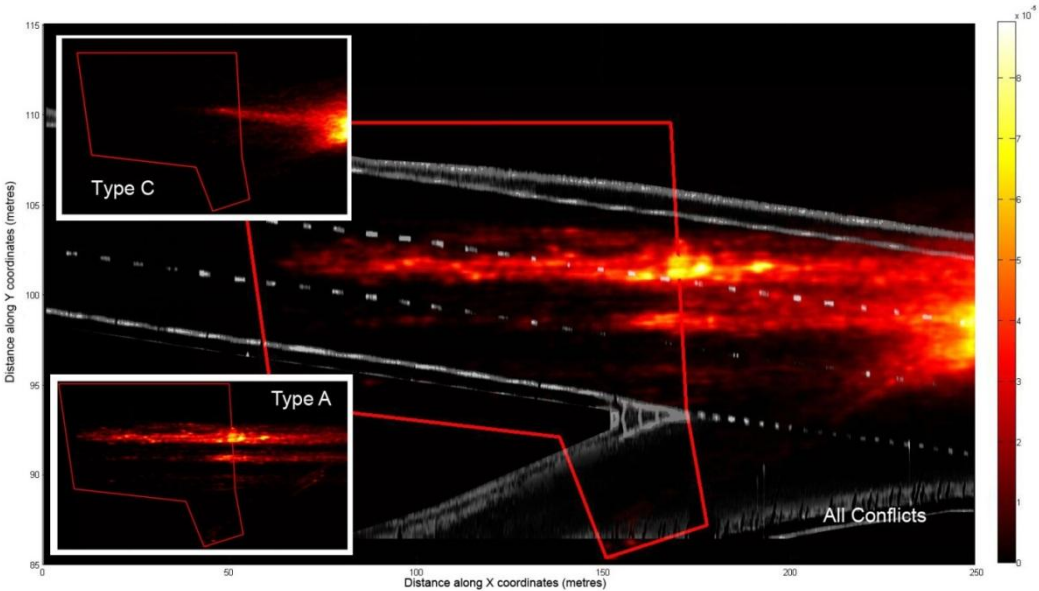


Figure 29 – Conflict analysis MTG-Cam20-E-55e-Entree (untreated).

Ramp: A720-E-E3 **Region(s):** PPreMZ, PMZ
Treatment: Yes **Analysis length:** 75 m

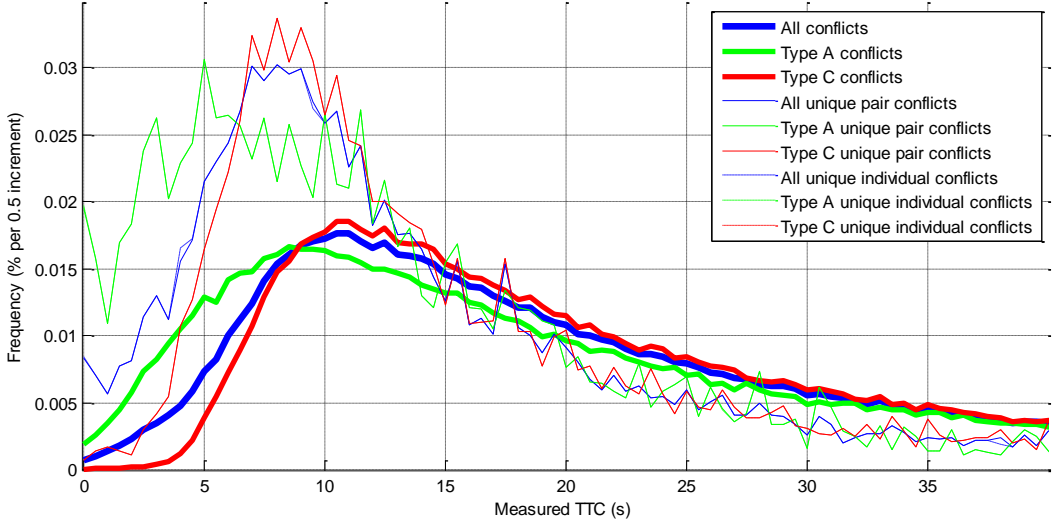
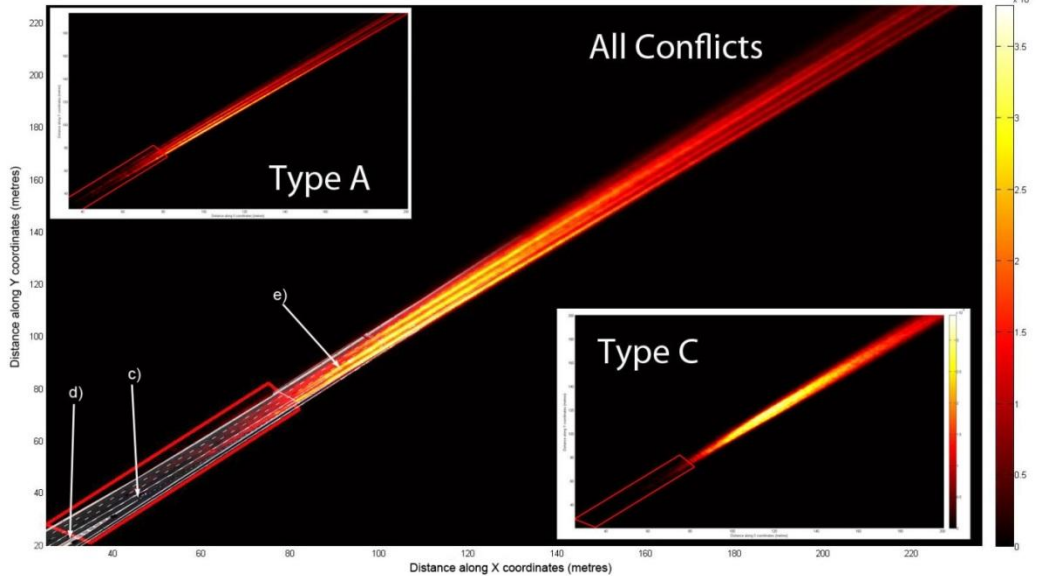


Figure 30 – Conflict analysis Cam720-05-Green (treated).

Ramp: A13-N-S3-1
Treatment: No

Region(s): PEZ, PPostEZ
Analysis length: 60m

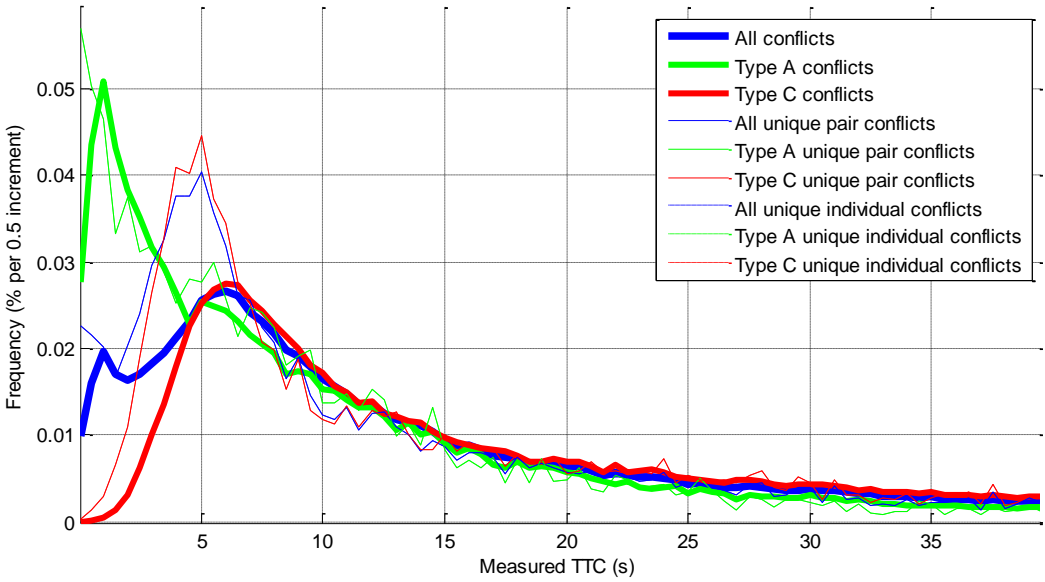
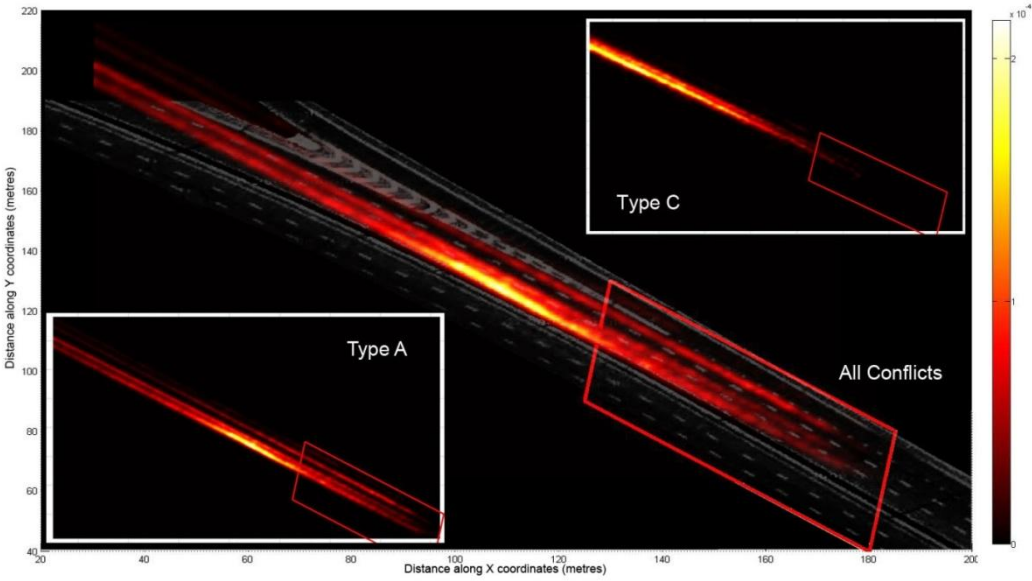


Figure 31 – Conflict analysis MTG-Cam13-520E-Sortie (untreated).

Ramp: A25-S-S5
Treatment: No

Region(s): PEZ, PPostEZ
Analysis length: 50m

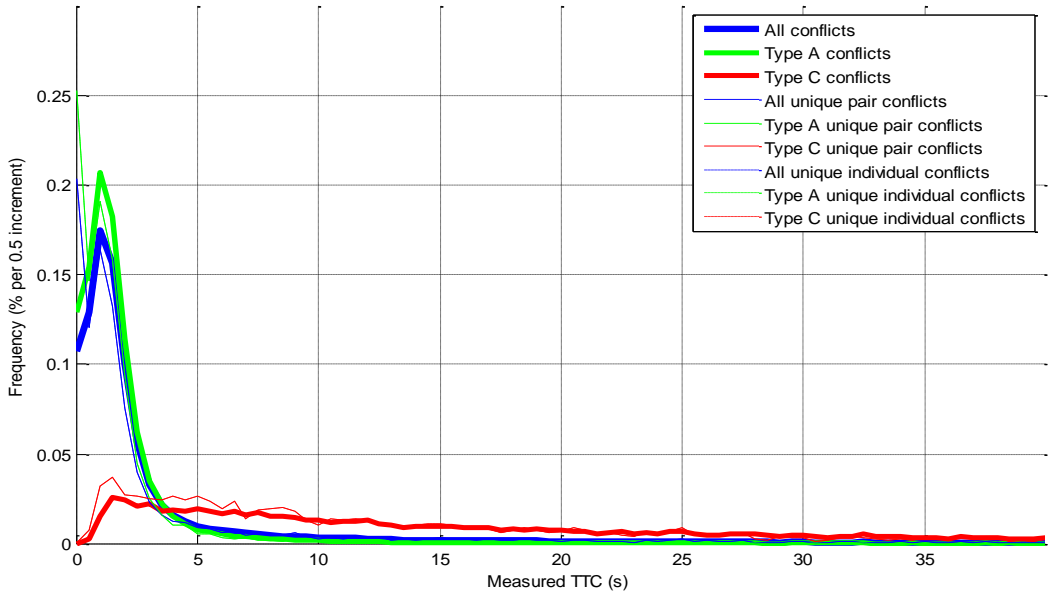
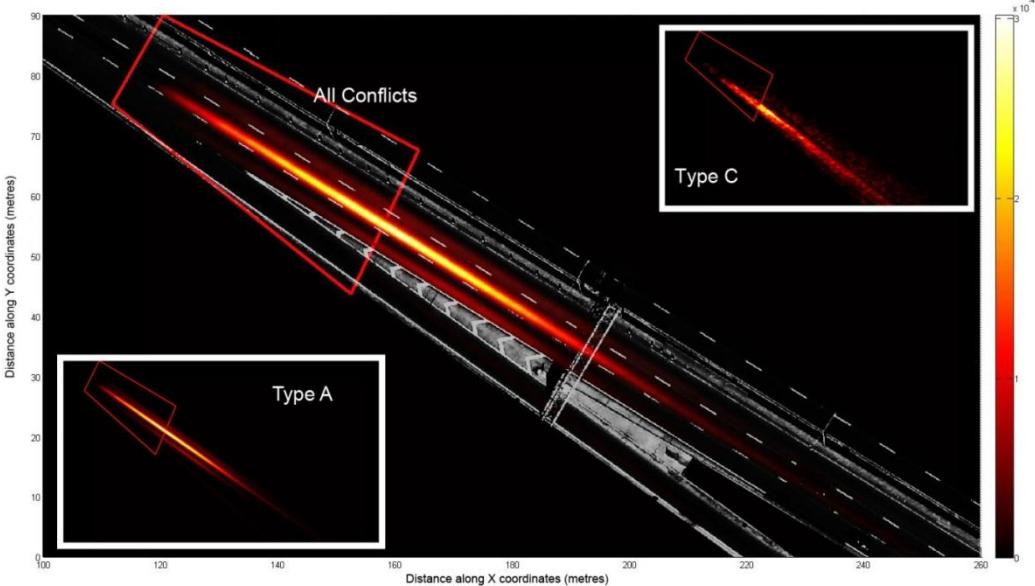


Figure 32 – Conflict analysis MTG-Cam25-S-Sherbrooke-Sortie (untreated).

Ramp: A20-E-S58
Treatment: Yes

Region(s): PPreEZ
Analysis length: 70 m

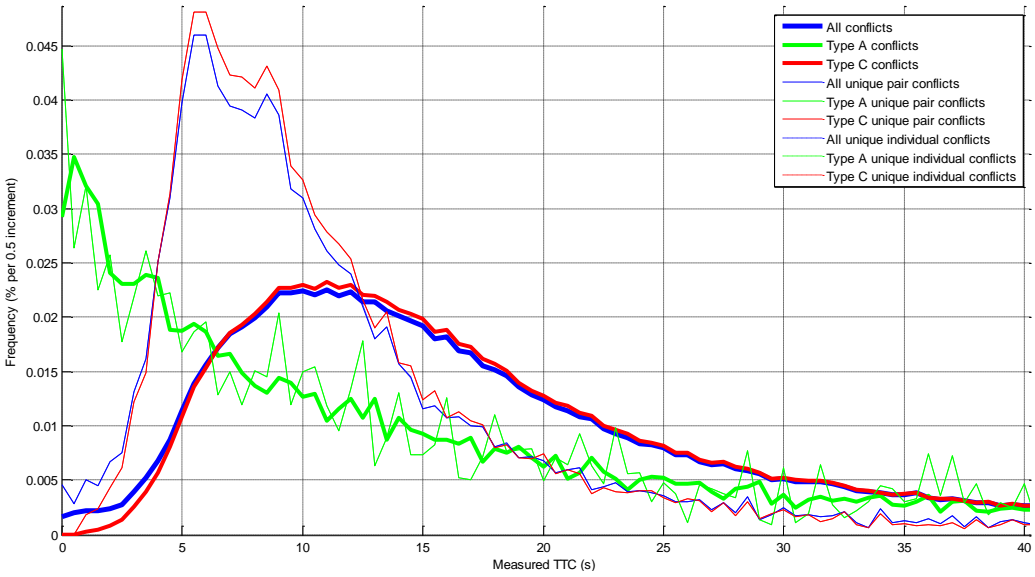
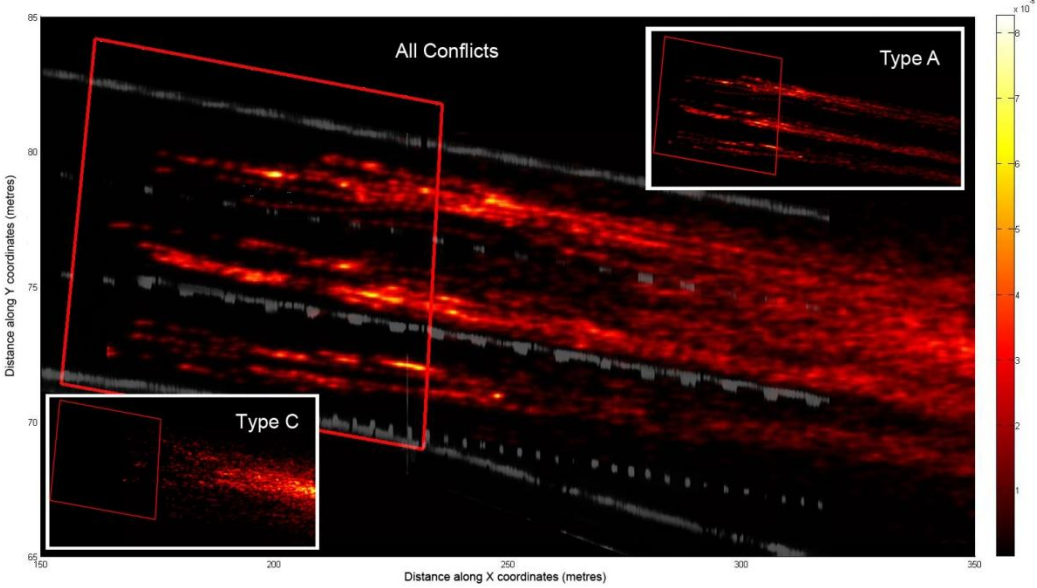


Figure 33 – Conflict analysis MTG-Cam20-E-55e-Sortie (treated).

Ramp: A25-N-S5
Treatment: Yes

Region(s): PEZ, PPostEZ
Analysis length: 50 m

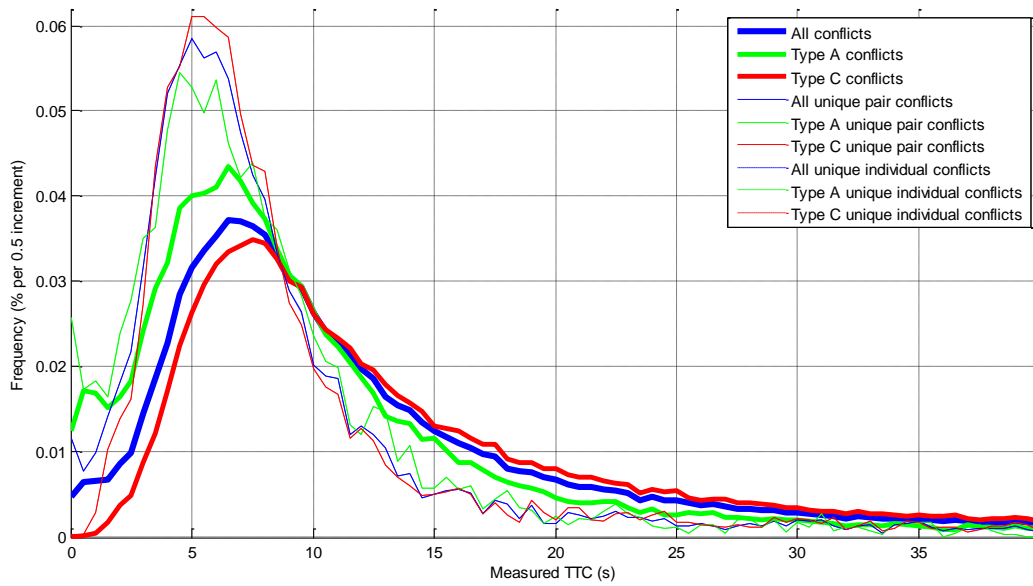
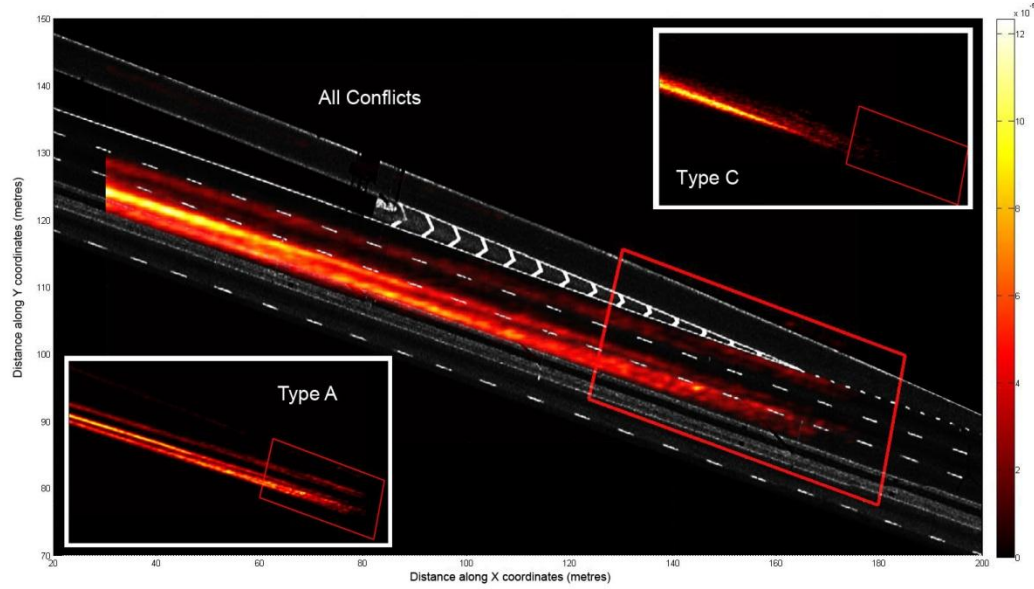


Figure 34 – Conflict analysis MTG-Cam25-N-Sherbrooke-Sortie (treated).

Figure 27 provides the conflict analysis report for A20-E-E56-3 (“Dorval”) using Cam20-16-Dorval (Untreated) and is an example of a UPreMZ analysis at an untreated entrance. It features heat trails in the second and third lanes, beginning inside the UPreMZ and extending into the PPreMZ and PMZ, particularly in the third lane. Comparing conflicts by type, from both the heatmap and conflict distribution we clearly see a dominance of Type A conflicts. Type C conflicts have a generally uniform pattern, with some concentrations at the UPreMZ boundary, particularly in the second and third lane. Conflicts distributions are generally centred around 5-8 seconds, TTC, with a unique secondary peak centred around 1 second, TTC, for Type A conflicts.

Figure 28 provides the conflict analysis report for A20-W-E62 (“Iiere”) using MTG-Cam20-Iere and is an example of a UPreMZ analysis at an untreated entrance. This site features heat in the third lane as well as the merging ramp, almost exclusively attributed to Type A conflicts. Observing the distribution graph, Type A conflicts peak between 0 and 2 seconds, while Type C conflicts peak at around 10 seconds. The exponentially decaying weight attributed to conflicts explains the negligible role of Type C conflicts for the site. It should also be mentioned that this site has a shorter-than-average merging length (*l_diffuseur*) and results in important Type A conflicts inside the merging ramp. Furthermore, this site roughly marks the transition from a 70 km/h posted speed limit to a 100 km/h posted speed limit on an important vertical curve, which might explain heavy Type A conflicts in the third lane, generally associated with passing cars and faster speeds.

Figure 29 provides the conflict analysis report for A20-E-E58 (“55e-Entree”) using MTG-Cam20-E-55e-Entree and is an example of a UPreMZ analysis at an untreated entrance. This site features an important vertical curve on the upstream portion of the ramp with narrow shoulders. The heavy merge conflicts associated with the tightness of the ramp itself and short approach visibility (*l_visibilite*) appear to have resulted in the application of a unique continuous line extending beyond the painted chevrons situated between the

traditional $l_{visibilite}$ and $l_{diffuseur}$ of approximately 50 metres in length (not shown on any available satellite imagery). For the record, $l_{diffuseur}$ begins immediately after the painted chevrons and includes this continuous line, as a large proportion of drivers appear to use this segment for merges in any event. In any case, the conflicts associated with limited approach visibility are clearly visible in the conflict analysis report. There is an overall tendency for vehicles to merge inside (towards the third lane) on approach of the ramp. Furthermore, the Type C conflicts of vehicles approaching from the merge ramp have a median of approximately 7 seconds, TTC. Despite no presence of an LCGV1 treatment, vehicles appear to move towards the inside lanes naturally to make way for entering vehicles (details in section 5.2.3) and in anticipation of a major junction from the outside 1200 metres downstream. Type A conflicts appear mostly in the third lane with a concentration of around 2 seconds, TTC.

Figure 30 provides the conflict analysis report for A720-E-E3 (“Green”) using Cam720-05-Green and is an example of a PPreMZ and PMZ analysis at a treated entrance. This site is notable for having four lanes instead of three, with a lower posted speed limit, and has an important downstream exit nearby. Consequently, differences in driver behaviour stand out: conflicts are equally spread between Type A and Type C conflicts (both in terms of probable distribution and weighted density distribution) and heat appears mostly on the merging ramp and first lanes. Type C conflicts appear to be much more uniform and dense than previous sites. The conflict distribution of both types peaks at 10 seconds, TTC, which generally explains why conflict heat is pushed further downstream of the analysis region (points of conflict occur further away from the originating interactions).

Figure 31 provides the conflict analysis report for A13-N-S3-1 (“13-520”) using MTG-Cam13-520E-Sortie and is an example of a PEZ and PPostEZ analysis at an untreated exit with auxiliary lane. Having an auxiliary lane, this site is characterized by a long $l_{diffuseur}$. Together with a relatively low $q_{\% exiting}$, this site is expected to have few merging conflicts and the heatmap shows it.

Virtually no important conflicts are detected on the ramp, with the majority of Type A and Type C conflicts concentrated in the third lane, Type A conflicts concentrated around 2 seconds, TTC, and Type C conflicts concentrated around 6 seconds, TTC.

Figure 32 provides the conflict analysis report for A25-S-S5 (“Sherbrooke-South”) using MTG-Cam25-S-Sherbrooke-Sortie and is an example of a PEZ and PPostEZ analysis at an untreated exit. This site is dominated by Type A conflicts (peaking at about 2 seconds, TTC) mostly situated in the second lane. A few type C conflicts can be found in inside of the ramp, possibly due to some increased trajectory noise at this site (flat curve).

Figure 33 provides the conflict analysis report for A20-E-S58 (“55e-Sortie”) using MTG-Cam20-E-55e-Sortie and is an example of a PPreEZ analysis at a treated exit. It features noisy patterns of conflict density uniformly distributed between all three lanes for both Type A and Type C conflicts. Type A conflicts are predominant with a steadily decreasing concentration of TTC starting at 1 second, TTC, while Type C conflicts are concentrated almost normally around 11 seconds, TTC.

Figure 34 provides the conflict analysis report for A25-N-S5 (“Sherbrooke-North”) using MTG-Cam25-N-Sherbrooke-Sortie and is an example of a PEZ and PPostEZ analysis at a treated exit. It features an equal mix of Type A and Type C conflicts, mostly located in the third lane, with almost no ramp conflicts. Conflicts of either type peak at around 7 seconds, TTC.

Table 11 summarizes briefly the previous observations for each conflict analysis report for convenient comparison. The first observation is that even roughly similar sites still have important differences. The predominant conflict type of entrances appears to be Type A (car following), while conflicts types at exits appear to be more uniform (car following and lane changing), although this observation is not particularly statistically significant (too few observations).

What does appear significant, however, is that neither entrances nor exits ever show predominantly Type C conflicts.

Except for A720-E-E3, which had important downstream exit queuing in lane 1, Type A conflicts are always the most predominant in the outermost lane. Knowing that speed is always greatest in the inner-most lanes, one might conclude from this that Type A conflicts are predominantly governed by speed.

Figure 35 and Figure 36 summarise all TTC cumulative distributions (including an additional observed distribution after-treatment for the A20-E-E56-3 (“Dorval”) site included in the next section) according to type A and type C conflicts respectively. Few patterns emerge other than to say that, overall, type A conflicts are more likely to generate a lower TTC than type C interactions. It should also be noted that although the raw number of hourly conflicts did vary from site to site by about 25% (and out of proportion to variation in traffic flow), the split between conflicting and diverging interactions always remained around 50% with variations no greater than .5% suggesting that variation in hourly raw number of conflict observations is simply attributed to a greater amount of interaction measurements, possibly generated from flow arrival variation only (for example, vehicles arriving more uniformly generate fewer interactions on a whole than clusters of vehicles, since the clusters of vehicles are more likely to be present in the analysis region together). It is difficult to attribute vehicle arrival rate with each small analysis region, since the vehicle arrival rate will depend on many upstream factors.

Table 11 – Conflict analysis report summary. Lists of lane concentrations are in order of importance from most to least (R= ramp lane).

| Site | Treatment | Design Notes | Analysis Area | Predominant Conflict Type | Type A conflicts | | | Type C conflicts | | |
|-------------|-----------|---|---------------|---------------------------|---------------------|----------------------|-----------|---------------------|------------------------------------|----------|
| | | | | | Lane Concentrations | Patterns | Peak TTC | Lane Concentrations | Patterns | Peak TTC |
| A20-E-E56-3 | No | Last of three consecutive entrances; site is normally treated; minor construction upstream; nearby exit downstream (<500 m) | UPreMZ | Type A | 3, 2, 1 | Smooth streaks | 5-8 s, 1s | 1 and 2 and 3 | Noisy uniform | 5-8 s |
| A20-W-E62 | No | Just upstream of an important vertical curvature; speed limit increase | UPreMZ | Type A | R, 3, 2 | Smooth streaks | 0-2 s | 3 | Smooth streak | 10 s |
| A20-E-E58 | No | Important ramp vertical curvature; tight ramp; extended solid line after chevrons; nearby exit upstream (<500 m) | UPreMZ | Type A | 3, 2 | Noisy angled streaks | 2 s | R, 3 | Smooth hotspot with angled streaks | 7 s |
| A720-E-E3 | Yes | 4 lanes, 70 km/h speed limit | PPreMZ, PMZ | Neither | R, 1, 2, 3 | Smooth streaks | 10 s | R, 1, 2 | Smooth uniform | 10 s |
| A13-N-S3-1 | No | 339 m-long auxiliary lane; low $q\%$ exiting | PEZ, PPostEZ | Neither | 3, 2 | Smooth streaks | 2 s | 3 | Smooth streak | 6 s |
| A25-S-S5 | No | Short merging lane | PEZ, PPostEZ | Type A | 2 | Smooth streaks | 1 s | 1 | Noisy streaks | 2 s |
| A20-E-S58 | Yes | Nearby entrance upstream (<500 m) | PPreEZ | Type A | 1 and 2 and 3 | Noisy streaks | 1 s | 1 and 2 and 3 | Noisy uniform with minor hotspot | 11 s |
| A25-N-S5 | Yes | Low observed speed | PEZ, PPostEZ | Neither | 3, 2 | Smooth streaks | 7 s | 3 | Noisy streak | 7 s |

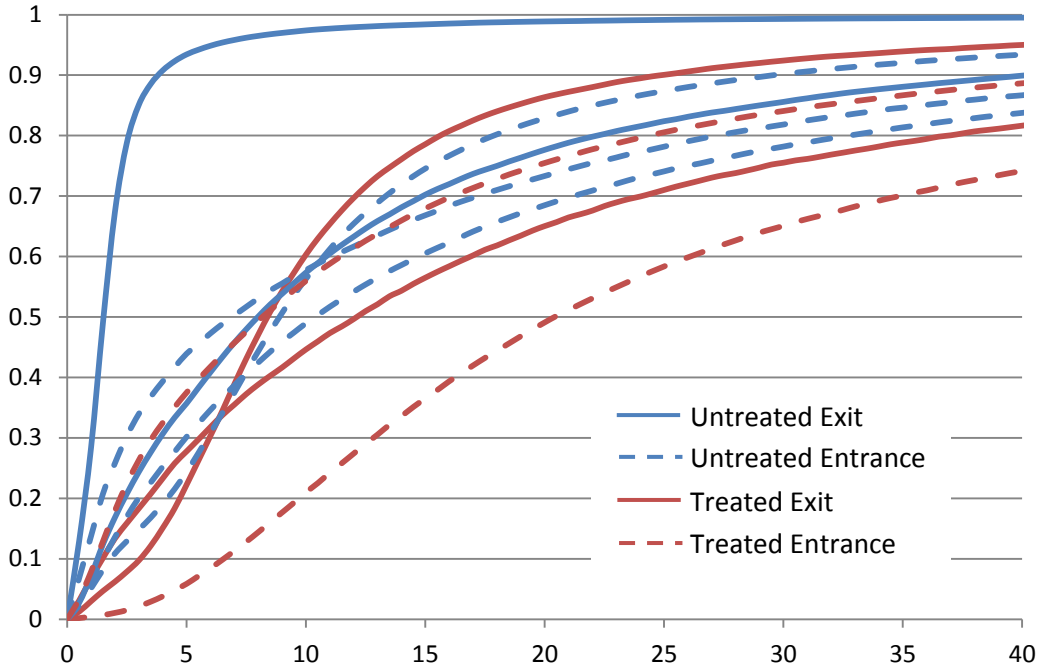


Figure 35 – Cumulative distribution of type A TTC as a percentage of observations until 200 seconds, TTC.

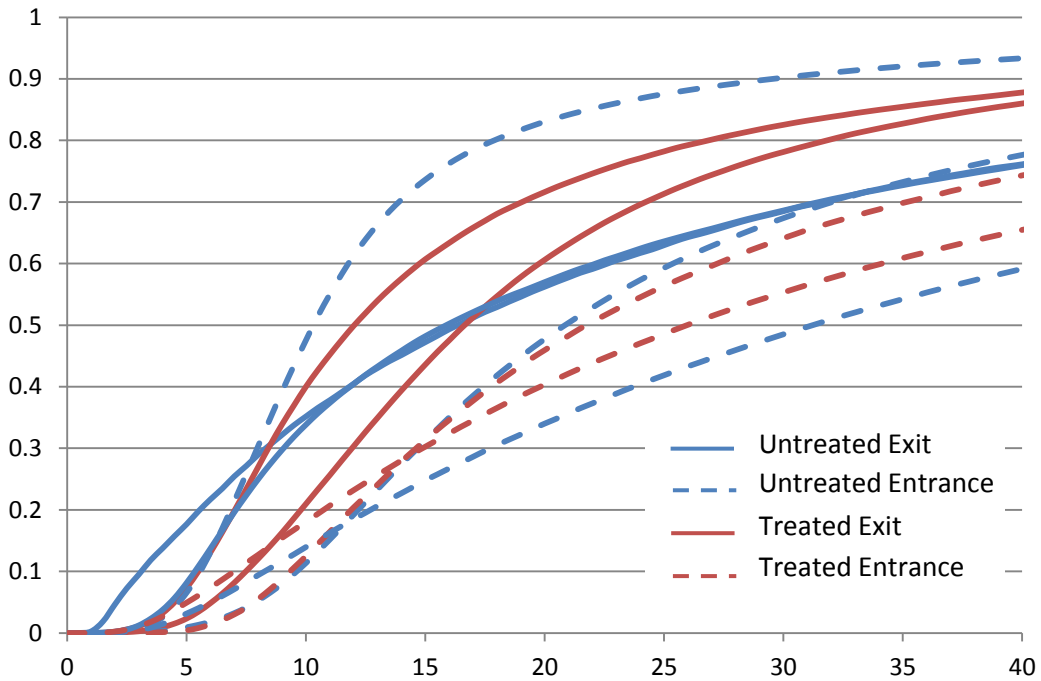


Figure 36 - Cumulative distribution of type C TTC as a percentage of observations until 200 seconds, TTC.

5.2.2 Before and after study

One of the major challenges of the study was to evaluate the effect of the treatment without experimenting on the existing infrastructure. This meant that no highway ramps could have the treatment added, or removed if they already were treated. Fortunately, one site had its treatment temporarily removed during filming as part of construction to the next ramp upstream in that direction.

Although this gave a unique opportunity to study before and after the treatment for a single site, caution must be used as the presence of construction activities at the upstream ramp may have provided some changes in driver behaviour during this time. Regardless, the after-treatment video provides an additional treated entrance to the list of sites analysed in the control-case study.

Table 12 – Before-after video data inventory.

| Site | Treatment | Objects per hour | μ Speed | σ Speed | Analysis length | Analysis time |
|-------------|-----------|---------------------|-------------|----------------|--------------------|------------------|
| A20-E-E56-3 | No | 2515 | 95 km/h | 11 km/h | 50 m | 5 h |
| A20-E-E56-3 | Yes | 2598 | 105 km/h | 12 km/h | 50 m | 10 h |

Figure 37 provides the conflict analysis report for A20-E-E56-3 (“Dorval”) using Cam20-16-Dorval (Treated) and is an example of a UPreMZ and PPreMZ analysis at a treated entrance (analysis regions and view equivalent to those of the conflict analysis report for the same site in section 5.2.1). It features a predominant concentration of Type A conflicts, mostly located in the second lane, with additional conflicts in the first lane. Type A conflicts peak at around 2 seconds, TTC. Type C conflicts are heavily concentrated in the second lane, right at the beginning of the treatment. Type C conflicts peak at around 9 seconds, TTC.

Comparing and contrasting the conflict analysis reports for the untreated and treated A20-E-E56-3 (“Dorval”) examples (Figure 27 and Figure 37, respectively), we remark an important distinction between conflict distributions by lane. The untreated site features Type A conflicts predominantly in the third and second lanes, whereas the treated site features Type A conflicts

predominantly in the second and first lanes. Type C conflicts are largely uniformly distributed (albeit with some noise) in the untreated case, whereas the same type of conflicts for the treated case are clearly clustered around the second lane (with some minor spill over into the third lane). This evidence supports the idea that the beginning of the treatment provides a critical point for drivers (i.e. forcing navigation decisions to occur at a single point instead of uniformly and naturally along an entire segment).

Comparing conflict measure distributions, a slight shift in Type A conflicts towards smaller measures (e.g. hypothetically greater probability of collision) manifests itself after treatment (though this may be a symptom of a higher mean and standard deviation of speed). Meanwhile the distribution of Type C measures remains largely unchanged.

Ramp: A20-E-E56-3
Treatment: Yes

Region(s): UPreMZ, PPreMZ
Analysis length: 50 m

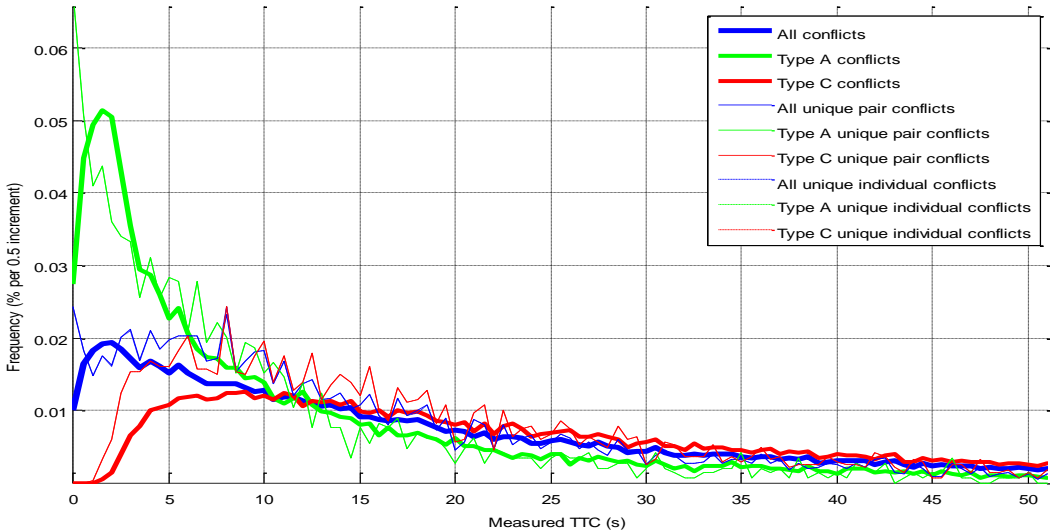
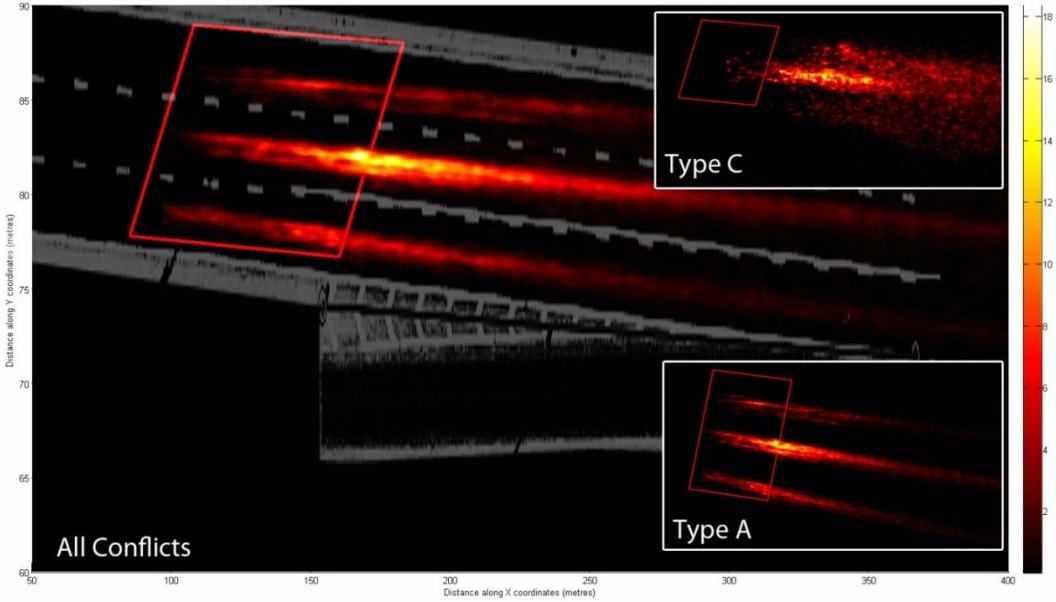
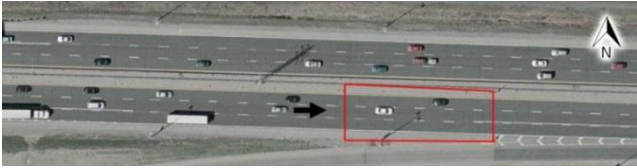


Figure 37 – Conflict analysis Cam20-16-Dorval (Treated).

5.2.3 Supplementary analysis

The videos also provide a valuable source of behavioural information in the areas of traffic flow theory.

Table 13 provides measured flow ratios by lane as well as the $q\%$ exiting/entering. Each flow ratio is averaged over 50 to 100 metre sections (according to each study area) with little observed variability ($\pm 2\%$ for most observations, see Figure 41 of the appendix for an example). Lane 2 is the busiest lane with an average of 41% of all flows followed by lane 3 with an average of 32% of all flows. Significant variability of flow ratios from site to site still exists however. Interestingly, the flow ratio of lane 1 for treated sites is somewhat higher. This observation is earlier reflected in the increased amount of observed conflicts in the first lane. This observation is surprising, considering the purpose of the treatment, although it is not a significant one, nor can we rule out collinearity with other factors such as overlapping influence zones.

Table 13 – Measured mean flow ratios by lane. Sections filmed directly over ramps include the ratio of the total measured ramp flow over the measured total highway flow at the same point.

| Site | Treatment | $q\%$ | Mean % flow by lane | | | |
|-------------|-----------|-------|---------------------|--------------|--------------|--------|
| | | | Lane 1 | Lane 2 | Lane 3 | Lane 4 |
| A20-E-E56-3 | No | - | 27.5% | 43.9% | 28.6% | - |
| A20-W-E62 | No | 24.0% | 21.3% | 42.2% | 36.5% | - |
| A20-E-E58 | No | 10.9% | 12.3 % | 33.5% | 54.2% | - |
| A720-E-E3 | Yes | 3.0% | 31.4% | 29.6% | 25.8% | 13.2% |
| A20-E-E56-3 | Yes | - | 32.5% | 43.2% | 24.3% | - |
| Mean | - | 12.6% | 25.9% | 39.4% | 34.7% | - |
| A13-N-S3-1 | No | 5.9% | 17.0% | 36.9% | 46.1% | - |
| A25-S-S5 | No | 18.2% | 38.5% | 46.5% | 15.1% | - |
| A20-E-S58 | Yes | - | 27.2% | 44.8% | 28.0% | - |
| A25-N-S5 | Yes | 9.7% | 23.4% | 44.5% | 32.1% | - |
| Mean | - | 11.3% | 26.5% | 43.2% | 30.3% | - |

Table 14 provides measured mean trajectory standard deviations for each lane. A vehicle's offset from the center of a lane is generally normally distributed. The standard deviation measures the boundary on either side of the center

alignment delimiting one standard deviation (34.1%) of the vehicles' trajectories per side. Effectively this means that 64.2% of all trajectories are located in the region bound by +/- 1 standard deviation around the center alignment. Similarly, 95.4% of all trajectories are located in the region bound by +/- 2 standard deviations around the center line. The remaining trajectories are generally the result of ongoing lane changes. As a reminder, trajectories are mapped to the geometric center of the vehicle's volume both in length and in width. It is worth mentioning that the inside-most lanes (furthest from the camera) as well as data from the mobile video collection unit have the greatest parallax error. This results in artificially inflated trajectory standard deviations at these locations and caution should be exercised when interpreting these results.

Table 14 – Measured mean trajectory standard deviation by lane. Sections filmed directly over ramps include data for ramps (and highway lanes opposite the ramp). Measurements in metres.

| Site | Treatment | Mean Trajectory std. dev. by lane (m) | | | | |
|-------------|-----------|---------------------------------------|--------|--------|--------|--------|
| | | Ramp | Lane 1 | Lane 2 | Lane 3 | Lane 4 |
| A20-E-E56-3 | No | - | 0.411 | 0.401 | 0.381 | - |
| A20-W-E62 | No | 0.563 | 0.664 | 0.511 | 0.763 | - |
| A20-E-E58 | No | 0.262 | 0.397 | 0.488 | 0.537 | - |
| A720-E-E3 | Yes | 0.366 | 0.253 | 0.212 | 0.197 | 0.130 |
| A20-E-E56-3 | Yes | - | 0.468 | 0.473 | 0.340 | - |
| Mean | - | 0.397 | 0.439 | 0.417 | 0.444 | 0.130 |
| A13-N-S3-1 | No | 0.276 | 0.434 | 0.501 | 0.664 | - |
| A25-S-S5 | No | 0.183 | 0.379 | 0.399 | 0.623 | - |
| A20-E-S58 | Yes | - | 0.465 | 0.455 | 0.490 | - |
| A25-N-S5 | Yes | 0.749 | 0.414 | 0.440 | 0.660 | - |
| Mean | - | 0.403 | 0.423 | 0.449 | 0.609 | - |

Table 15 lists the observed lane change rate in lane changes per vehicle-kilometre (lc/veh-km) according to originating and destination (lane changes over multiple lanes are broken down into individual lane changes with each adjacent lane). As a reminder, the lane change that is protected by the LCGV1 treatment is the lane change from lane 2 to lane 1. The lane change rate represents the number of individual lane changes that occur for every veh-kilometre driven. As with

every other trajectory-based measure, this is an aggregated average over the study area. It is also observed to be generally uniform over the length of the study area.

Table 15 – Lane changes per vehicle-kilometre (lc/veh-km) **Lane changes from the second to the first lane are those which the treatment is designed to forbid.

| Site | Treatment | 1 → 2 | 2 → 3 | 3 → 4 | 4 → 3 | 3 → 2 | 2 → 1** |
|-------------|-----------|--------------|--------------|-------|-------|-------|--------------|
| A20-E-E56-3 | No | 0.148 | 0.192 | - | - | 0.097 | 0.136 |
| A20-W-E62 | No | 0.126 | 0.327 | - | - | 0.120 | 0.103 |
| A20-E-E58 | No | 0.024 | 0.068 | - | - | 0.096 | 0.140 |
| A720-E-E3 | Yes | 0.151 | 0.091 | 0.046 | 0.025 | 0.065 | 0.093 |
| A20-E-E56-3 | Yes | 0.118 | 0.068 | - | - | 0.053 | 0.139 |
| A13-N-S3-1 | No | 0.175 | 0.345 | - | - | 0.344 | 0.126 |
| A25-S-S5 | No | 0.184 | 0.162 | - | - | 0.042 | 0.279 |
| A20-E-S58 | Yes | 0.126 | 0.097 | - | - | 0.099 | 0.176 |
| A25-N-S5 | Yes | 0.184 | 0.510 | - | - | 0.225 | 0.145 |

As with everything else, there is no clear pattern in lane changes according to treatment. What is clear, however, is the fact that many users, and sometimes even the majority of users, choose to change lanes over the treatment regardless. This could explain why the treatment isn't appreciably effective at reducing accidents at all sites: any theoretical benefits on accident rates through the prevention of lane changes might simply be offset by a lack of compliance with the treatment. This is however difficult to explore with the current dataset.

Chapter 6: Conclusions and directions for future work

6.1 Research Conclusions

From the historical cross-sectional accident analysis we conclude that the overarching safety benefits of the treatment are found to be negligible or statistically insignificant for the sites under analysis.

- Some evidence suggests that the presence of the treatment has a positive effect at some sites while other evidence suggests the contrary for other sites. More specifically, the impact of the treatment appears to be positive for entrance ramps and negative for exit ramps. However, in most of the cases, the impact of the treatment is not statistically significant.
- More importantly, the safety of a ramp appears to depend more on other geometric factors after controlling for traffic conditions. Some contributing factors that seem to play an important role include the distance to the upstream ramp and visibility length of the painted chevrons. There is, however, insufficient data (too few sites) to make conclusions with regard to the effects of the treatment vis-à-vis these individual factors.
- Moreover, for entrance ramps, safety deteriorates as upstream flow and/or the flow ratio on the ramp increases. This observation is in accordance with international consensus.
- Some minor migration effects are observed, particularly upstream of exits.
- Presence of the treatment has no appreciable effect on accident severity. Treated and non-treated sites have similar accident severity patterns. These conclusions, however, are not as in-depth as those for accident probability.

From the video-based driver behaviour analysis we conclude that, in general, most observations support the conclusion of the historical cross-sectional accident analysis. There exists no clear pattern relating the presence of the treatment to distributions of time-to-collision or other microscopic

conflict/exposure measures. Time-to-collision distribution appears to be more accurately explained by similar factors found in the historical accident analysis.

- Based on the before-after analysis of one single entrance (A20-E-E56-3 “Dorval/Bouchard”), the time-to-collision distribution after treatment is observed to decrease, suggesting that safety at this site deteriorates.
- The study area for this particular before-after case was located at the transition point between an unprotected area and a protected area (UPreMZ followed by a PPreMZ) highlighting the theory that the beginning of the LCGV1 treatment may pose a critical point where conflicts concentrate. We observe that, with the treatment, conflict concentrations shift from the third lane into the first lane, with a relative increase in the second lane. Type C conflicts (lane-changes) become very predominant at the beginning of the LCGV1 treatment and this conflict migration may have some explaining power to the overall accident migration noted in the historical accident analysis.
- For some sites, lane changes occur across the lane-change ban regardless of its presence. In fact some treated sites feature the greatest amount of lane changes from the second lane to the first. The lane change rate seems to be more associated to geometric factors including influence zone overlap.
- Rear-end conflicts are generally more predominant than merging conflicts for all site types, both in number of observations and in magnitude of measure (rear-end conflicts are more likely to have a low TTC).
- Overall, we conclude that few driver behaviour patterns associated with the treatment emerge. Some evidence suggests that the presence of the treatment has a positive effect while other evidence suggests the contrary.

6.2 Immediate recommendations

Because no general justification for the use of the LCGV1 treatment at “normal” sites is found, and there is some evidence to suggest the LCGV1 can potentially negatively impact traffic flow and safety in certain situations, the practice of indiscriminately applying the treatment at sites is strongly discouraged. Specifically:

- Generally, the treatment has the disadvantages of restricting driver navigation (which can lead to driver frustration or distraction) and creates artificial critical points inside of highway segments. From a safety point of view, these relevant costs have to be weighed with the relevant benefits associated with restricting driver behaviour for particular environments.
- Thus, the use of the LCGV1 can be justified on a case-by-case basis, particularly for urban environments (or “underdesigned” environments), but its formulaic use is discouraged.
- Items recommended for analysis include upstream and downstream ramp distances, chevron length, merging length, measures of visibility between highway lanes and ramps, and vertical signage.
- It is inferred that properly designed interchanges with limited area of influence overlap (e.g. rural or suburban environments) generally do not benefit from the presence of the treatment, however they may run the risk of inheriting the problems associated with the treatment. A similar study of rural sites is needed to confirm this.
- For the urban exits under analysis, the removal of any length of treatment beyond (downstream of) painted chevrons is recommended, for these segments serve no behavioural purpose other than to restrict navigation and frustrate drivers (the original design for exits is out-of-date according to HCM 2000, although the applicability of the area of influence for entrances as defined by HCM 2000 is questionable).
- The length of the LCGV1 design should be optimized such that only the minimum length of treatment necessary is justified for application. Also,

establishment of a minimum length of untreated road length upstream of treated exits is recommended such that drivers are not “locked out” of the exit before the first *prominent* occurrence of vertical signage when another treated ramp precedes nearby.

Furthermore, the study questions the practice of defining and finding a “standard” urban ramp and whether or not such a definition is relevant, as the majority of ramps in urban areas have overlapping areas of influence, varying design properties, and all are situated in appreciably unique environments.

6.3 Discussion on the use of video-based conflict analysis

This research project has provided ample practical experience regarding video-based conflict analysis. There are many hurdles to overcome and the methodology incorporates a wide range of scientific fields beyond the scope of this paper including computer vision sciences, electrical engineering, data mining and management techniques, road safety engineering, and human behavioural sciences, but vehicle trajectories collected from video sensors offer a rich source of driving behaviour information and successful research into the conflict analysis has the potential of turning reactive road safety analysis into proactive safety analysis.

It is quite clear that transportation infrastructure will evolve to be more intelligent over the coming decades, by use of, for example, video tracking, as computer vision algorithms evolve (limited perhaps only by privacy concerns). The usefulness of the conflict analysis technique is still, however, somewhat debatable as the degree of relevance of conflicts as a consistent measure of accident probability is still an open question. At the very least, TTC specifically offers a measure of driver behaviour alongside other metrics such as following distance or speed and will always be of some use to any study investigating changes in driver behaviour. Our current understanding of the theory however limits the application of the methodology in its current form to the comparison of similar sites only as the exact relationship between TTC, accident probability, and varying environments or interaction types has yet to be investigated and is subject to future research.

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Appendix



Figure 38 – Study sites categorised by *L_diffuseur*.

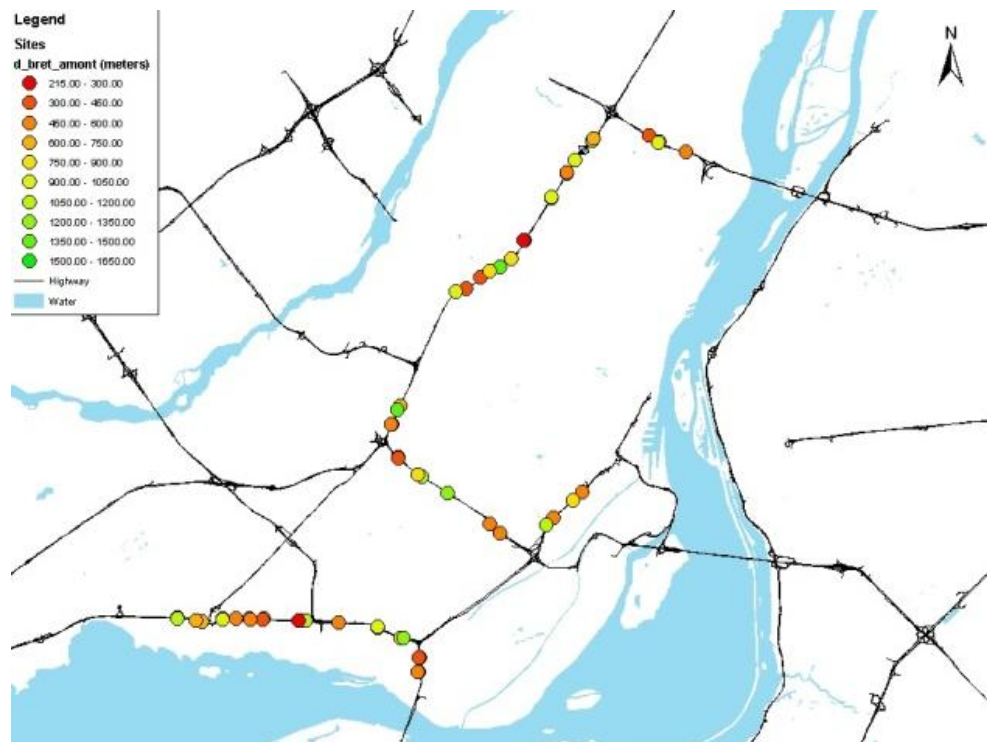


Figure 39 – Study sites categorised by *d_bret_amont*.



Figure 40 – Study sites categorised by $L_{visibilité}$.

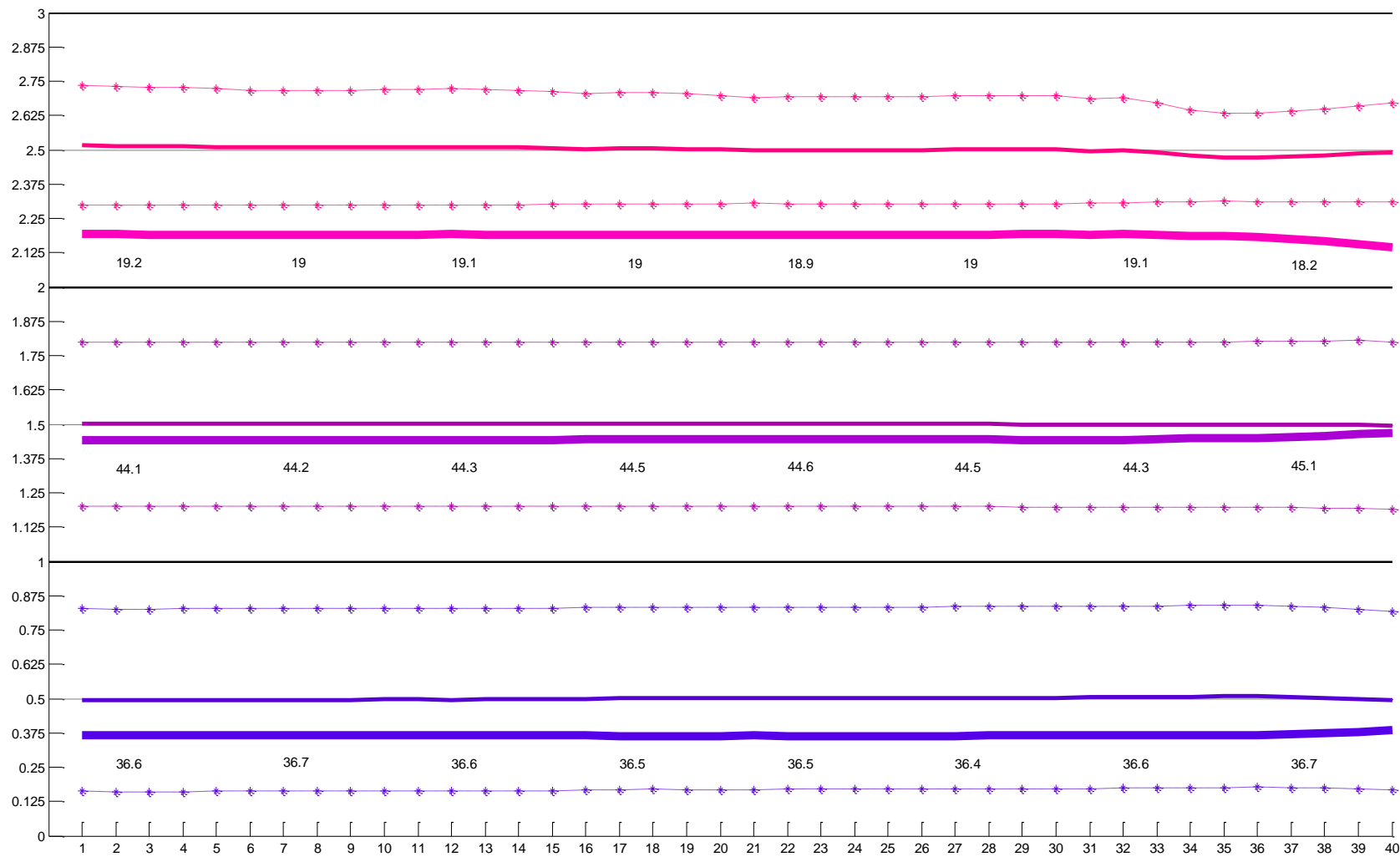


Figure 41 – Sample lane flow ratios and median trajectories. Site: A20-E-E56-3, Cam20-16-Dorval (Treated)

