A SAFETY ANALYSIS OF LEFT-TURNING MANEUVERS FOR LONG-COMBINATION VEHICLES

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Abstract

A growing number of goods are shipped in Ontario using long-combination vehicles (LCVs). LCVs can significantly increase the amount of cargo carried per shipment, while reducing shipping costs and environmental impacts of freight road travel. Questions remain regarding their safety due to their larger size and wider maneuvering. This thesis furthers our understanding of LCV safety considerations in last-mile areas during left-turning maneuvers by focusing on conflicts with infrastructure and road users. Intersections are selected along Peel Region's Strategic Goods Movement Network and left-turning swept path analyses are performed using AutoTurn. A microsimulation of the intersections is developed, where the existing network is modeled in Vissim while LCVs are added as potential road users. The Surrogate Safety Assessment Model is then used to analyze the microsimulation results pertaining to potential collisions. It was found that existing last-mile infrastructure is often ill-equipped to accommodate LCVs and requires adjustments for future LCV use.

Dedicated to my Family with Love

~

Dedicato alla mia Famiglia con Amore

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Table of Contents

List of Tables

List of Figures

List of Acronyms and Abbreviations

AASHTO – American Association of State Highway and Transportation Officials

AGR – Annual Growth Rate

CAD – Computer-Aided Design

CS – Complete Street

DNE – Does Not Exist

DOT – Department of Transportation

EBL – Eastbound Left

EBR – Eastbound Right

EBT – Eastbound Through

FDW – Flashing Don’t Walk

FHWA – Federal Highway Administration

GEH – Geoffrey E. Havers

GTA – Greater Toronto Area

GTHA – Greater Toronto and Hamilton Area

HGV – Heavy Goods Vehicle

LCV – Long-Combination Vehicle

LGV – Light Goods Vehicle

LWT – Lane Width Threshold

MAPE – Mean Absolute Percent Error

MaRMD – Major-road-major-direction

MiRMD – Minor-road-major-direction

MSU – Medium Single-Unit Truck

MTO – Ontario Ministry of Transportation

NBL – Northbound Left

NBR – Northbound Right

NBT – Northbound Through

NCHRP – National Cooperative Highway Research Program

OD – Origin-Destination

OTA – Ontario Trucking Association

PDO – Property Damage Only

PET – Post-Encroachment Time

RNSE – Root Normalized Square Error

SBL – Southbound Left

SBR – Southbound Right

SBT – Southbound Through

SI – Système International d'Unités (International System of Units)

SSAM – Surrogate Safety Assessment Model

SBDT – Setback Distance Threshold

TTC – Time-to-Collision

VMT – Vehicle Miles Travelled

WB – Wheelbase

WBL – Westbound Left

WBR – Westbound Right

WBT – Westbound Through

Chapter 1. Introduction

1.1 – Long-Combination Vehicles for Freight Delivery

In the past century, freight transportation has played a significant role in many aspects of daily life. The industry is defined by the movements of commercial goods and services from one place to another via road, rail, air, or barge. In the final segment of a freight delivery, known colloquially as the last-mile, which primarily takes place in urbanized areas, trips are often completed via road transportation using tractor-trailers, single-unit trucks, and fleet vans. For consumers, freight shipments transport desired products and services from producers to retail distributors, where they can access and purchase these products. In addition, with the recent rise of online shopping, this has increased the demand for direct-to-home delivery, where freight vehicles are required to enter residential areas to deliver packages directly to consumers (McDonald et al., 2019). At the same time, freight transportation has many implications for local and global economies, as the strengths of these economies are dictated by the efficiency of freight movement across many links in a supply chain.

One emerging trend in road freight transportation that has been gaining traction in recent years is the use of long-combination vehicles (LCVs) for the transportation of commercial goods. LCVs are defined as freight tractor-trailer combinations with two or more trailers to transport larger volumes than standard freight vehicles (Province of Ontario, 2022a). In Ontario, the LCV Pilot Program was introduced on August 13th, 2009 as part of a partnership between the Province of Ontario and the Ontario Trucking Association (OTA) (Province of Ontario, 2022b; Ontario Trucking Association, 2023). This pilot program was introduced to allow LCVs to operate on public roads and highways for the first time in Ontario while maintaining safety standards (Ontario Trucking Association, 2023). More recently, many municipalities within Ontario have followed suit in preparing their own plans to increase the usage of LCVs in their jurisdictions. For instance, the Region of Peel (2017), one of Ontario's major freight transportation hubs, has outlined its intentions to expand the use of LCVs in its Goods Movement Strategic Plan. Existing freight routes could be expanded for LCV use by assessing the design requirements that are needed along these routes to safely accommodate LCVs (Region of Peel, 2017). Many municipalities are interested in expanding the use of LCVs due to the potential benefits they offer over conventional tractor-trailers by reducing labour costs, fuel costs, and pollution (Province of Ontario, 2022a). Thus, a more widespread application of LCVs in freight transportation can substantially improve these aspects of daily life.

While efficient freight transportation is necessary for a strong economy, one area of public concern in urbanized areas relates to road safety. The large size of freight vehicles leads them to occupy substantial space when completing turning maneuvers on last-mile roads. This can cause the body of the vehicle to collide with other objects when passing into adjacent driving lanes or onto roadside infrastructure. Overall freight-related collisions leading to injury or death in the United States since 2010 have been increasing at a rate faster than the increase in all collisions involving any road user (McDonald et al., 2019). Additionally, freight accounts for 5% of all road collisions but contributes to 9.6% of all collision fatalities (Giuliano et al., 2013). LCVs have produced mixed results in terms of their own safety performance due to the small sample size of collision data from limited operation. For instance, Grislis (2010) found that LCVs produce 25% fewer collisions than conventional trucks, while Woodroofe et al. (2012) finds a 500% decrease in collisions when using LCVs. On the other hand, Regehr et al. (2009) found that LCV collision frequencies and severities are consistent with those of conventional tractor-trailers.

Many governments around the world are adopting initiatives and policies such as Vision Zero (City of Toronto, 2023) to improve road safety. Potential safety concerns arise when freight transportation is introduced into urbanized areas for their last-mile deliveries. These locations have historically been designed primarily for the dimensions of passenger vehicles (Brown et al., 2009), so the immense size and weight of freight vehicles makes it difficult to maneuver in tight areas and forces these vehicles into potentially dangerous interactions with other modes of transportation. This problem is then exacerbated for LCVs, which occupy an even larger space when maneuvering last-mile intersections. This has led to heavy monitoring and restrictions for LCV operation in municipalities across North America (Province of Ontario, 2022b; Province of British Columbia, 2023; Woodrooffe et al., 2012; Intermodal Surface Transportation Efficiency Act, 1991). Furthermore, while many politicians and the general public have been in favour of improving the mobility of underrepresented road users, such as public transit users, cyclists, and pedestrians, the safety interests of these groups may conflict with the use of LCVs. This is because the expansion of active transportation or public transit infrastructure often leads to reductions in infrastructure for motorized transportation, making it difficult for LCVs to navigate these environments without putting the safety of other road users at risk. Therefore, the personal, economic, and safety priorities of individuals, businesses, and municipalities may conflict with one another when LCVs are introduced into last-mile roads.

1.2 – Thesis Goal

Research must be conducted to further understand the safety concerns of LCVs when navigating a last-mile area. As a result, the motivation of this thesis is to investigate the safety risks associated with LCVs as they navigate left turns during the last-mile of their deliveries, with a focus on LCV use in the Greater Toronto Area (GTA).

A considerable lack of knowledge pertaining to last-mile LCV safety exists. Meanwhile, previous studies that have attempted to study LCVs generally follow similar methodologies, study areas, and data sets. One of the most common analysis techniques used by LCV studies is the examination of historical LCV collision data. For instance, Grislis' (2010) study investigates LCV safety using historical fatal collision data to determine fatal collision rates of LCVs, while Woodrooffe et al.'s (2012) study also relied on collision data to determine crash rates and severities of LCVs in Alberta. Both studies are hampered by small sample sizes of collision data limited to LCV-approved regions, which may skew safety results (Grislis, 2010; Woodrooffe et al., 2012). There is also a gap in knowledge related to LCV safety in the last-mile, as these studies focus on freeways where the majority of freight activity takes place.

In addition, one area where LCV research is also lacking is the use of microsimulations to assess LCV road safety. The use of microsimulations is a valid approach to studying safety at specific intersections or corridors within a road network and has been used in the past for many traffic analysis studies. For example, consider the study by Abed & Ewadh (2021), which used microsimulations to assess general traffic safety at signalized intersections in Al-Hilla, Iraq. Multiple intersections were digitized in the microsimulation software Vissim, including information on signalization phases, vehicle behaviours and compositions, and geometry of the intersections. In running these simulations, files representing the trajectories of the vehicles within the networks were produced and transferred into the Federal Highway Administration's (FHWA) Surrogate Safety Assessment Model (SSAM) to assess collision risk within the network. These SSAM analyses then provided Abed & Ewadh (2021) with information regarding both the frequency, severity, and locations of traffic conflicts within the Al-Hilla intersections to determine their collision risk. While this approach to studying safety has been used in past traffic studies, this method has not been attempted for LCVs in particular.

Alternatively, a study on the use of LCVs with a focus on the geometry has been conducted in Saha's (2021) *A Swept Path Analysis of Intersection Designs for Long Combination Vehicles*. The study aimed to assess intersections in the Region of Peel to determine whether their geometric characteristics can safety accommodate the large size of LCVs. A swept path analysis, which plots the area occupied by the body of the LCV as it maneuvers right turns, was completed in AutoTurn to identify any potential conflicts between the body of the LCV and local infrastructure. The thesis then used the results of these swept path analyses to develop a logit model to determine the likelihood that an LCV can safely navigate an intersection based on geometric characteristics (Saha, 2021). However, the scope of the study was limited to right-turning maneuvers only.

Therefore, the purpose of this thesis is to assess the safety of LCVs as they navigate left-turning maneuvers of last-mile intersections in the GTA. This goal fills the knowledge gap remaining from previous papers that excluded last-mile intersections or did not include left-turning maneuvers. This thesis will investigate last-mile LCV safety risks associated with conflicts with geometry of road infrastructure, as well as conflicts with other road users leading to potential collisions.

1.3 – Objectives

The Region of Peel is selected as the study area in investigating the safety of LCVs in Ontario. This area is selected due to its key role as one of Ontario's main freight centers with heavy truck traffic travelling to and from the many distribution centers in the region. Furthermore, this area is selected due to the Region of Peel's plans to increase LCV usage in the future and expand their existing freight network to include LCVs. There are three primary objectives that this thesis aims to achieve:

1. Identify issues for left-turning maneuvers of LCVs and subsequently determine safety trends along Peel Region's Strategic Goods Movement Network.
2. Conduct a swept path analysis for each selected intersection to analyze potential conflicts between LCVs and geometric features.
3. Evaluate the safety performance for each selected intersection based on collisions and near-misses.

This thesis fills current gaps in freight research by focusing on LCV safety and conflicts in the intersections selected in Objective 1. Objectives 2 and 3 then improve upon previous studies by replacing historical collision data with swept path analysis and microsimulation. Objective 2 focuses on a swept path analysis for left-turning LCVs, which is achieved using AutoTurn software. Furthermore, Objective 3 allows for LCV left-turning safety to be assessed with respect to other road users, in addition to road geometry assessed in the swept path analysis, through the use of a Vissim microsimulation and the FHWA's SSAM analysis tool.

1.4 – Scope and Roadmap

The scope of the thesis will not include research into LCVs operating on urban freeways or in a rural setting. The thesis is focused on the operation of LCVs in a last-mile setting using arterial, collector, and local roads in the Region of Peel. Additionally, the results of this thesis will rely on design guidelines specific to Ontario. For instance, the vehicle type assessed in this thesis will be the turnpike double, which is the largest allowable LCV type in Ontario. As a result, one should be wary of applying the results of this study to other geographic contexts with vastly different settings, such as in Asia or Europe. Lastly, this thesis will not study right-turning or through maneuvers of LCVs in intersections. This project prioritizes the safety issues LCVs encounter during left-turning maneuvers as they traverse last-mile intersections.

The remainder of this thesis is composed of seven chapters. *Chapter 2. Literature Review* will review existing research focused on freight safety as it pertains to LCVs. Due to scarce research focused on LCVs in particular, the literature review will first discuss the typical safety issues and vulnerable users related to conventional freight vehicles in last-mile intersections. The current research into LCVs will then be introduced as it relates to the road safety, while acknowledging the current setbacks hindering this research. Next, *Chapter 3. Data* will discuss the study area associated with this research project, while also discussing the types of data required for the planned modelling techniques and the sources of this information. This will then lead into *Chapter 4. Swept Path Methodology* and *Chapter 5. Microsimulation Methodology*, which will outline in detail the techniques used to achieve Objective 2 and Objective 3 within the scope of this thesis. This is then to be followed by *Chapter 6. Swept Path Analysis Results* and *Chapter 7. Microsimulation Results*, which will interpret the results achieved by the methodologies for both the swept path analysis and microsimulation. Lastly, *Chapter 8. Conclusion* will summarize the objectives achieved for this thesis and the conclusions that are reached. This section will also discuss the meaning of this paper within the broader research area of LCVs and will recommend areas of future research that are needed.

Chapter 2. Literature Review

Studies focusing on LCV safety are scarce. Therefore, this literature review first discusses many of the safety concerns posed by traditional freight trucks, as well as the road users most vulnerable in collisions with freight vehicles. These identified safety issues can then be extended to LCVs, as the larger size and weight of LCVs exacerbate the risks caused by traditional trucks. Next, LCVs themselves are discussed, including their use in the context of Ontario and current research that has been conducted into their safety. Lastly, the methods of analysis used to assess freight and LCV safety are explained, including microsimulation modelling and swept path analysis.

2.1 – Transportation Safety and Road Geometry

In transportation planning and engineering, many jurisdictions and governing bodies have traditionally prioritized transportation efficiency as their key goals, ensuring vehicles within their transportation networks can travel to their desired destinations as quickly as possible by reducing traffic congestion and delays. More recently, a paradigm shift has occurred within the industry, placing more emphasis on accessibility and road safety. One key campaign behind this ideological shift in priorities that many jurisdictions have adopted is known as Vision Zero, which aims to fully eliminate casualties of road collisions (Tingvall & Haworth, 1999; Region of Peel, 2018; City of Toronto, 2023). With traffic safety becoming a key design priority in Ontario and many other jurisdictions in Canada, many municipalities are reviewing their road networks to ensure that all motorized and non-motorized users may safely travel without the risk of severe collisions with one another or infrastructure.

When freight vehicles are required to complete last-mile deliveries in dense urban areas, there are many safety complications that can occur due to the larger size of combination tractor-trailers when compared with passenger cars. This has led to some concerning statistics regarding freight transportation in urban areas. According to a 2009 data set of 2987 fatal freight collisions across urban and rural regions in the United States, over a third of fatal crashes, for a total of 1049 fatal crashes involving freight trucks, occurred within urban areas (Giuliano et al., 2013). Furthermore, within these urban areas, despite the fact that the vast majority of freight travel occurs on provincial or state highways, 591 of the 1049 (or 56%) fatal urban crashes involving freight vehicles occurred on urban arterials, collectors, and local roads in 2009 (Giuliano et al., 2013;

Yuan & Wang, 2021). This is also reinforced by McDonald et al.'s (2019) study of urban freight and road safety in the United States. This study compared multiple characteristics of fatal collisions, such as road type or age of person involved in fatality, between any road user and freight vehicles specifically. For a sample of 35484 fatal crashes in 2015 involving any travel mode, 17573 occurred in urban regions. Breaking this down further, 1377 of those urban fatal collisions involved freight vehicles and 16196 involved other road users. Meanwhile, 872 of these fatal freight collisions occurred on urban non-highway roads, making up 63% of urban fatal freight collisions (McDonald et al., 2019). Many of these statistics do not account for exposure, which would consider the freight collision rates per distance travelled by the freight vehicles. By not accounting for exposure, these statistics may mask how dangerous freight vehicles truly are. Therefore, it is crucial to understand the types of safety issues encountered by freight vehicles.

2.1.1 – Freight Conflicts with Urban Infrastructure

Inadequate road infrastructure is one of the primary problems that trucks encounter. Some roads do not have geometric designs to allow large vehicles to safely maneuver. Wygonik et al. (2015) notes that many urban regions were established well before the widespread use of motorized transportation, creating highly compact urban centers with very limited road space available for vehicles. Prior to 1874, the Gunter's Chain served as a primary surveying tool and unit of measure for linear distances (Williamson, 1984), where 1 chain is equal to 66 feet or 20.1 meters. In dense urban regions designed during this era of land surveying, it was common for infrastructure to be designed with respect to chains (Williamson, 1984), including many road allowances in Ontario that remain 66 feet wide (Province of Ontario, 2007). Additionally, urban planners and engineers were quick to utilize the limited road area to ensure that passenger vehicles could efficiently travel as the dominant road user once automobiles became popular with the general public (Brown et al., 2009; Wygonik et al., 2015). As a result of these historical priorities, the geometric design of infrastructure in downtown urban areas conforms primarily to the needs and dimensions of passenger vehicles, making it difficult for freight vehicles to avoid conflicts.

One of the largest issues identified in many studies is the swept path of freight vehicles, which is defined as the envelope traversed by a vehicle during a driving maneuver. The swept path characteristics of large vehicles can be attributed to the physical dimensions and capabilities of these vehicles. The American Association of State Highway and Transportation Officials

(AASHTO) has outlined four primary classes of vehicles, namely passenger vehicles, buses, trucks, and recreational vehicles (AASHTO, 2011). Within the truck vehicle class, AASHTO has defined seven subclasses of trucks based on two characteristics; the wheelbase distance and the number of trailers. The wheelbase is defined as the distance between the frontmost axle on the tractor and the rearmost axle on the trailer. Thus, the subclasses of trucks are denoted by the wheelbase (WB) nomenclature, followed by the wheelbase dimension of the truck, as well as the letters D or T in the case of a double or triple trailer respectively. The seven subclasses of trucks and their dimensions in the international system of units (SI) can be found in *Table 1* below. Therefore, with these typical dimensions, the physical area occupied by the vehicle body during maneuvers is much larger, requiring additional space to complete turns.

Table 1: AASHTO Truck Classifications (AASHTO, 2011)

Truck Type	Nomenclature	Overall Height	Overall Width	Overall Length	Tractor WB	Trailer 1 WB	Trailer 2 WB	Trailer 3 WB
Intermediate Semitrailer	WB-12	4.11m	2.44m	13.87m	3.81m	7.77m	-	-
Interstate Semitrailer	WB-19	4.11m	2.59m	21.03m	5.94m	12.50m	-	-
Interstate Semitrailer	WB-20	4.11m	2.59m	22.40m	5.94m	13.87m	-	-
Double-Bottom Semitrailer	WB-20D	4.11m	2.59m	22.04m	3.35m	7.01m	6.86m	-
Rocky Mountain Double	WB-28D	4.11m	2.59m	29.67m	5.33m	12.19m	6.86m	-
Triple Semitrailer	WB-30T	4.11m	2.59m	31.94m	3.35m	6.86m	6.86m	6.86m
Turnpike Double Semitrailer	WB-33D	4.11m	2.59m	34.75m	3.72m	12.19m	12.19m	-

Many geometric designs in dense urban areas contain narrow rights-of-way for travelling vehicles as well as tighter corner radii at intersections, with these areas primarily designed to serve passenger vehicles (Wygonik et al., 2015; Conway et al., 2016; Moshiri et al., 2022). The City of Toronto (2017) has developed its own Curb Radii Guidelines to assist in urban geometric design. This guide provides many curb radius dimensions based on several characteristics of the intersection under consideration, including the design vehicle and control vehicle, the frequency of each vehicle type, vehicle turning speeds, road classification, intersection angle, and the widths of the origin and destination lanes. In this context, a design vehicle refers to the most frequent vehicle type that will perform right-turn maneuvers at the intersection, where the curb design will ensure these vehicle types can sufficiently make turns at the intersection without occupying additional space beyond their lane. On the other hand, the control vehicle is the largest vehicle type expected to use the right turn in the intersection, although not frequently. Therefore, it is expected that these large vehicles should be able to complete the right turn at the intersection but may occupy additional space in other lanes of traffic in order to do so (City of Toronto, 2017).

Since many dense urban areas were previously designed with passenger vehicles as the design vehicle, little regard was given towards large freight vehicles even as control vehicles. This causes a variety of problems when freight vehicles attempt to maneuver through these areas with their swept path characteristics, particularly when completing turning maneuvers. The City of Portland's (2008) freight movements design manual, as well as studies by Wygonik et al. (2015) and Moshiri et al. (2022), assessed the swept path needs of commercial vehicles when making several turning maneuvers in urban areas. Specifically, Moshiri et al. (2022) focused primarily on identifying issues associated with urban right-turning maneuvers, while the City of Portland (2008) and Wygonik et al. (2015) also investigated left-turning and through movements. These papers found that inadequate curb radii, which are acceptable for passenger vehicles and their dimensions, were too small to accommodate the right-turning maneuvers of freight vehicles, leading to one of two dangerous scenarios. In the first scenario, the freight truck would attempt a right-turning movement along the curved curb, however due to the shape of the vehicle's swept path, this results in the rear end of the tractor-trailer conflicting with the curb design, ultimately climbing the curb and potentially colliding with pedestrians or amenities along the side of the road (City of Portland, 2008; Moshiri et al., 2022).

In a second scenario, truck drivers may attempt to avoid climbing the curb using an oversteering maneuver, keeping the entire trailer within the boundaries of the right-of-way, but this instead results in the front tractor of the vehicle infringing on adjacent driving lanes (City of Portland, 2008; Moshiri et al., 2022). This is also a major safety risk, as infringing into adjacent lanes exposes the commercial vehicle to potential conflicts and collisions with other vehicles and travel modes in these adjacent lanes. Furthermore, these studies identified that in areas with very limited rights-of-way, oversteering could even result in the tractor-trailer passing into the opposite direction traffic lanes, leading to head-on collisions with oncoming traffic, which are often among the most severe types of collisions, or collisions with medians separating the lane directions (City of Portland, 2008; Moshiri et al., 2022).

Both the City of Toronto's *Curb Radii Guideline* (2017) and the Transportation Association of Canada's *Geometric Design Guide* (2017) have indicated that large freight vehicles are permitted to occasionally utilize additional road lanes when completing their turns. In fact, the City of Toronto (2017) has outlined specific design rules regarding scenarios where freight vehicles utilizing adjacent lanes during right turns is assumed. For example, the guide provides information for trucks turning onto arterial roads, collector and local roads with lane markings, and collector and local roads without lane markings. For arterial roads, large trucks like WB-20s are assumed to make turning maneuvers utilizing up to three travel lanes if available, while always maintaining a minimum distance of 300 millimeters from the centreline or centre median of the roadway. For both collector and local roads with or without lane markings, a large truck would be assumed to utilize the entire width of the roadway, including oncoming lanes, while always maintaining an offset distance of 300 millimeters from the edge of the road on the left side of the truck (City of Toronto, 2017). Nevertheless, while these types of freight turning maneuvers are assumed in modern intersection designs, these operations still place occupants of these other traffic lanes at serious risk and require cooperation from other road users, especially in the case of infringing into adjacent lanes. This is unfortunately an issue for many historical urban designs centred on passenger vehicles, where arterials may also require large freight vehicles to occupy oncoming lanes and place road users at risk.

Meanwhile, freight vehicles completing left-turning maneuvers also face similar issues in navigating urban geometric designs. In attempting a left turn where the tractor is in line with the centerline of the road before and after the maneuver, the trailer is likely to conflict with any raised medians alongside either the origin or destination lanes, as well as potentially passing through oncoming lanes of traffic adjacent to the origin and destination lanes. At the same time, if an over-steering maneuver is attempted, the trailer will not conflict with any raised medians, but the tractor will pass into adjacent lanes or over the curb onto roadside amenities in the direction of the destination lane (City of Portland, 2008).

Lastly, another issue related to the limited width of urban rights-of-way is the narrow dimensions of the traffic lanes themselves. Many dense urban areas are typically composed of narrow travel lanes, which are primarily designed for the dimensions of passenger vehicles and may be insufficient for the width of a freight truck. Consequently, there are several conflicts that could occur between commercial vehicles and adjacent lanes, as well as roadside pedestrians and structures while travelling through these areas (Wygonik et al., 2015; Conway et al., 2016; Moshiri et al., 2022).

2.1.2 – Safety Conflicts with Vulnerable Road Users

One important aspect of safety is the driver's field of view. The field of view can vary based on the type of vehicle being operated, as the size and shape of the vehicle can impact a driver's awareness of their surroundings both near and far from their vehicle. For large vehicles such as freight trucks, these vehicles create substantial blind spots in the freight driver's field of view, posing a safety concern in urban areas. In studying the blind spots of freight vehicles and the safety concerns they pose, Niewoehner & Berg (2005) found that blind spots for trucks, which are much more substantial than passenger vehicles, are fundamental causes of many conflicts between trucks and other road users. Due to the size and shape of freight vehicles, they often have considerable blind spots along both sides of the vehicle body, at the rear of the body, and at locations within the direct vicinity around the front of the truck. These are a significant safety concern for smaller travel modes, particularly in densely constructed urban intersections in the last-mile of deliveries where these smaller road users may be forced to travel within these close-proximity blind spots (Niewoehner & Berg, 2005).

Considering the safety concerns caused by freight vehicle blind spots in urban areas, it is also important to understand which road users are most affected. Some studies perform a statistical analysis on collected collision data to identify general trends associated with different types of collisions. A review of this literature found that the road users that typically carry the greatest risk when operating in urban areas alongside freight vehicles are non-motorized transportation modes, including pedestrians and cyclists, as they risk colliding with one of the largest and heaviest road users on urban roads (Niewoehner & Berg, 2005). Pedestrians and cyclists are also largely unprotected, compared to passenger automobiles equipped with safety features, causing them to be more prone to injury and death (Niewoehner & Berg, 2005; Chen et al., 2020). Thus, while collisions between freight vehicles and non-motorized transportation modes may not be the most frequent type of freight collision, they are often amongst the most severe (Niewoehner & Berg, 2005).

In reviewing pedestrian-freight collisions, many studies have focused on both the frequency and severity of collisions. Retting (1993) reviewed fatal crashes that occurred between freight vehicles and pedestrians across metropolitan cities in the United States, while examining the types of crashes that have occurred over a 5-year span and various characteristics of each crash, such as environmental factors, driver characteristics, and pedestrian characteristics. Upon a review of the collision environment, this study found that 51% of the urban fatal collisions occurred at an intersection. Meanwhile, in a breakdown of the intersection maneuvers leading to a collision, the study found that 47% occurred due to through movements, 14% from right turns, 10% from left turns, and 28% due to other or unknown maneuvers (Retting, 1993). Another study by Clifton et al. (2009) completed a comparative analysis of the pedestrian collisions with various types of vehicles in Baltimore, Maryland using an ordered probit model. This study found that while automobiles more commonly collide with pedestrians, larger vehicles such as trucks and buses account for a higher percentage of collisions that result in injury or death. Namely, for injury collisions, 49.4% of automobile-pedestrian collisions lead to injuries, relatively similar compared to 45.6% for freight-pedestrian collisions. However, the percentage of fatal pedestrian collisions involving freight vehicles, at 2.4%, is nearly double the percentage of fatal collisions caused by automobiles, at 1.3%, concluding that freight-pedestrian collisions are much more likely to lead to severe outcomes than pedestrian collisions with other road users (Clifton et al., 2009).

With respect to bicycle-freight collisions, many studies have also examined the relationship between these two modes when encountering one another in urban areas. These analyses investigated the severity, frequency, and riskiest locations associated with freight-cyclist collisions. Morgan et al.’s (2010) paper studied cyclist deaths due to road collisions in London, England, focusing on cyclist collisions and fatalities after colliding with different driving modes to determine which transport modes posed the greatest threat to cyclists. In reviewing a collision data set from 1992-2006, 242 fatal cyclist collisions occurred throughout the study period. While freight vehicles were determined to only account for 4% of all traffic flow in London during the study period, heavy goods vehicles were involved in 102 fatal cyclist collisions, representing 43% of all cyclist fatalities (Morgan et al., 2010). Therefore, freight vehicles clearly pose a significant threat to cyclists considering the disproportionate relationship between the percentage of freight vehicles on the roads of London and the percentage of cyclist deaths due to collisions with commercial vehicles. Furthermore, it was found that 53% of freight-cyclist collisions occurred while the truck was making a left turn at an intersection (Morgan et al., 2010). However, since this study took place in the United Kingdom, a country that obeys left-side traffic rules, this is consistent with a right turn in North America.

Another study in New York observed the effects of freight-cyclist collisions when cyclists have been provided dedicated bike infrastructure (Conway et al., 2016). In reviewing cyclist collision statistics from the New York Police Department from 2012-2015, this study found that while bicycles were only involved in 1.9% of all collisions in the city, bicycle-freight collisions made up 2.9% of all bicycle collisions that occurred over that span. Furthermore, when observing the spatial distribution of collisions involving freight trucks and cyclists, Conway et al. (2016) revealed that most freight-cyclist collisions are concentrated in areas where the freight corridors overlap with New York's bike network, with 64% occurring in these areas. This is not surprising, as an increased exposure of freight-cyclist interactions, combined with a reduced right-of-way for motor vehicles in making their driving maneuvers, has likely increased the risk of collisions.

2.1.3 – Additional Safety Factors

Another key safety consideration relates to weather conditions when driving. Varying weather and climate conditions in Canada are a factor that all drivers face. Throughout the summer months, Canadian drivers are faced with hot weather, which often creates conditions that can lead to heavy rainfall events. Meanwhile, frigid conditions occur throughout the winter, where drivers must prepare to drive during snowstorms and on slippery road surfaces. Fluctuating daily conditions also occur during the transitional spring and autumn months, where weather conditions may transition between rain, snow, and fog throughout the day. One commonality amongst these varying weather conditions is that they can create many environments where a driver's visibility of the road ahead is greatly reduced, such as during heavy rain, fog, snowstorms, and whiteout conditions (Province of Ontario, 2022c). Reduced visibility is a serious weather-related safety risk, as a reduced ability to see the road ahead and the upcoming traffic conditions increases the likelihood of road collisions (Das et al., 2017).

Additionally, the presence of rain, snow, or ice on a road surface during the different weather conditions reduces the friction between the vehicle's tires and the road. This can increase the stopping distance required of vehicles when travelling at high speeds and causes skidding when making sudden stopping or turning maneuvers, leading to potential collisions (Province of Ontario, 2022c). While the safety risks associated with weather may also be greater for freight vehicles due to their exaggerated size, weight, and maneuvering characteristics, this thesis will not review weather as a safety factor, leaving weather conditions out of the project's scope. All experiments will be performed under the assumption of ideal weather conditions.

Another factor that impacts the road safety for all road users is driver behavior and characteristics. Individual drivers can each exhibit different behaviors based on several of their characteristics and abilities, such as level of experience, age, alertness, driving aggressiveness, and vehicle type (Newell, 2002; Transportation Association of Canada, 2017). As a result, this can affect a vehicle's behavior when driving alongside other vehicles on the road. For example, different reaction times will occur when responding to sudden events. Drivers may also choose different following distances when driving behind other vehicles, often based on their reaction times, aggressiveness, and braking capabilities (Gazis et al., 1961; Newell, 2002). Driver behavior can also encompass a drivers' courteousness towards other drivers, such as their ability to work with other drivers to complete lane changes, as opposed to selfishly making sudden lane changes without regard for other drivers (Kesting et al., 2007). Each of these driver behaviors can ultimately impact road safety, as some driver behaviour and human error can increase the risk of collisions. In this thesis, driver behavior will not be investigated for freight vehicles. However, driver behavior will be embedded within the simulations, so the effects of these human behaviours will be taken into account.

2.2 – Long-Combination Vehicles

Freight vehicles have historically been shown to struggle with many of the aforementioned safety issues when traversing urbanized areas. Therefore, the use of LCVs for the transport of goods increases these safety risks associated with freight vehicles in last-mile areas.

The growing interest in using LCVs for the transport of freight goods is attributed to the many potential benefits associated with their use. LCVs are expected to yield many economic benefits for the companies that use them, as these high-capacity vehicles can carry 25-100% more cargo than existing tractor-trailers, ultimately increasing the productivity of each truck shipment completed by a commercial business (Grislis, 2010; Province of Ontario, 2022a; California DOT, 2023). At the same time, businesses will also benefit economically in the form of reduced travel costs, as hourly wages paid to multiple drivers to operate single-trailer trucks can be replaced with fewer drivers with multi-trailer vehicles (Grislis, 2010; Province of Ontario, 2022a; California DOT, 2023). Another key benefit of LCVs is environmental benefits. By converting multiple single-trailer vehicles into fewer multi-trailer trucks, the overall pollution produced per unit of cargo transported, including greenhouse gas emissions, light pollution, and noise pollution, are significantly reduced (Grislis, 2010; California DOT, 2023). Additionally, LCVs can also lead to possible mobility and congestion benefits, as LCVs reducing the number of freight trucks in a road network by combining trailers into a single vehicle ultimately reduces the amount of traffic on the road (Grislis, 2010; California DOT, 2023).

2.2.1 – Types of Long-Combination Vehicles in Canada

Long-combination vehicles (LCVs) are defined as freight tractors that can pull two or more semi-trailers, ultimately exceeding the typical length and weight restrictions placed on standard freight vehicles (Grislis, 2010; Woodrooffe et al., 2012; Province of Ontario, 2022a; California DOT, 2023). In general, there are typically three primary classes of LCVs with different trailer configurations and dimensions that operate in Canada. These are known as the rocky mountain double, the turnpike double, and the triple trailer. A rocky mountain double is an LCV composed of a tractor with two differently sized freight trailers, as shown in *Figure 1*. The first trailer following the tractor is a large semi-trailer between 12.2 and 15.2 meters (40 to 53 feet) in length, while the second semi-trailer is shorter with a length between 7.3 and 8.5 meters (24 to 28 feet) (Woodrooffe et al., 2012; California DOT, 2023). Combined, rocky mountain doubles have a maximum total length of 31 meters (Woodrooffe et al., 2012; California DOT, 2023; Regehr et al., 2009). Furthermore, depending on the different axle configurations or trailer articulation setups, the maximum gross weight of a rocky mountain double can range between 53500 and 63500 kilograms (Woodrooffe et al., 2012).

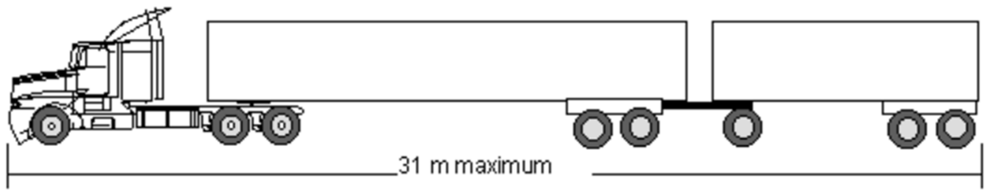

Figure 1: Rocky Mountain Double (Woodrooffe et al, 2012)

Next, a turnpike double is an LCV tractor-trailer combination composed of two large semi-trailers of equal length, as shown in *Figure 2*. The two semi-trailers have lengths between 12.2 meters and 16.2 meters, for a maximum total vehicle length of 38 meters (Woodrooffe et al., 2012; California DOT, 2023; Regehr et al., 2009; Province of Ontario, 2022b). The maximum gross vehicle weight of a turnpike double is up to 63500 kilograms dependent on the length of the trailers, types of dollies used, and axle configurations (Woodrooffe et al., 2012; Province of Ontario, 2022b).

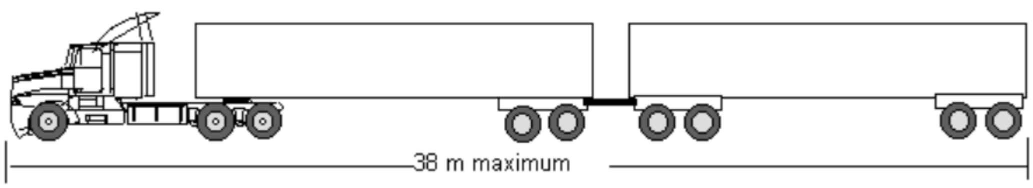

Figure 2: Turnpike Double (Woodrooffe et al, 2012)

Lastly, a triple trailer LCV is a tractor-trailer combination made up of a tractor pulling three semi-trailers, as exhibited in *Figure 3*. The three trailers in a triple trailer configuration typically have approximately equal lengths ranging from 7.3 to 8.5 meters long (Woodrooffe et al., 2012; California DOT, 2023). Overall, these triple trailers typically have a maximum length of 35 meters, along with a maximum gross weight of 63500 kilograms (Woodrooffe et al., 2012).

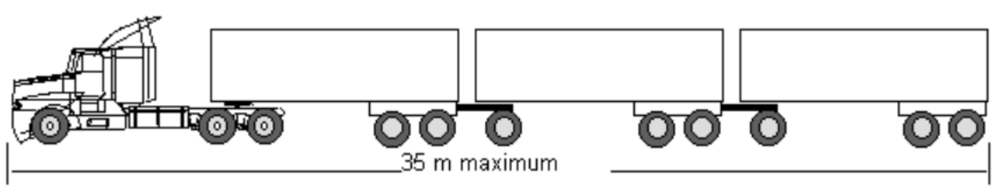

Figure 3: Triple Trailer (Woodrooffe et al., 2012)

The dimensions and gross weight of the three main vehicle types are summarized in *Table 2*. Aside from the three main LCV classifications, there are also several other classes of LCVs which may vary slightly from these three classes due to articulation and dolly configurations or the number of axles on the tractor-trailer, including A-train doubles, B-train doubles, B-train turnpikes, or twin stinger-steer auto carriers (Woodrooffe et al., 2012; Province of Ontario, 2022b). Furthermore, it is important to note that a freight operator may select from these different types of LCVs based on many different factors of the freight activities being performed, including the characteristics of the cargo being transported, the nature of the origin and destination of the trip, the road classes being utilized, and the need to redistribute LCV semi-trailers into multiple single-trailer trucks during the trip (Woodrooffe et al., 2012).

Table 2: Long-Combination Vehicle Classifications in Alberta (Woodrooffe et al., 2012)

LCV Class	Rocky Mountain Double	Turnpike Double	Triple
Number of Trailers	2	2	3
Maximum Total Length (m)	31	38	35
Trailer 1 Length (m)	12.2 – 15.2	12.2 – 16.2	7.3 – 8.5
Trailer 2 Length (m)	7.3 – 8.5	12.2 – 16.2	7.3 – 8.5
Trailer 3 Length (m)	-	-	7.3 – 8.5
Maximum Gross Weight (kg)	53500 - 63500	63500	63500

2.2.2 – Operational Characteristics and Limitations

In addition to the unique physical characteristics of LCVs, jurisdictions have also enacted many restrictions on LCVs that impact their operational characteristics. One key limitation LCVs are often subject to in North America is the sections of the road network they are permitted to traverse. In Canada, it is very common for provincial governments to limit the use of LCVs primarily to urban and rural freeways, which are already equipped with adequate geometric and infrastructure designs to accommodate the size and maneuvering requirements of LCVs. Special permission or licensing must be obtained for LCVs to operate on other less prepared roads, such as arterial or local roads (Province of Ontario, 2022a; Woodrooffe et al., 2012; Province of British Columbia, 2023). This is demonstrated in Ontario, which has outlined their Primary LCV Highway Network across Southern Ontario (Province of Ontario, 2022a). Ontario's LCV network is currently limited to controlled access, multi-lane, divided highways and the ramps that connect the highways (Province of Ontario, 2022b).

Figure 4 displays a map of Ontario's Primary LCV Network within the Greater Toronto and Hamilton Area (GTHA). There are heavy limitations placed on LCVs when travelling outside the LCV network, permitting exceptions for authorized rest and emergency stop locations or truck inspection stations, both of which are typically within 500 meters of the Primary LCV Network (Province of Ontario, 2022b). Additionally, for last-mile destinations of LCV deliveries in an origin-destination (OD) pair, a permit must be acquired from Ontario's Ministry of Transportation (MTO) to allow LCVs to travel on the route. This permit is acquired by a driver once a full engineering assessment of the proposed route has been completed, where an engineering consultant will assess the safety of the highway design and traffic conditions to determine if the route is safe.

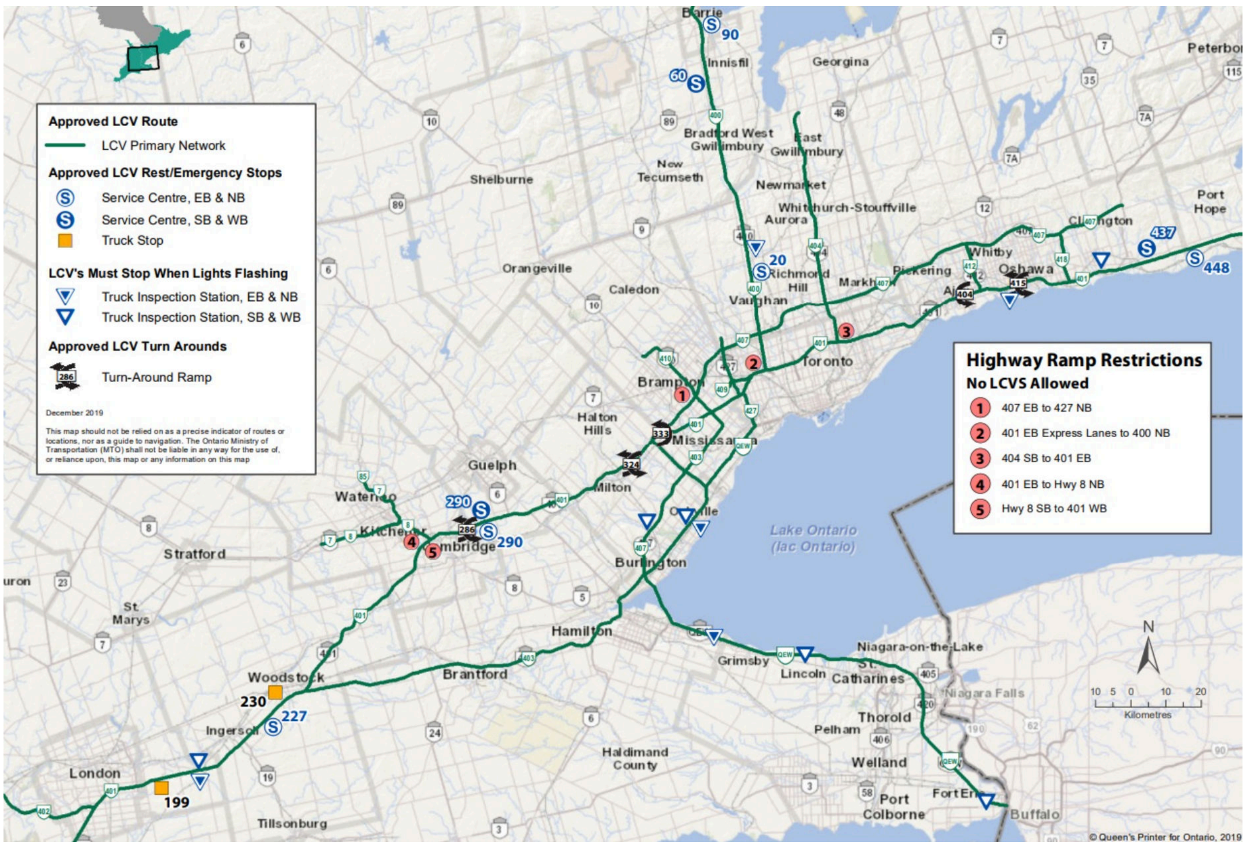

Figure 4: Ontario's LCV Network in the GTHA (Province of Ontario, 2022a)

Across Canada, many other provinces and territories generally follow a similar system. For example, in Alberta, LCVs are limited to specific routes in their provincial highway system, while also requiring drivers to obtain a safety permit for the purpose of travelling routes not included in the LCV network (Woodrooffe et al., 2012). Furthermore, Alberta has outlined specific highway classes that are allowed depending on the type of LCV being driven, where turnpike doubles and triple trailers are permitted only on four-lane highways, while rocky mountain doubles are permitted on four-lane highways and some two-lane highways in the network (Woodrooffe et al., 2012). The province of British Columbia has also taken a similar approach to Alberta, listing specific routes available for use by LCVs, while also listing out the types of LCVs allowed on each route in the province (Province of British Columbia, 2023). Meanwhile, in the United States, LCV regulations vary between states due to federal legislature, where some states allow the use of LCVs on interstate and federal highways, while others limit their use to specific roads (California DOT, 2023). In 1991, the United States Congress passed the *Intermodal Transportation Efficiency Act*, which ultimately prohibited states from increasing the weight and size limitations of articulated vehicles on their roads beyond the vehicle restrictions already set by each state (Intermodal Surface Transportation Efficiency Act, 1991).

Aside from limits on the routes they may use, Ontario has also placed many restrictions on other operational characteristics of these vehicles. For example, there are many limitations on when LCVs may complete their trips in Ontario. In some urban regions, LCVs cannot operate on roads or freeways during peak commuting times on weekdays, such as from 7:00 am to 9:30 am and 3:30 pm to 6:00 pm in regions near Ottawa (Province of Ontario, 2022b). LCV permits also ban the use of LCVs on holidays and often the days or evenings preceding them. Ontario has also mandated that LCVs may not operate during times of inclement weather, where visibility may be reduced, and roads may be covered with snow or debris. Another operational limit on LCVs is the type of cargo allowed to be transported by these vehicles. For instance, livestock may not be transported via LCVs and significant restrictions are placed on the transport of dangerous goods. Lastly, LCVs must not exceed a travel speed of 90 kilometers per hour.

Many requirements have also been established that must be met by the driver and the vehicles themselves. In addition to at least five years of experience operating tractor-trailers vehicles and no driving-related criminal convictions, drivers must also obtain an Ontario LCV Driver Certificate, ensuring they have completed Ontario's LCV Training Program with a licensed instructor (Province of Ontario, 2022b). This ensures that all LCV drivers have obtained the required driving and safety training before operating these vehicles. Meanwhile, there are several technology requirements that vehicles must be equipped with to ensure they are safe for road use, including electronic stability controls, enhanced braking systems, splash and spray controls, on-board speed recording devices for the purposed of enforcement, enhanced vehicle lighting, a minimum engine rating of 425 horsepower, and a steering axle capable of a minimum steering angle of 40 degrees.

2.2.3 – Long-Combination Vehicle Safety

Despite the many potential benefits of using LCVs, the reluctance to widespread use, particularly in urban areas, stems from uncertainties involving their safe operation within last-mile areas. Currently, many jurisdictions limit the use of LCVs primarily to freeways, which are more accommodating for large vehicles (Regehr et al., 2009; Woodrooffe et al., 2012; Province of Ontario, 2022a). Meanwhile travelling through urbanized areas for last-mile trips is extremely limited, as proposed routes through these areas must be examined and approved in order to obtain a permit. When travelling in urban areas, many of the issues with traditional freight tractor trailers are exacerbated due to the larger size of LCVs. LCVs produce an even larger swept path when making their maneuvers, so the risk of collisions with infrastructure, other lanes of traffic, and other road users is amplified when driving in last-mile areas with narrow rights-of-way and limited curb radii (Grislis, 2010). Additionally, LCVs carry a greater risk of rollovers when navigating areas with steep grades, which is a severe safety concern. Limitations in their acceleration and deceleration capabilities when approaching an intersection is also an issue that could lead to collisions with other road users (Grislis, 2010). Therefore, while the use of LCVs is promising due to the possible benefits it yields, concerns regarding their safe operation remain.

Due to the limitations in LCV use in urban areas, there are many differing opinions on the safety of LCVs. One study aimed to assess LCVs in terms of road safety by analyzing accident data from the United States to estimate LCV fatal collision rates (Grislis, 2010). This analysis found that for urban non-freeway roads, LCVs produced 2.13 fatal collisions per 100 million vehicle miles travelled (VMT), which is lower than the 2.84 fatal collisions per 100 million VMT for traditional tractor-trailers. Decreases in fatal crash rates can also be observed for urban freeways, but crash rate increases occur for rural freeways and rural non-freeways. While this does represent an improvement for urban arterials, there are reasons to be cautious of these results. Grislis (2010) notes that collision data involving LCVs is very limited since they are not widely implemented.

Regehr et al.'s (2009) study was faced with similar sample-size issues. In this study, they attempted to compare the safety performance of LCVs to the safety of other more standard articulated freight trucks. In reviewing LCV collision data in Alberta from 1999 to 2005, collision rates per 100 million VMT were determined for each truck type when travelling along Alberta's LCV network. Comparing a summary of all types of LCVs to tractor-trailers, LCVs had a collision rate of 25 collisions per 100 million VMT, compared to 42 collisions per 100 million VMT for standard tractor-trailers. In reviewing severity of collisions involving trucks, the number of property damage only (PDO), injury, and fatal collisions were relatively consistent between LCVs and other tractor-trailers. Again, this study mentions a lack of available LCV information as a fundamental issue with LCV studies, citing the fact that LCVs are only permitted on specific highways included in Alberta's LCV network for this specific study. Additionally, this study provides no information in the context of urban last-mile deliveries, as they are not permitted on urban roads and are excluded from this analysis.

Another study by Woodrooffe et al. (2012) also examined crash rates for LCVs in Alberta by investigating crash rates per 100 million vehicle kilometers travelled. This study reviewed these collision rates for fatal, injury and PDO crashes, finding that LCVs produced about five times fewer collisions across each severity category than standard tractor-trailers, concluding that LCVs are five times safer. Interestingly, Woodrooffe et al. notes that data collected for collision rates may also be biased due to the conditions needed to operate an LCV. Since LCVs may only be used with a permit after routes have been carefully examined beforehand, the routes have already been deemed relatively safe, so collision rates may be correlated with this safety permit that has been approved. Therefore, this data would not be reflective of the safety conditions of ordinary routes that may be underprepared for the characteristics of an LCV.

Lastly, Saha (2021) utilized a swept path analysis to evaluate the safety of right-turning maneuvers in arterial intersections in the Region of Peel. This involved recreating the geometry of multiple intersections of interest in an engineering drawing software and then using a swept path analysis tool, called AutoTurn, to plot the area the body of the LCV occupies when making a right-turning maneuver. In doing this, the paper was able to assess the different characteristics of each intersection in the case study, such as lane widths and curb radii, to determine whether they are adequate for the expected dimensions of an LCV. This was completed for multiple right-turning scenarios, finding that LCVs often benefit from having access to adjacent traffic lanes when the right-most lane is inadequate in width. Furthermore, this study used the results of the AutoTurn analyses to develop a logit model, which could be used to calculate the probability that an LCV can maneuver an intersection based on its characteristics. NLOGIT software was applied to determine the statistical significance of both the lane widths and curb radii on the ability of the LCV to complete a right turn at each arterial intersection. This study was successful in assessing the safety of LCVs as they relate to intersection geometry during right turns, presenting the opportunity for future studies to assess other types of LCV maneuvers.

2.3 – Analysis Techniques

In analyzing the safety of freight vehicles and LCVs when operating in urban areas, there are three primary techniques used. These techniques include reviewing historical collision data trends, swept path analysis, and microsimulation modeling.

2.3.1 – Collision Trends

The most common method through which the safety of freight vehicles in urban areas is evaluated is by reviewing reports of collision data from numerous databases. Multiple papers discussed in the previous sections of this literature review involved analyzing historical data collected over a study period for collisions involving freight vehicles, including LCVs, with other users (Retting, 1993; Clifton et al., 2009; Regehr et al., 2009; Grislis, 2010; Morgan et al., 2010; Woodrooffe et al., 2012; Conway et al., 2016). However, there are several issues with this approach. Freight collisions are very infrequent compared to other modes, as they make up only a small percentage of all traffic volume, and the majority of freight and LCV travel takes place on urban and rural freeways (Giuliano et al., 2013). As a result of this low exposure of freight vehicles in urban areas, the sample size of recorded collisions can be very limited, preventing a comprehensive safety analysis. Furthermore, data is much more limited for specific intersections or corridors, as well as extremely rare collision types, such as freight-pedestrian or freight-bicycle collisions (Conway et al., 2016).

A second issue with collision data is the nature of the data collection itself, as freight collision data collection often relies on police reports of collisions (Hauer, 1997). As a result, information can be biased or incomplete since police reports are primarily created for the purposes of civil law rather than safety reporting. This can also lead to underreporting of PDO collisions, as these types of collisions may not always result in a police report (Hauer, 1997). Lastly, collision data involving new technology or concepts may be insufficient for a safety analysis, such as collisions involving LCVs or in areas designed as complete streets (CS) (Regehr et al., 2009; Grislis, 2010). Thus, safety analyses of freight in urban areas moving forward should steer away from relying solely on collision data, in favour of other methods that can evaluate collision potential of specific regions, intersections, or corridors.

2.3.2 – Vehicle Dynamics

An alternative method used to assess LCV safety is the analysis of LCV vehicle mechanics, including characteristics related to the vehicle's steering, acceleration, and stability. Since some LCVs weigh much more than typical tractor-trailers, these vehicles are limited in their ability to accelerate and decelerate when driving (Grislis, 2010). Thus, these vehicles must be equipped with adequate acceleration capabilities needed for urban and rural travel, as inadequate acceleration capabilities could lead to collisions with other road users. Additionally, steering and stability characteristics must be taken into consideration, as the stability of trailers during high- or low-speed turning maneuvers at various possible steering angles can pose the risk of rollover collisions (Grislis, 2010). At high speeds over 80 kilometers per hour, a sharp steering maneuver can result in rearward amplification, causing the rear trailers to topple or skid due to the weight and size of the trailers, leading to dangerous collisions (Grislis, 2010; Zhu et al., 2021). As a result, monitoring the stability characteristics of these LCV trailers can be an effective measure of safety for LCVs (Zhu et al., 2021).

2.3.3 – Swept Path Maneuvering

Another alternative method of analysis for the safety of freight vehicles in last-mile areas would be a swept path analysis of intersections or corridors in a road network using a computer-aided design (CAD) software. A swept path envelope is the area occupied by any part of a vehicle's body throughout an attempted maneuver. Based on the vehicle type and maneuver attempted, the swept path envelope can vary greatly from the size and shape of the vehicle when stationary. AutoTurn is a modelling software used by transportation engineers to replicate the maneuvers of vehicles within an existing or proposed geometric road design (Transoft Solutions, 2023). In this software, a two- or three-dimensional model of a road design can be reproduced in AutoCAD to represent the many geometric design features of a proposed road, such as curb radii, lane widths, and median dimensions. AutoTurn can then be used in conjunction with this AutoCAD drawing to simulate the movements of many types of vehicles within the drawn environment, including the swept path of vehicles during turning simulations, vertical and lateral clearance of vehicle bodies, and sight lines of drivers in the environment. Additionally, potential conflicts with infrastructure or the swept path of other road users can also be identified when simulating vehicle maneuvers in the drawn environment (Transoft Solutions, 2023).

As applicable to freight vehicles in last-mile areas, AutoTurn can be used to assess the conditions freight vehicles encounter in dense urbanized areas. Multiple different types of freight vehicles are included within AutoTurn, including different types of tractor-trailers, single-unit freight vehicles, and LCVs, each with dimensions resembling government regulations from many regions of the world (Transoft Solutions, 2023). Therefore, these freight vehicles can be assessed in a drawn environment to determine where potential conflicts could occur along the vehicle's swept path at different possible operating speeds. This method does not rely on existing collision data, so the many issues associated with historical collision data is avoided. However, the use of swept path analysis will not directly identify the severity of collisions.

2.3.4 – Microsimulation Modelling

Another analysis method that is available to assess freight vehicles in last-mile settings is through the development of a microsimulation model. A microsimulation model recreates the traffic environment in a digital format to mimic traffic behavior. One effective microsimulation software used in many traffic studies is PTV Vissim (PTV Group, 2023a). Within PTV Vissim, it is possible to create a detailed model of the road network in question, including many geometric aspects of the roadway, such as lane widths and lane tapering characteristics, and operational characteristics of the roadways, such as speed limits, traffic light and pedestrian signalization patterns, and dedicated lanes. In addition, many characteristics of the users of the transportation network are also incorporated into a Vissim model based on existing or projected data, including traffic volumes on each link in the network, proportion of traffic volume travelling in each direction, percentage of traffic volume composed of passenger vehicles, transit vehicles, cyclists and freight vehicles, and many behavioral characteristics of each travel mode when interacting within the network. In performing microsimulations, researchers can generate many diagnostic measures of the roadway that are helpful in evaluating transportation networks and proposed changes, including measures of travel time, vehicle delays, queue lengths, levels of service, and vehicle trajectories (PTV Group, 2023a).

For the purposes of safety analyses, Vissim alone does not assess collisions between road users. However, a Vissim model can be used in conjunction with other software, such as the Federal Highway Administration's (FHWA) *Surrogate Safety Assessment Model* (SSAM), to determine safety concerns in a road network (Gettman & Head, 2003; FHWA, 2022). SSAM software is a tool developed by the FHWA to evaluate microsimulation models for safety metrics in the modelled environment (Gettman & Head, 2003; FHWA, 2022). In evaluating road networks for safety, the number of collisions occurring in the network is the most prominent metric. A higher collision frequency indicates a network is less safe. Another safety metric attempts to quantify safety using near-miss collisions, where the road users risk colliding if they do not take evasive actions (Gettman & Head, 2003). Transportation engineers often use potential conflicts as a surrogate measure of safety, as conflicts occur much more frequently than actual collisions and conflict metrics can provide information into the severity of the collisions. While there are many different surrogate measures of safety conflicts, two of the most prominent types are time-to-collision (TTC) and post-encroachment time (PET). TTC is the expected time that two vehicles will collide if neither vehicle adjusts its current path or speed (Gettman & Head, 2003). Thus, a lower TTC indicates a higher likelihood of a collision. On the other hand, PET is the time between one vehicle leaving the trajectory of a second vehicle and the second vehicle reaching the course of the first vehicle (Gettman & Head, 2003). A lower PET indicates that a collision is more immediate.

With respect to conflicts, SSAM uses the surrogate measures TTC and PET to measure the level of safety in a modelled road network (Gettman & Head, 2003; FHWA, 2022). Using the vehicle trajectory files produced by the Vissim model, SSAM can provide an extensive summary of potential safety issues within the modelled study area, including both the frequency and severity of conflicts using measures of TTC and PET. In terms of conflict frequency, a list of all the potential collisions is provided by SSAM, detailing characteristics of each collision, including collision angle, vehicles involved, vehicle speeds at the moment of the collision or conflict, and the location of the conflicts within a map of the traffic network. Meanwhile, for the severity of collisions, the TTC and PET, among other surrogate safety measures, is calculated for each conflict and classified into ranges according to severity, where a lower TTC or PET indicates a greater conflict severity (Gettman & Head, 2003; FHWA, 2022). For both the TTC and PET, SSAM calculates these values for each conflict as the difference in time between when the two vehicles reach the point of conflict on their trajectories. This is demonstrated in the conflict point diagram in *Figure 5*, where Gettman & Head (2003) define TTC as the difference between t3 and t4, while PET is defined as the difference between t3 and t5 for a crossing conflict.

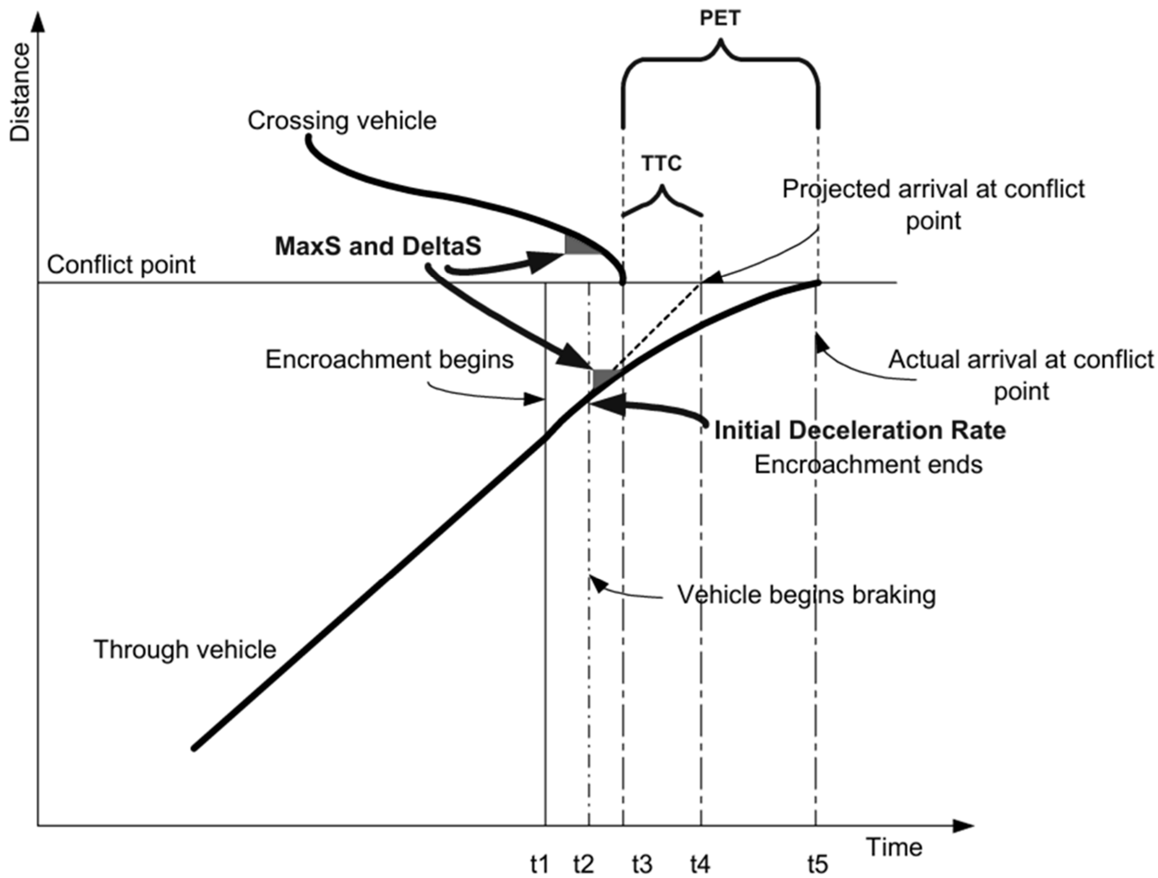

Figure 5: Conflict Point Diagram for TTC and PET (Gettman & Head, 2003)

The combination of Vissim and SSAM can be very useful as a microsimulation method of assessing freight safety. In replicating the traffic network in Vissim, the appropriate proportions of freight vehicles and other modes of transport, as well as their behavior in the simulation, can be used to create a vehicle trajectory file. This trajectory file can then be input into SSAM to determine the frequency and severity of different conflicts, focusing particularly on conflicts involving freight vehicles in the road network. This can help to draw conclusions about the typical locations where freight conflicts are more likely to occur, as well as the severity of freight conflicts with passenger vehicles, cyclists, or pedestrians. In fact, Abed & Ewadh (2021) used Vissim in their road safety study. In this project, they recreated Al-Hilla's signalized intersections within Vissim using traffic and signalization data, followed by using SSAM to successfully determine conflict severity and frequency at different locations within the intersections (Abed & Ewadh, 2021). Therefore, microsimulation is a viable method to evaluate freight safety in last-mile areas without the need for existing collision data. However, it should be noted that Vissim and SSAM do not provide much information involving conflicts between vehicles and the local infrastructure, as it only focuses on conflicts between vehicles.

In applying Vissim and SSAM to assess the safety of a given intersection or road network, it is also important to fully understand the validity of the results produced by these software programs. That is, how reliable are these software simulations in replicating the real-world number of traffic conflicts and collisions occurring at these intersections? Since the introduction of SSAM by the FHWA, several researchers have taken measures to assess the validity of the safety results produced by the software compared to real-world conflicts. Amongst these papers, results were moderately favourable for using SSAM to predict traffic conflicts but ultimately varied based on the level of Vissim and SSAM calibration or conflict type.

One paper by Huang et al. (2013) developed Vissim models for a set of signalized intersections based on collected traffic data and road geometry. The Vissim results were fed into SSAM to identify conflicts simulated in the model. Concurrently, they also conducted a field study to record the number of real-world conflicts occurring at each intersection. Comparing the simulated and real-world conflicts using the mean absolute percent error (MAPE), it was found that the simulation underestimated the total number of conflicts and was inconsistent between conflict types, with MAPE values for rear end, crossing, and lane changing conflicts of 24%, 70% and 85% respectively. By further calibrating the selected TTC and PET threshold values in SSAM, the total conflict MAPE was reduced to 24%, the rear end conflict MAPE to 16%, and the crossing MAPE to 23%. Meanwhile, the simulation was less consistent in predicting lane changing collisions, as the MAPE was still at 79% after calibration. Huang et al. (2013) proposed that this inconsistency in the simulation is because the simulation is unable to capture the unpredictable nature of human driving behavior, especially in cases where unexpected or illegal maneuvers lead to conflicts.

At the same time, Muley et al. (2018) took a similar approach to compare simulated and observed conflicts in Doha by developing a Vissim model alongside a field study observing traffic conflicts. In adjusting the TTC and PET threshold ranges, the differences between vehicle-vehicle conflicts and vehicle-pedestrian conflicts in the simulation and field study were assessed, resulting in more accurate results. It was found that for vehicle-vehicle conflicts, the location and number of simulated and observed conflicts were relatively accurate, with the simulation overestimating the number of vehicle-vehicle conflicts. Meanwhile, for pedestrian-vehicle conflicts, SSAM tended to underestimate the number of conflicts, likely due to limitations in Vissim's pedestrian modelling capabilities.

Lastly, Vasconcelos et al. (2014) also conducted a viability assessment of SSAM by comparing SSAM's simulated conflict output with observed field conflicts as well as against conventional accident prediction models developed in the United States, the United Kingdom, and Portugal. The conflict results of the simulation provided consistent estimates but varied between selected accident prediction models. Across the four intersection types assessed, the general patterns of simulated conflicts matched the observed conflict trends, but the simulation underestimated the number of conflicts in each case due to the selected TTC threshold in SSAM. Vasconselos et al. (2014) also concluded that in using SSAM to simulate conflicts, the validity of the conflict estimate is highly sensitive to the accuracy of the created microsimulation model.

2.4 – Summary

In summary, with an expanded political focus on improving road safety due to initiatives such as Vision Zero, the safety of freight vehicles when operating within dense last-mile areas must be reviewed. One of the most important issues associated with freight safety is related to potential collisions with arterial infrastructure due to road geometry, where street dimensions do not adequately accommodate for the exaggerated swept paths of the many different types of LCVs. The other key safety issue with freight vehicles is their blind spots, where their limited field of view can lead to increased safety concerns in these obstructed areas. Furthermore, it is clear that both pedestrians and cyclists are the road users most vulnerable to severe collisions with large commercial vehicles since they are the most unprotected users compared to motorized vehicles and can be easily hidden in a freight truck's blind spot.

Each of the four methods of analysis for freight safety have advantages and disadvantages. Past researchers have struggled with sample-size issues due to a lack of widespread LCV utilization, making reviewing historical collision data potentially inaccurate and biased in a last-mile setting. Additionally, assessing LCV designs for vehicle dynamics is another technique for evaluating LCV safety. However, vehicle mechanics exist outside the scope of this thesis. Therefore, a swept path analysis is conducted in this thesis to assess the left-turning maneuvers of LCVs within a selected study area of intersections to complement previous research on right-turning maneuvers by Saha (2021). A microsimulation model is also created in Vissim and SSAM to assess conflict frequencies and severities when LCVs are travelling within intersections alongside other road users. In utilizing swept path models and microsimulations, this thesis overcomes the issues associated with historical collision data. While there are some inconsistencies associated with conflict prediction using microsimulation and SSAM, results of previous studies testing its validity are in favour of using SSAM to predict conflicts if extensive calibration of the microsimulation model is performed.

Chapter 3. Data

The following sections will review the data used throughout this thesis to assess the safety of LCV left-turning maneuvers. This includes a discussion of the selected study area for analysis, as well as the criteria used to select a specific route passing through multiple key intersections. Lastly, based on the previous papers utilizing swept-path analyses in AutoTurn and microsimulation modelling in Vissim, there are several types of data required in order for these methods to be successful. Therefore, for each analysis method, the types of data required, their sources, and their use in each method of analysis will be examined.

3.1 – Study Area

The Region of Peel is a municipality located in the Greater Toronto Area (GTA) that encompasses three main municipalities: the Town of Caledon, the City of Brampton, and the City of Mississauga. Peel Region is regarded as one of Canada's freight capitals, with an estimated $1.8 billion in commodities travelling to, from and through the region daily (Region of Peel, 2017). With much of the region's economy dedicated to warehousing, distribution centers, manufacturing, agriculture, and wholesale trade, it is vital that the local transportation infrastructure is prepared to accommodate high levels of freight activity. Thus, Peel Region's freight-centric economy provides ample opportunity to assess the potential safe usage of LCVs in the area for last-mile activities.

A study area is selected to encompass Peel Region's high levels of freight activity. The selected area located in Mississauga, highlighted in purple in *Figure 6*, will serve as the primary study area of this project. The location southwest of Pearson International Airport is bound by Dixie Road in the east, Courtneypark Drive East in the north, Highway 401 in the south, and Highway 410 in the west. This area will likely have more LCV activity in the future, as both Highway 401 and Highway 410 are part of Ontario's primary LCV Network (Province of Ontario, 2022a).

The study area in *Figure 6* contains a high-density industrial zone oriented towards freight activities, with establishments such as distribution centers, warehouses, manufacturing, and freight logistics companies located in this area. These businesses rely heavily on the commercial use of freight trucks and would likely consider adopting LCVs if presented with the opportunity. With respect to the existing road infrastructure, this zone is composed of multiple major and minor arterial roads, as well as many collector roads that provide access directly to local businesses.

These roads are connected with a mix of both three-legged and four-legged intersections, with both signalized and unsignalized controls throughout the area. The study area also includes two highway interchanges: between Highway 410 and Courtneypark Drive East and between Dixie Road and Highway 401.

Figure 6: Study Area Location

To assess LCV safety in the study area, a route is selected such that an LCV encounters multiple different left turning scenarios. This includes the selection of a route with a mix of signalized and unsignalized intersections, a sufficient number of left turns at intersections, and a range of different road types. Additionally, while there are numerous possible routes that can be completed within the study area, a realistic route that can be followed by a freight vehicle should be selected. Therefore, the route should not include an excessive number of turns or overlapping paths. Overall, five primary paths are considered for this project as realistic route options. Each path in *Figure 7* starts at the off-ramp from Highway 401 Eastbound onto Dixie Road, and ends at Courtneypark Drive East turning onto the Highway 410 Northbound On-Ramp. In reviewing these five alternative routes, characteristics regarding the number and type of left turns at intersections is assessed, as well at the classification of roads that are included in the route intersections, as the LCV travels from the start point to the end point. A summary of these attributes is exhibited in *Table 3*.

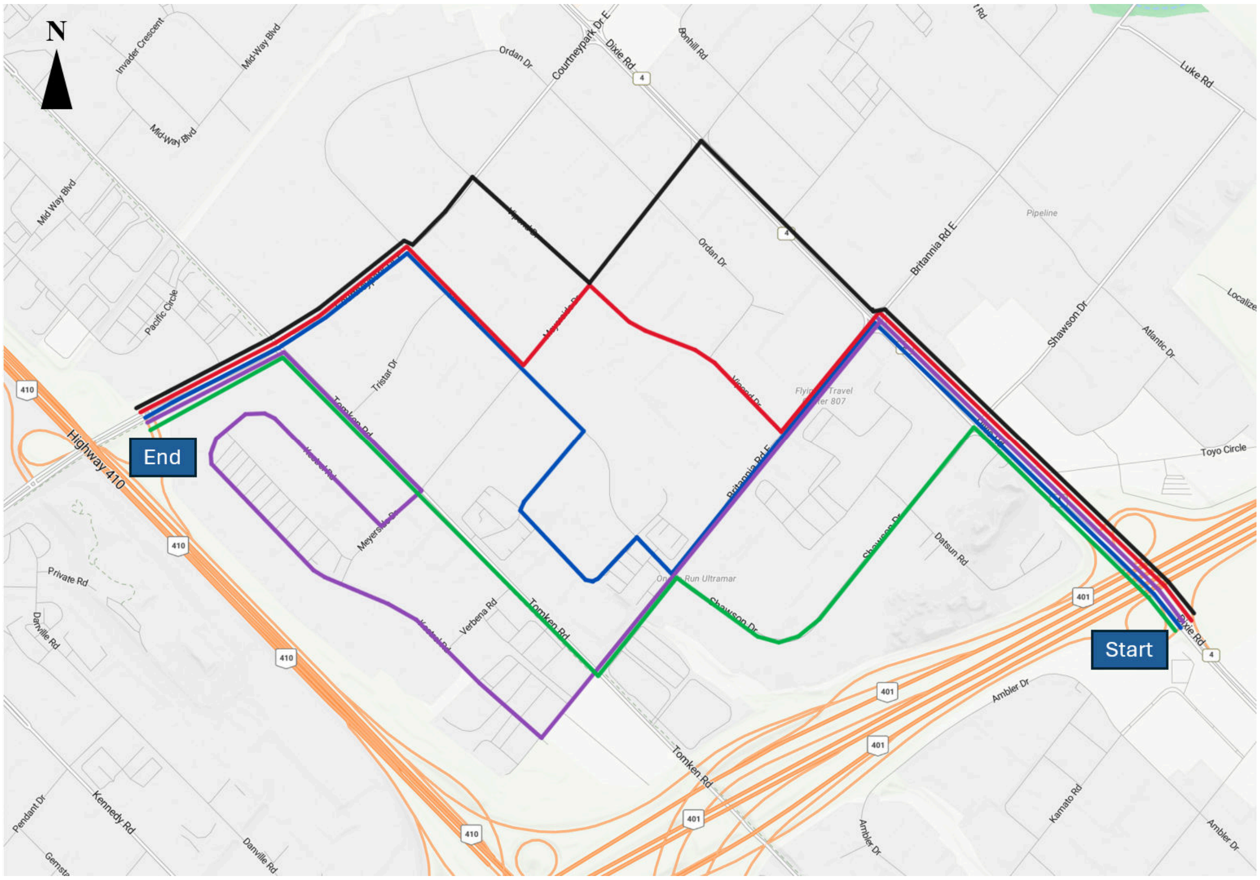

Figure 7: Candidate Routes in Study Area

Table 3: Characteristics of Candidate Routes

Route	Total # of Intersections	# of Left Turns	# of Left Turns at Signalized Intersections	# of Left Turns at Unsignalized Intersections	Frequency of Road Classes Encountered at Left-Turning Intersections		
					Collector Roads	Minor Arterial Roads	Major Arterial Roads
Green	11	5	5	0	1	2	2
Blue	13	6	4	2	2	1	2
Red	12	5	5	0	3	1	2
Black	12	4	4	0	2	0	2
Purple	14	6	5	1	2	2	2

In *Table 3*, it should be noted that the Frequency of Road Classes Encountered at Left-Turning Intersections corresponds to the types of roads encountered on both the departing and destination legs of each left turn. Here, a road class counted at one intersection as either the departing or destination road is not counted again if the exact same road is encountered at another intersection. Referring to the blue route as an example, the route includes a segment where the vehicle turns left from Westport Cresent onto Shawson Drive and then turns left from Shawson Drive onto Courtneypark Drive. In the first left turn, both the departing road and destination road are collector roads, bringing the number of collector road left turns to two. Then with the next left turn onto Courtneypark Drive, the tally of major arterial roads increases since the receiving road Courtneypark Drive is classified as a major arterial road. Meanwhile, the count of collector roads remains at two, as Shawson Drive as the departing road does not increase the count of the collector roads since it was already accounted for in a previous left-turning intersection.

Based on the review of the candidate route attributes, the blue route is selected as the path for the LCV in this thesis. The selected route starts by turning left from the Highway 401 Eastbound Off-Ramp onto Dixie Road and travels northbound. The LCV will then turn left onto another arterial road, Britannia Road, using a signalized intersection. The route then continues to turn right onto Shawson Drive, while also turning left into Westport Crescent, a minor collector road. The path then turns left to return to Shawson Drive and travels northbound until a left turn onto the major arterial road Courtneypark Drive East. The LCV will continue west along this road until a final left turn onto the Highway 410 Northbound On-Ramp. The selected path features a variety of both signalized and unsignalized left turns, corresponding to the six left turns counted in *Table 3*. The blue path also ensures multiple types of road classes are included on the departing and destination legs of left-turning intersections to assess the safety effects of road type on LCV left turns. The selected path includes several intersections with crosswalks for pedestrians. Additionally, there is no dedicated cycling infrastructure anywhere in the study area, which is typical of industrial areas with low bicycle traffic volumes. In addition to these six left-turning intersections, there are also seven other intersections that the LCV will encounter, including intersections between Dixie Road and the Highway 401 Westbound Off-Ramp, Dixie Road and Shawson Drive, Britannia Road and Vipond Drive, Britannia Road and Shawson Drive, Shawson Drive and Meyerside Drive, Shawson Drive and Tristar Drive, and Courtneypark Drive East and Tomken Road. These intersections, which contain either through movements or right turns, are not included in the swept path analysis but are still relevant for the microsimulation model. A detailed image of the blue path can be observed in *Figure 8*, with left-turn symbols to denote where the left turns are taking place and traffic light symbols to identify intersections that are signalized. Furthermore, intersection numbers have been assigned to the thirteen intersections in the chronological order they appear in the route, as listed in *Table 4*.

Table 4: Intersection Numbers

Intersection Number	Intersection Name
1	Dixie Rd and Highway 401 EB*
2	Dixie Rd and Highway 401 WB
3	Dixie Rd and Shawson Dr
4	Dixie Rd and Britannia Rd*
5	Vipond Dr and Britannia Rd
6	Britannia Rd and Shawson Dr
7	Shawson Dr and Westport Cr South*
8	Shawson Dr and Westport Cr North*
9	Shawson Dr and Meyerside Dr
10	Shawson Dr and Tristar Dr
11	Shawson Dr and Courtneypark Dr*
12	Courtneypark Dr and Tomken Rd
13	Courtneypark Dr and Highway 410 NB*

*Indicates intersection is assessed in both AutoTurn and Vissim.

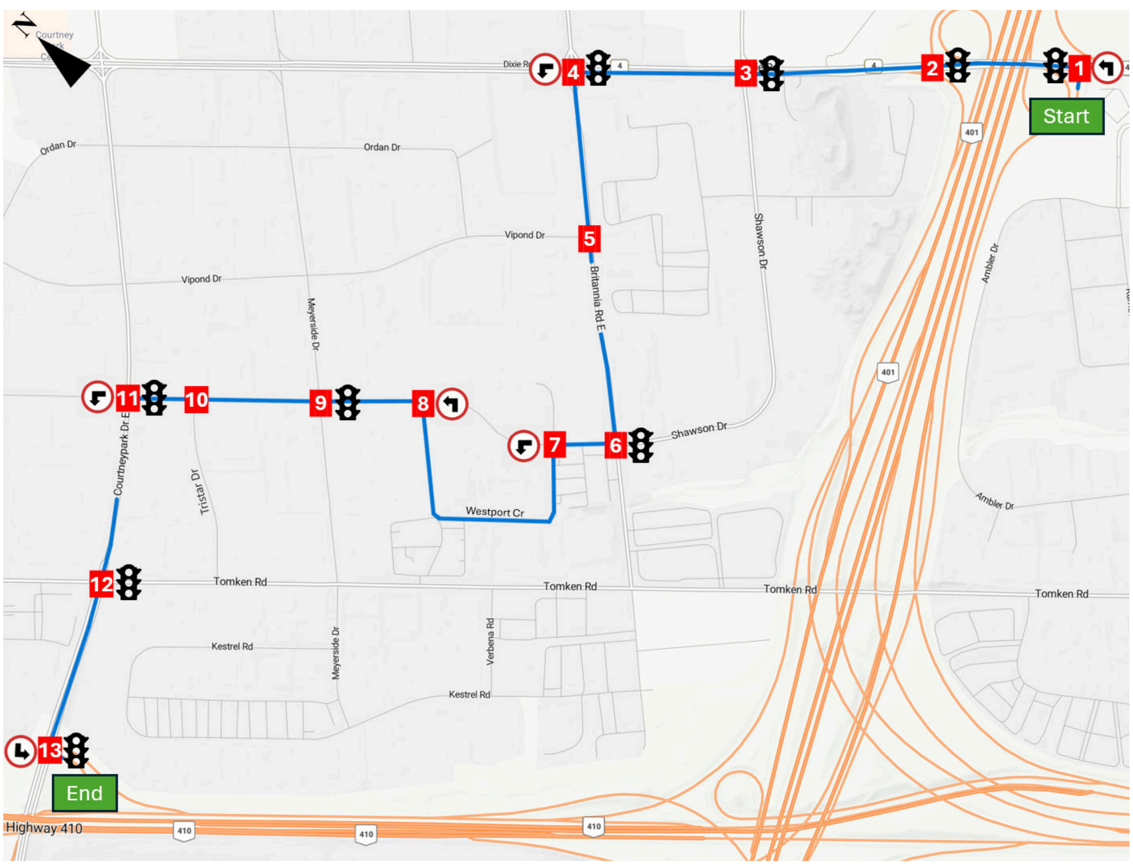

Figure 8: Selected Study Route

3.2 – Route Intersections

The selected route displayed in *Figure 8* features six intersections in which a left turn will take place and seven non-left-turning intersections as the projected LCV travels from the off-ramp at Highway 401 and Dixie Road to the on-ramp at Courtneypark Drive East and Highway 410. There are several sources of data that are relied upon to obtain necessary information on the physical geometry, signalization, and traffic volumes of these thirteen intersections.

To assess LCV conflicts with infrastructure in these intersections, it is important to have an accurate understanding of the physical environment the LCVs will be navigating alongside other vehicles. Satellite imagery is used as a primary resource to achieve an accurate understanding of each intersection's physical geometry. In the swept path analysis, AutoCAD software offers a built-in geolocation setting, which allows for drawings to be superimposed on existing satellite imagery from Microsoft's Bing Maps. These satellite images of the area are introduced with a uniform scale with the AutoCAD drawing. Vissim also relies on built-in satellite imagery from Bing Maps. Therefore, road geometry, including lane widths, curb radii, and median dimensions, is drawn to scale in each respective software using the existing satellite imagery. Due to the image resolution of Bing Maps, Google Earth Pro is used to double-check and verify geometry measurements provided by Bing Maps.

To obtain information regarding the timing of the traffic lights and measured volumes of traffic at each intersection, the local government's transportation departments were contacted. The study area is composed of roads controlled by two jurisdictional authorities. Dixie Road is classified as a regional road under the jurisdiction of Peel Region. Meanwhile, the remaining roads are classified as local roads under the jurisdiction of the City of Mississauga. Therefore, signalization and volume data for intersections along Dixie Road are obtained from the Region of Peel, while the remaining intersections are requested from the City of Mississauga. This data is used during the microsimulation phase of this project.

Signalization data is obtained for all intersections along the proposed route. Extensive information is obtained on the signal timings, including the green time, amber time, red time, and pedestrian crosswalk timings. The data provided also indicates the presence of vehicle detectors and the type of actuation in place at each intersection.

The traffic data provided for all intersections along the route is obtained through traffic studies previously carried out by each jurisdiction. This data includes the volume of traffic passing through the intersection each day, the volume count travelling in each direction, the proportion of cars, buses, and heavy vehicles for each maneuver, the number of pedestrians crossing in each direction, and the number of bikes using either the road or the crosswalks. As an example, *Table 24* in *Appendix B.1.1* showcases a summarized version of the raw data obtained for traffic volumes and their respective turning proportions at each intersection. Using this data, an accurate microsimulation of the intersections that reflects the existing conditions can be devised.

Amongst these intersections, it is important to highlight the physical features of the six left-turning intersections, as these intersections are included in detail for both the swept path analysis and microsimulation modelling. Thus, the following sections will review in detail each left-turning intersection encountered in chronological order as the LCV travels along the proposed route.

3.2.1 – Highway 401 Off-Ramp and Dixie Road

The first left turn takes place at the intersection between the Highway 401 Eastbound Off-Ramp and Dixie Road, where the LCV is turning northbound onto Dixie Road from the highway off-ramp. This intersection is shown in *Figure 9*. This is a three-legged intersection, as the highway off-ramp does not continue on the other side of Dixie Road, and therefore the vehicles coming off the highway can only turn left or right. The intersection is also signalized, meaning that traffic flow on Dixie Road is stalled in both directions while vehicles exit the off-ramp. In attempting a left-turning maneuver, the off-ramp provides vehicles with two left-turning lanes that can be used simultaneously. Based on the Transportation Association of Canada's *Geometric Design Guide* (2017), it is assumed that a freight vehicle utilizes the left-turn lane closer to the center of the road. Additionally, Dixie Road includes three driving lanes in each direction, while the centerline contains a raised median separating the two driving directions. Lastly, a pedestrian crosswalk is provided where the off-ramp connects to the Dixie Road, but no crosswalks are provided for pedestrians to cross Dixie Road.

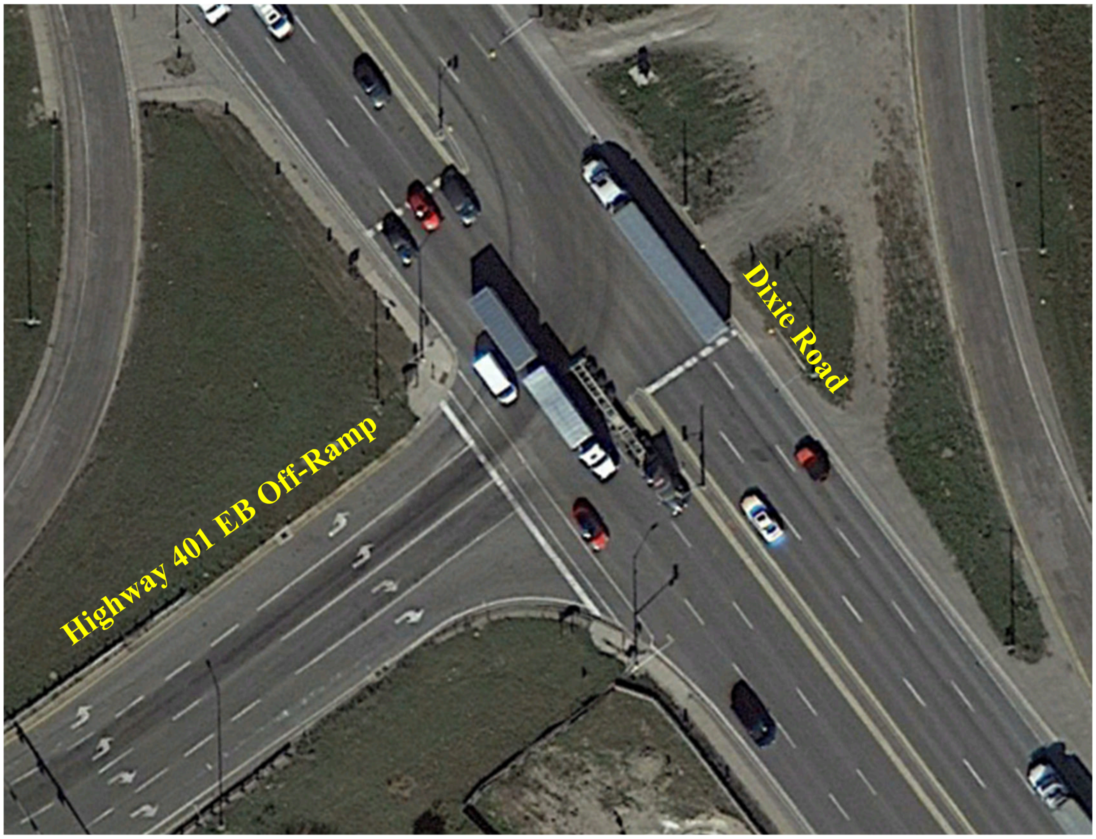

Figure 9: Satellite Image of the Hwy 401 NB Off-Ramp and Dixie Rd

3.2.2 – Dixie Road and Britannia Road

Next, the LCV makes a left turn from Dixie Road Northbound onto Britannia Road heading westbound. This intersection, shown in *Figure 10*, is a four-legged intersection, with Dixie Road, a major arterial road, in the North-South direction and Britannia Road, a minor arterial road, in the East-West direction. This intersection is fully signalized, including signal phases for left turns. In turning left onto Britannia Road, one dedicated left-turn lane is provided on Dixie Road alongside three through lanes. Britannia Road then has two traffic lanes in the westbound direction to receive the vehicles as they turn left from Dixie Road. In addition, the vehicles turning left from Britannia Road to travel southbound on Dixie Road have two simultaneous left-turn lanes available to them. Both roads also have a raised curb median separating the two driving directions. All four right-turning maneuvers have also been provided a dedicated right-turning lane, which is separated from the through lanes using a pedestrian island at all four corners of the intersection. Therefore, pedestrian amenities include signalized crosswalks connecting these pedestrian islands, as well as unsignalized crosswalks connecting the four pedestrian islands to the main sidewalks.

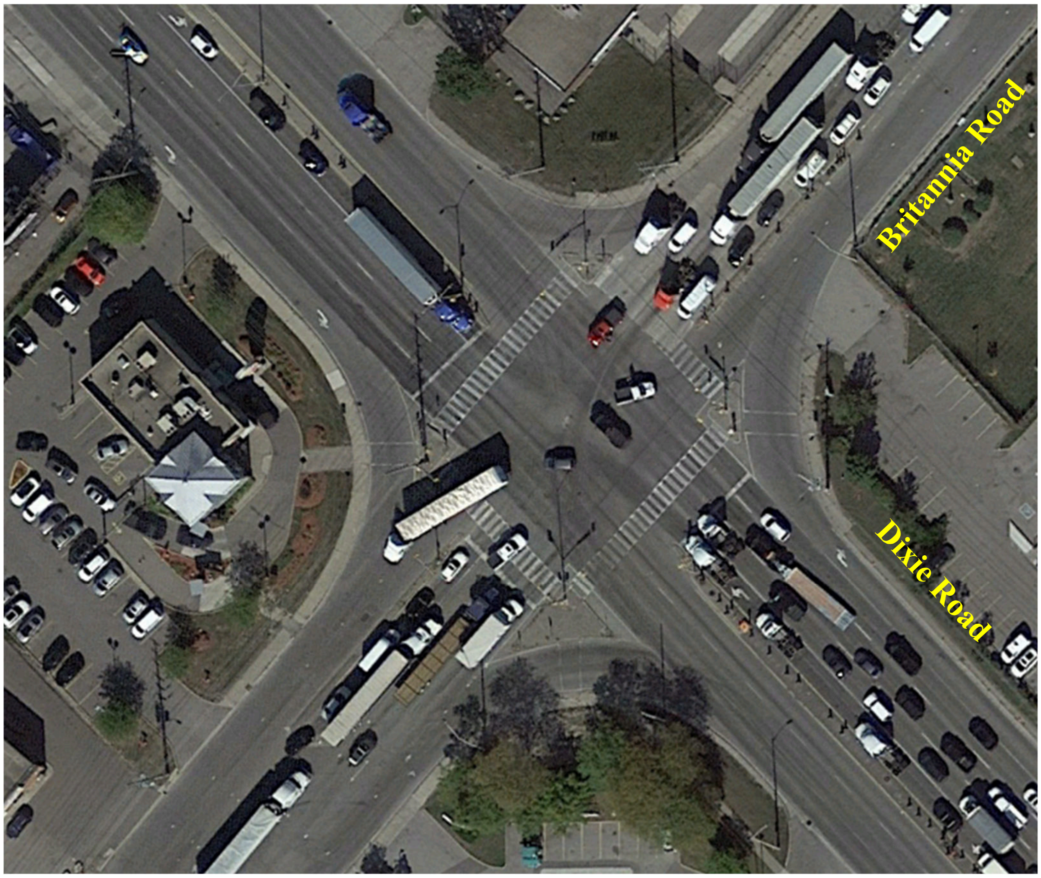

Figure 10: Satellite Image of Dixie Rd and Britannia Rd

3.2.3 – Shawson Drive and Westport Crescent (South Location)

After completing a right turn from Britannia Road onto Shawson Drive, the first unsignalized left-turning intersection of the route is encountered, turning left from Shawson Drive onto Westport Crescent. This crescent intersects with Shawson Drive twice, so this first intersection located further south will be denoted as Shawson Drive and Westport Crescent (South). This four-legged intersection, shown in *Figure 11*, is made up of two local roads with one assumed lane in each direction. Stop signs are used to control traffic on Westport Crescent, giving the right-of-way to vehicles along Shawson Drive. Pedestrian crossings are not marked on the road but are available in the North-South direction along Shawson Drive.

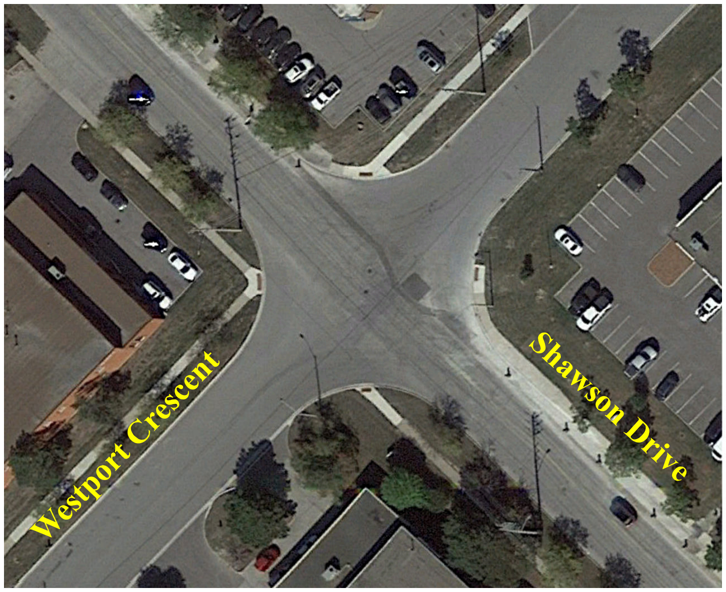

Figure 11: Satellite Image of Shawson Dr and Westport Cr S

3.2.4 – Shawson Drive and Westport Crescent (North Location)

Continuing along Westport Crescent, the road eventually connects back to Shawson Drive north of the previous intersection, allowing for this intersection to be denoted as Shawson Drive and Westport Crescent (North). This three-legged intersection enables vehicles to turn back onto Shawson Drive. Similar to the southern intersection, this intersection is unsignalized and only contains a stop sign on Westport Crescent. Therefore, the right-of-way is again granted to vehicles travelling along Shawson Drive. A pedestrian crosswalk is also provided at the stop sign where Westport Crescent meets with Shawson Drive, but is not clearly marked on the pavement.

Figure 12: Satellite Image of Shawson Dr and Westport Cr N

3.2.5 – Shawson Drive and Courtneypark Drive East

The next left turn involves turning from Shawson Drive onto Courtneypark Drive East, an arterial road with two lanes in each direction. This intersection is shown in *Figure 13*. As Shawson Drive approaches this four-legged intersection, the single lane of travel splits into two lanes, providing a dedicated left-turn lane that the vehicle uses to turn onto Courtneypark Drive East. The center of the road along Courtneypark Drive East is equipped with raised curb medians to separate the directions of traffic flow. This intersection is also signalized, with traffic lights controlling the different signal phases of the intersection. Lastly, pedestrian crosswalks are provided in all directions of the intersection.

Figure 13: Satellite Image of Shawson Dr and Courtneypark Dr E

3.2.6 – Courtneypark Drive East and Highway 410 On-Ramp

Lastly, after travelling westbound along Courtneypark Drive East, the route ends with a left turn onto the northbound Highway 410 On-Ramp. This is a four-legged intersection, where Courtneypark Drive is met with the Highway 410 on-ramp and off-ramp, as well as the entrance to a carpooling parking lot to the north. At this left turn, the two-lane Courtneypark Drive East has expanded into three lanes; two through lanes to continue westbound along Courtneypark Drive and one dedicated left-turn lane to enter the on-ramp. The left-turn lane is also separated from the through lanes by a painted median on the road and separated from the eastbound traffic lanes by a raised median along the centerline. This intersection is also fully signalized and is equipped with pedestrian crossings in the eastbound and westbound directions only. Satellite imagery of this intersection is exhibited in *Figure 14*.

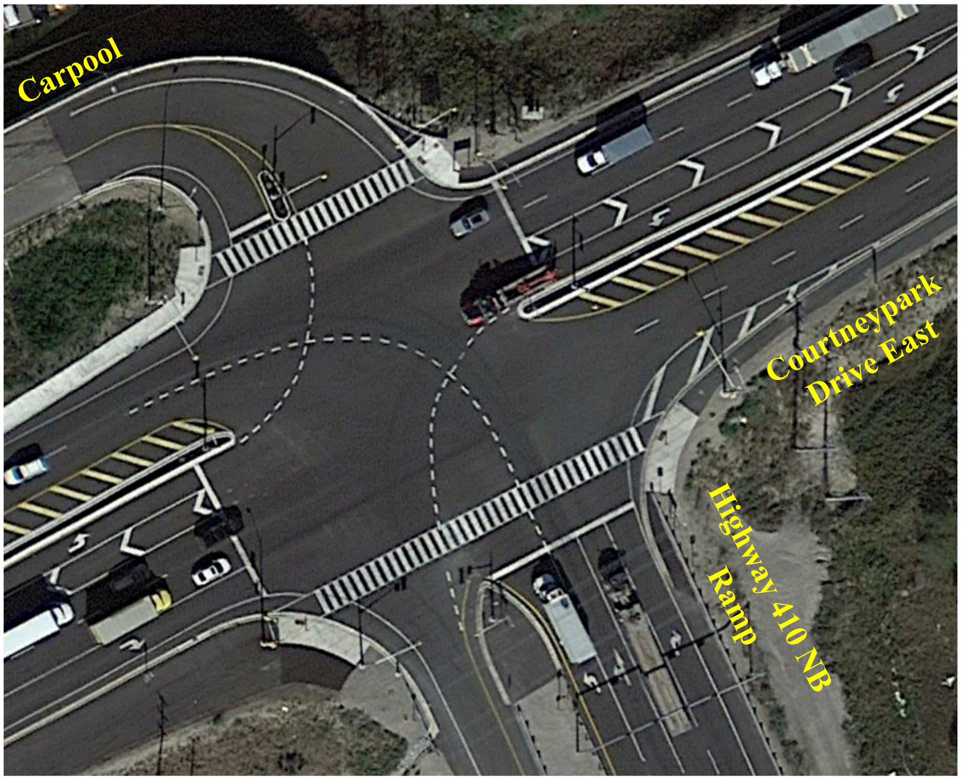

Figure 14: Satellite Image of Courtneypark Drive E and the Hwy 410 NB On-Ramp

Chapter 4. Swept Path Methodology

Throughout this thesis, two primary analysis methods are used to assess LCV safety in the chosen study area. The first analysis method is a swept path analysis, which will review conflicts that a typical LCV will experience with existing road geometry and infrastructure. The following sections will discuss the methodology used to complete this analysis.

4.1 – Intersection Digitization in AutoCAD

Before AutoTurn can be used to assess the swept path of LCVs, the physical environment must first be accurately replicated using the computer-aided drawing software known as AutoCAD. AutoTurn can then be used as a plug-in for AutoCAD to complete the swept path analysis within the drawn environment. To accurately replicate the physical environment, the geolocation feature is used to superimpose the drawing onto satellite imagery provided by Bing Maps at a 1:1 scale. To ensure that the accuracy of the dimensions in the imagery closely match the dimensions of the drawing, the NAD27-UTM17 coordinate system is used for the map. Throughout this process, many dimensions are double-checked using Google Earth Pro to verify their accuracy. One of the limitations of any satellite imagery is related to image quality, as image resolution and shadows could make it difficult at times to accurately replicate existing dimensions. Again, Google Earth Pro is used to overcome this, as Google Earth Pro offers both alternative satellite images to Bing Maps as well as street-level images. Additionally, the Courtneypark Drive-Highway 410 On-Ramp Intersection and Dixie Road-Britannia Road Intersection were both under construction during the time that Bing Maps updated their satellite images. Therefore, these two intersections are drawn using Google Earth Pro.

In drawing these intersections in AutoCAD, several features of the road infrastructure are replicated, including the widths of driving lanes, the curb-to-curb width of the roadways, road shoulders, presence of painted or raised medians, and the locations of crosswalks. Several layers are used in AutoCAD to differentiate between these different road features and to label each roadway. This process results in AutoCAD drawings of the six left-turning intersections shown in *Figure 8* where an LCV left turn will take place. An example of these drawings can be seen in *Figure 15*, which is the AutoCAD drawing of the intersection between Dixie Road and the Highway 401 Off-Ramp. In this drawing, curbs are shown in red, stopping lines and crosswalks in green, and traffic lanes in yellow. The remaining intersection drawings can be found in *Appendix A.1*.

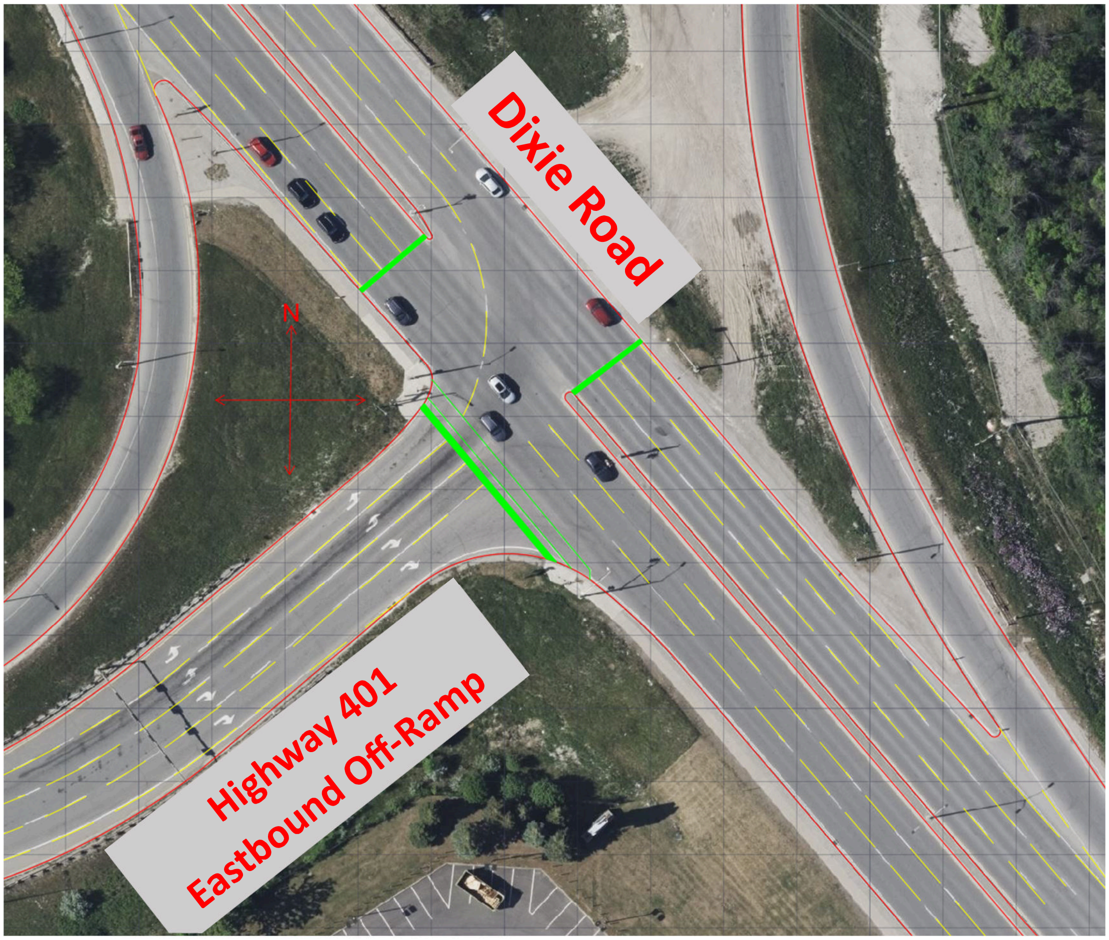

Figure 15: AutoCAD Drawing of the Dixie Rd - Hwy 401 Off-Ramp Intersection

4.2 – AutoTurn Modelling of Left Turns

Once the AutoCAD drawings of the six intersections of interest are completed, the next step is to model the paths travelled by a left-turning LCV at each intersection. AutoTurn software is used to assess the swept path envelope of LCVs as they complete the left turns at the six left-turning intersections. This analysis method is selected due to the need to assess multiple geometric design features of last-mile intersections. In using AutoTurn, conflicts between the LCV's swept path and the intersection geometry are directly identified. The IntelliPath feature in AutoTurn generates the optimal path of a vehicle within the drawn environment for a swept path analysis. This optimal path generation iteratively tests different turning paths of the vehicle based on several information inputs, such as vehicle types, vehicle travel speeds, and the type of swept path generated. In using this tool, an assessment is performed to determine whether an LCV can realistically complete a left turn at each intersection. A representation of the ideal turning route travelled by LCVs is generated if a left turn is possible. If the vehicle is unable to complete the movement, a path will be drawn up until the vehicle's swept path envelope experiences a conflict with the external environment.

The first step in applying IntelliPath to each intersection drawing involves defining the study area for the simulation to take place. A border is initially drawn around the relevant road segments to designate them as driveable areas that will be assessed for the left turn in the drawing. Exclusion lines are then identified, which define the edges of the roadway where the LCV swept path envelope may not cross beyond. In each drawing, several geometric road features are highlighted as exclusion lines to limit where the LCV may turn, including road edges, curbs, and medians. Depending on the left-turn scenario being tested, some or all adjacent traffic lanes may also be designated as exclusion lines to prevent the LCV from infringing in these lanes. Ultimately, this process yields the final driveable area, where any LCV movements within this area can be assumed to be safe and do not involve conflicts with infrastructure, adjacent lanes of traffic, or roadside amenities. For example, *Figure 16* highlights the outcome of the IntelliPath process for the intersection between Courtneypark Dr and Shawson Dr, with the driveable area highlighted in burgundy. As can be seen here, the driveable area is contained within the boundaries of the curbs of the road and the raised medians.

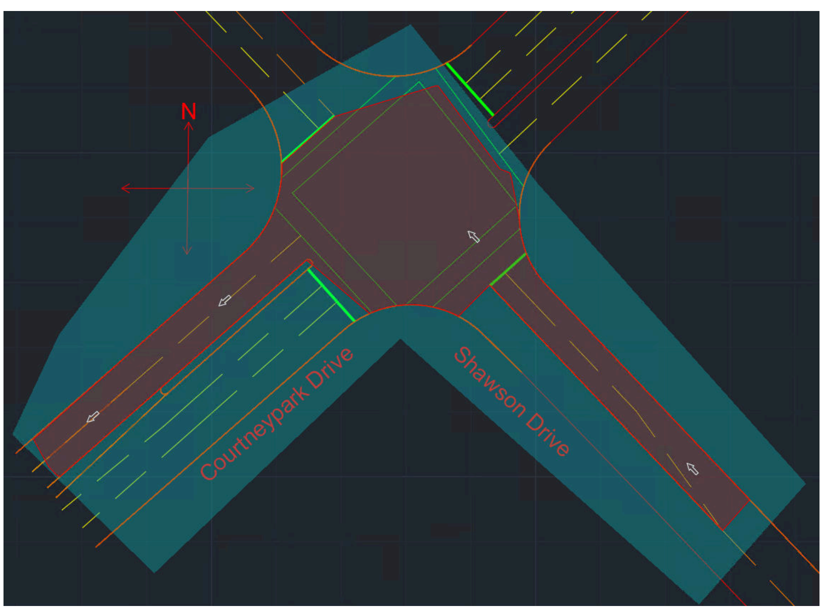

Figure 16: IntelliPath Boundaries for the Courtneypark Dr – Shawson Dr Intersection

Within this driveable area, the next step is to define the general direction of the proposed route the LCV will follow in each intersection. This includes selecting the starting point of the route and then slowly tracing the possible path of the LCV until the end of the route is reached and selected. In doing this, IntelliPath highlights the entire driveable area available to the LCV when travelling from the start and end points of the route and indicates the direction of travel. The area can also be adjusted manually to expand the route into any acceptable areas of travel that have not been highlighted. The white arrows seen in *Figure 16* show the direction of travel of the proposed route, travelling northbound on Shawson Drive and turning left to travel westbound on Courtneypark Drive.

With the driveable areas and route direction now specified, a vehicle must be configured and placed into the environment for the swept path to be generated. This includes selecting the type of vehicle to be simulated, the type of swept path generated, the starting position of the vehicle, and the vehicle's travel speed. An LCV model vehicle must be selected that will provide an accurate representation of the interactions with infrastructure and other road users. For the purposes of this study, a turnpike double is selected as the design vehicle. As discussed in *Section 2.2.1*, the turnpike double is the largest of the three common LCV types, with a maximum length of up to 38 meters. Since it is the largest LCV category, it is expected that conflicts with infrastructure and other road users are more prominent than when using a rocky-mountain double or a triple. Therefore, using a turnpike double would provide a better representation of the issues created in last-mile contexts. In reviewing the AutoTurn's built-in vehicle library, the WB-36 from the Edmonton 2018 (CA) library folder is selected. This is a turnpike double LCV with two 16.20-meter trailers for a total length of 38 meters. This vehicle is consistent with the maximum turnpike double dimensions outlined in *Section 2.2.1* and is the same vehicle permitted within the Region of Peel (Province of Ontario, 2022b). The full dimensions of the WB-36, as shown in the AutoTurn vehicle library, are exhibited in *Figure 17*.

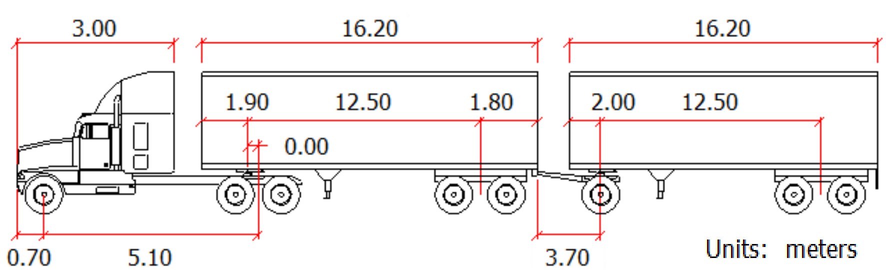

Figure 17: WB-36 Turnpike Double (Transoft Solutions, 2023)

In terms of the swept path envelope type, IntelliPath offers two options to assess conflicts: using the vehicle body envelope or the vehicle tire envelope. The vehicle body envelope is the physical area occupied by the entire vehicle's body when maneuvering the route, including any wide loads or trailer characteristics of the vehicle. On the other hand, the vehicle tire envelope is the area that will be occupied by the front and rear tires of the vehicle when completing a maneuver, ultimately disregarding any overhang of the vehicle body beyond the tires. Due to the need to assess LCV conflicts with raised medians, roadside amenities, and other road users, the tire-based envelope is not sufficient, as it would not recognize critical conflicts between the geometric environment and the parts of the vehicle body that extend beyond the axles of the LCV. Therefore, the vehicle body envelope is selected.

The vehicle travel speed also plays a key role in assessing the ability to complete these left-turning maneuvers, as higher speeds typically require more space to turn. According to the City of Toronto's (2017) *Curb Radii Design Guidelines*, the design turning speed used to assess large vehicles, such as trucks, as either the design or control vehicles is 5 km/hr. Thus, a 5 km/hr turning speed is selected in AutoTurn. The last step of the IntelliPath methodology is to place the vehicle at a starting point within the route area. The starting location and orientation of the vehicle has a major impact on the ability of the vehicle to navigate the area without infringing on restricted or undriveable areas. Thus, the orientation of the vehicle is always aligned with the lane lines or curb lines adjacent to the starting position. In providing a starting point and end point for the vehicle, IntelliPath will run simulations to determine whether the vehicle has the ability to complete the left turn required at the specified speed.

4.3 – AutoTurn Scenarios

To assess the viability of the selected intersections to accommodate the left-turning LCVs, multiple left turn scenarios are tested at each intersection. Taking inspiration from Saha's (2021) study of urban right-turning maneuvers of LCVs, the viability of each intersection is assessed from scenarios based on permissions to use different lanes of the road. As per the City of Toronto's (2017) Curb Radii Guidelines, it is common for freight vehicles, which are often considered as control vehicles, to be permitted to utilize additional lanes of traffic in order to complete turning maneuvers when presented with limited rights-of-way. Therefore, four key scenarios that mirror Saha's (2021) scenarios are utilized in this study based on permissions to use adjacent driving lanes on both the departing road and destination road, as shown in *Figure 18*. These scenarios, labelled A through D, are explained in detail in *Sections 4.3.1* through *4.3.4* and are assessed at each intersection by adjusting the exclusion lines within the IntelliPath simulations.

An initial turning speed of 5 km/hr is assumed for each of the left turns. IntelliPath then assigns a passing or failing grade for each scenario along with the optimal left-turning path in each case. A passing grade is assigned when the turnpike double is able to complete the projected left turn without coming into conflict with the existing road geometry. Meanwhile, a failing grade is assigned when Intellipath has determined that it is not possible for the LCV to complete the turn without conflicting with the road infrastructure. For scenarios where an intersection has successfully achieved a passing grade, IntelliPath is then tasked with finding the maximum speed at which the turn can successfully be completed. This provides an indication of how much leeway an intersection provides to an LCV since higher speeds require more space to complete turning maneuvers.

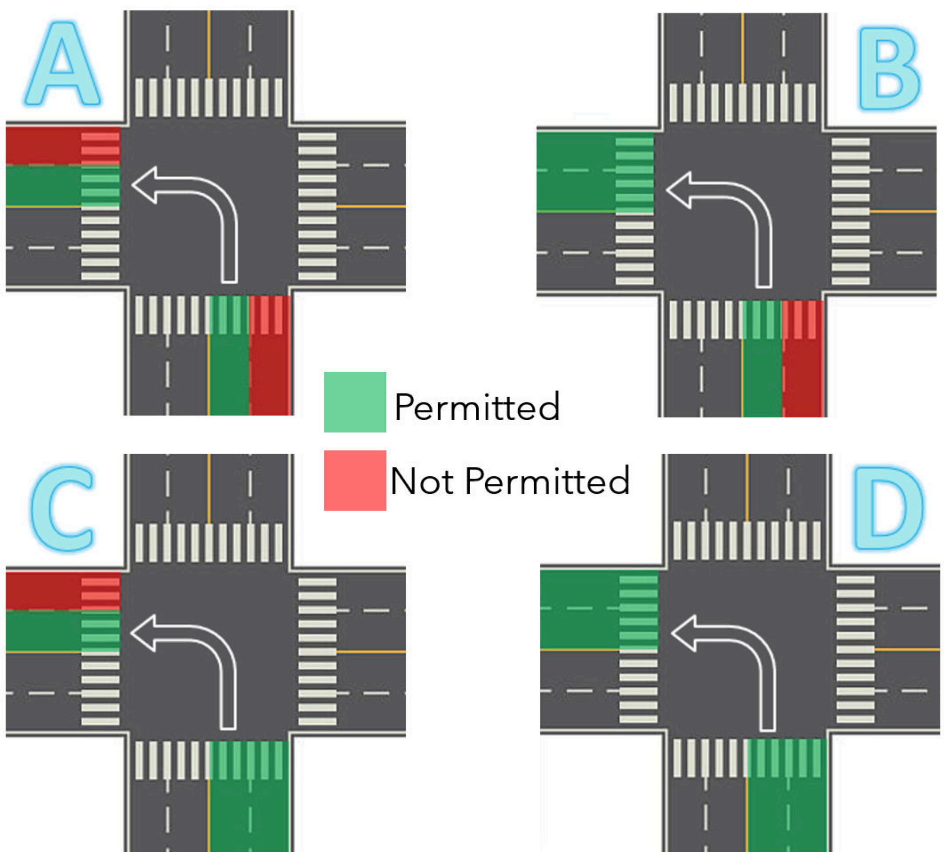

Figure 18: Swept Path Analysis Scenarios A-D

4.3.1 - Scenario A: Only One Lane Permitted on Both the Departing and Destination Roads

In Scenario A, only the intended left-turn lane of the departing road and the intended destination lane of the destination road are permitted for use. This is the most restrictive scenario, as it assumes that the LCV must make the left turn without infringing on any adjacent or oncoming lanes. This involves using the lane lines as additional exclusion lines in the IntelliPath simulation. An intersection design can be considered ideal for LCVs when it allows for a Scenario A left turn to be completed successfully.

4.3.2 - Scenario B: One Lane Permitted on Departing Road and Two Lanes Permitted on Destination Road

In Scenario B, the LCV is still restricted to using the intended left-turn lane on the departing road but is granted the freedom to use one additional lane on the destination road. In most cases, this will involve the LCV turning into the left-most lane of the destination road as well as the adjacent lane to the right, but in some cases can instead involve the adjacent oncoming lane as the additional lane. Thus, safety risks are presented when using an additional lane due to potential conflicts with other road users occupying these lanes.

4.3.3 - Scenario C: Two Lanes Permitted on Departing Road and One Lane Permitted on Destination Road

Scenario C is similar to Scenario B but instead restricts lane usage on the destination road to one lane. On the departing road, the intended left-turning lane can be used in addition to one adjacent or oncoming lane of traffic. This means that when the LCV is making a left turn, it starts from the leftmost departing lane and may expand into an additional departing lane if needed as it is turning into the leftmost destination lane. Conflicts with adjacent or oncoming road users on the departing road remains a risk in this scenario.

4.3.4 - Scenario D: Two Lanes Permitted on Both the Departing and Destination Road

Scenario D allows for the LCV to utilize one adjacent or oncoming lane on both the departing and destination roads. In this scenario, the LCV will start out in the leftmost departing lane and aims to arrive in the leftmost destination lane. However, the LCV may expand its path into an additional lane on either the departing or destination road to avoid conflicts and aid in the left turn. This is the least restrictive scenario, as the LCV is permitted to infringe on an additional lane at both ends of the left turn. At the same time, this scenario carries the most safety risk, as road users travelling in these lanes may conflict with the LCVs.

4.4 – Sensitivity Analysis

In analyzing each intersection, it is important to understand the existing buffer space at each intersection and scenario. A passing grade is assigned when the LCV completes the left turn without conflicting with the road geometry. Meanwhile, the buffer space is defined as the additional space available to LCVs between the existing intersection geometry and the intersection dimensions at which a passing grade changes to a failing grade. As part of the swept path simulations, two sensitivity analyses are completed independently to assess the buffer space available at each intersection. These include the lane width sensitivity analysis and the setback distance sensitivity analysis.

4.4.1 – Lane Width Analysis

The purpose of the lane width sensitivity analysis is to assess the amount of leeway available at each intersection when LCVs are making their left turns by reviewing the existing buffer space. This is achieved by adjusting the lane widths at each intersection to identify the lane width threshold (LWT) at which a passing grade changes to a failing grade and vice versa. To find the LWT, lane widths are adjusted in increments until the passing or failing status changes. In scenarios with passing grades in the existing geometry, lane widths are reduced until the LCV has no more room to maneuver without conflict, at which point IntelliPath produces a failing result. In failing scenarios, the lane widths are gradually increased until there is enough room for the LCV to complete their left turn without conflict, at which point IntelliPath changes its result from a failure to a pass.

Throughout this process, it is important to note that when making changes to the widths of the lanes, only the two primary lanes, the lanes involved in Scenario A, are adjusted in each instance. Referring to *Figure 19*, the four lanes in each scenario are denoted W, X, Y, and Z. Only the widths of Lanes W and Y will be adjusted to find the LWT by focusing on these two primary lanes. Meanwhile, Lanes X and Z are considered secondary lanes.

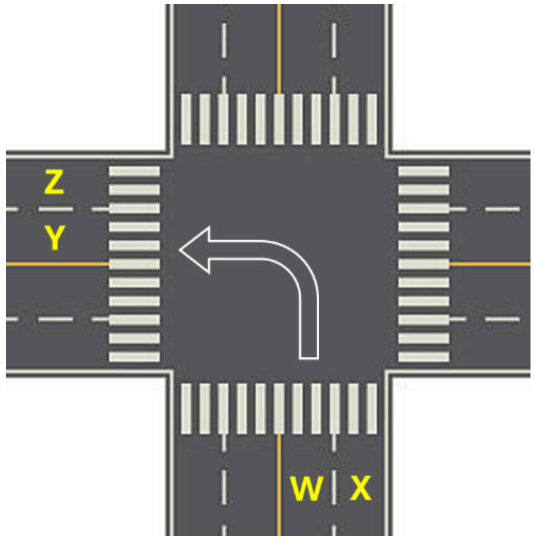

Figure 19: Lane Width Adjustment Nomenclature in Default Intersections

When increasing or decreasing lane widths, another notable rule in this process relates to how Lanes X and Z are affected when the primary lanes are changed. In scenarios where only one lane is permitted on a road, it is assumed that any width added to a primary lane encroaches into the secondary lane. Meanwhile, when assessing scenarios where two lanes are permitted on a road, the primary lanes are expanded while the secondary lane width is maintained, ultimately increasing the overall right-of-way width. For example, in assessing Scenarios A or B by adjusting lanes on the departing road, Lane W is adjusted, taking space from Lane X. On the other hand, for assessing Scenarios C and D by adjusting lanes on the departing road, the Lane W width is adjusted, while the Lane X width remains unchanged. Instead, as Lane W changes, Lane X's position moves and the total right-of-way width changes. Since the WB-36 turnpike double has a vehicle width of 2.6 m, this value is used as the minimum lane width. However, this means that there is no buffer space for the LCVs using the lane, which is not advisable in practice.

It should be noted that in the intersections involving Westport Crescent, the nomenclature of the secondary Lanes X and Z changes since there is only one lane heading in each direction. At these two intersections, the secondary lanes X and Z are assigned to the oncoming lanes on the departing and destination roads respectively, as demonstrated in *Figure 20*.

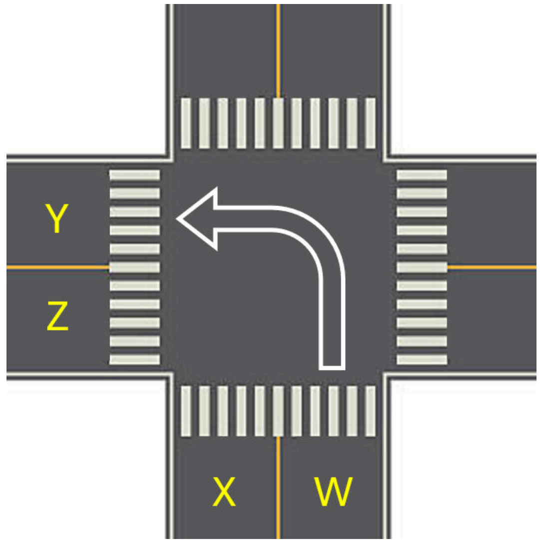

Figure 20: Lane Width Adjustment Nomenclature in Westport Crescent Intersections

It should be noted that the departing lane width sensitivity analysis is completed separately from the destination lane width sensitivity analysis. Therefore, this means that at each intersection, the LWT for Scenarios A through D are assessed by first adjusting Lane W until the grade status changes. Then in a separate analysis, additional LWTs for Scenarios A through D are assessed by adjusting the width of Lane Y until the grade status changes. For each intersection and scenario, the initial and final lane widths are also recorded.

<u>4.4.2 – Setback Distance Analysis</u>

The second part of the sensitivity analysis is assessing the setback distance at each intersection. In this analysis, adjustments are made to the stopping location of traffic in oncoming lanes to allow more space for LCVs to maneuver during their left turns. This involves moving back the stopping line a specified distance from the main intersection in the oncoming lane adjacent to the receiving lanes. In setting these stopping lines back, the wide shape of the LCV's swept path envelope will be accommodated by this additional space on the receiving road.

In completing this setback distance sensitivity analysis, the stopping line in adjacent oncoming lanes is moved at each intersection to determine the setback distance threshold (SBDT). That is, the setback distance at which a failing grade changes to a passing grade and vice versa. Similar to the lane width sensitivity analysis, the SBDT is identified for each intersection for Scenarios A through D. For each scenario, the initial setback distance (S_0) along with the final setback distance (S_f) is recorded. The initial and final setback distances are measured according to their perpendicular distance from the curb of the departing road. *Figure 21* shows a sample measurement of the existing setback distance for the intersection at Westport Crescent South and Shawson Drive, where the stopping line is currently set back 15 meters.

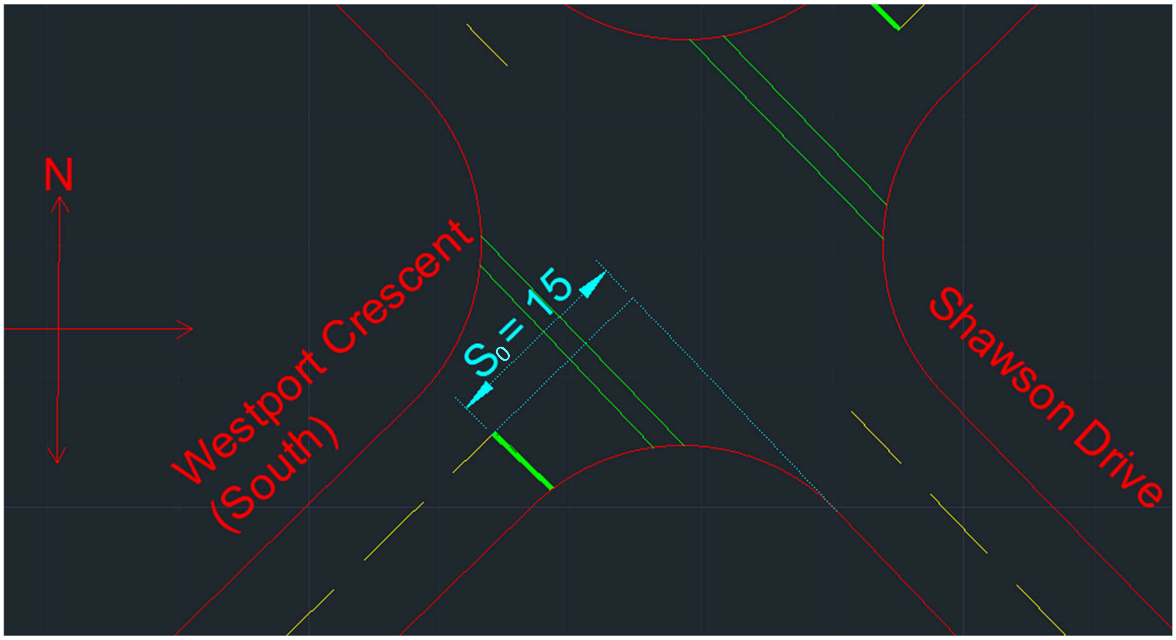

Figure 21: Sample Setback Distance Measurement - Shawson Dr and Westport Cr S

In performing the sensitivity analysis to change a failing grade to a passing grade, the stopping line in the oncoming lane is moved back to make space for the turning LCV. This is accompanied by cutting or removing any adjacent medians. In this process, cutting the median back to match the location of the stopping line and maintaining the Lane Y width is first attempted. If the entire median is cut back and the grade status has not changed, then the median is removed entirely. Cutting the median back is preferred over removing the median entirely, as removing the raised median represents a significant change to the existing intersection and essentially widens the receiving Lane Y. Furthermore, there may be some cases where the setback distance is increased beyond the limits of the previously drawn AutoCAD diagrams of the intersection. In this case, if the setback distance has been increased beyond this limit, a “does not exist” (DNE) status is assigned to the setback distance. This assumes that the setback distance is so far back from the main intersection that it would be unreasonable to implement as a solution.

In assessing when a passing grade setback distance changes to a failing grade setback distance, the setback distance is changed by moving the stopping line forward towards the intersection. This is accompanied by extending any medians to match the location of the stopping line. However, it is important to note that in the presence of crosswalks at each intersection, the stopping line is only extended until it is in line with the crosswalk. This is because it is illogical to extend the stopping location of vehicles into the designated pedestrian crossing zones. In these cases where the stopping location lines up with the crosswalk, a DNE is assigned to the SBDT.

4.5 – Logit Modelling

To further assess the impact of changing lane widths and setback distances on the passing or failing grade of an LCV during a left-turning maneuver, an additional step is taken using the results of the sensitivity analyses to determine the statistical significance of these variables. A binomial logit model is estimated using the passing or failing grade status of each scenario as the dependent variable. The explanatory variables then include the lane widths, lanes permitted, and setback distance. This process replicates the methodology used in Saha's (2021) study using NLOGIT software. The utility equations are shown below,

Equation 1: $$U_p = V_p + \varepsilon = \beta x + \varepsilon$$

Equation 2: $$U_f = 0$$

where U_p and U_f represent the total utility of passing or failing a given simulation, respectively. Then, V_p represents the observed utility of passing a given simulation. Additionally, ε is an error term assuming a logistic pattern, while β is a vector of parameters estimated for each variable x. The probability P of an LCV successfully maneuvering the left-turn is then given as follows:

Equation 3: $$P = \frac{\exp{(U_p)}}{\exp{(U_f)} + \exp{(U_p)}}$$

Two adjustments are made to the swept path analysis data prior to estimating the logit model. First, the average departing road width and the average destination road width are calculated, resulting in W_{Avg_Dep} and W_{Avg_Dest} variables respectively. These variables are calculated by dividing the sum of the available road width by the number of lanes available for use in each scenario, which is either one lane or two lanes. This treatment of the variables removes the inherent correlation between total lane width and the number of lanes.

The second adjustment adds intermediate data points for the lane width and setback distances between the existing baseline geometry and the inflection points when the grade status changes, as previously denoted with the lane width threshold (LWT) and setback distance threshold (SBDT), respectively. These data points were not originally recorded during the swept path analysis, but have been added to improve the sample size of the logit model and align with the data used by Saha (2021). Intermediate widths are added in 0.5 meter- or 1 meter-increments depending on the gap between the initial lane width and inflection lane width, with a maximum of five intermediate points. Intermediate points are also added into the model between the initial setback distance and inflection setback distance, with a maximum of five intermediate points added.

Chapter 5. Microsimulation Methodology

The other analysis performed in this thesis is a safety performance evaluation, which involves two primary steps. First, the road network is modeled in a simulated environment using PTV Vissim. Replication of existing signalization and traffic volumes is completed, while LCVs are added into the environment as potential road users. Then, the Federal Highway Administration's (FHWA) Surrogate Safety Assessment Model (SSAM) software is used to analyze trajectory files generated from the microsimulation results pertaining to collisions and near-miss collisions. Given the challenges and biases introduced when relying on limited historical collision data of LCVs to assess their safety, the development of a microsimulation model in conjunction with SSAM is justified as the analysis method selected for this project. By simulating LCV activity within an established road network model, their behavior with respect to other road users is observed, while SSAM software is selected due to its advantages and reliability in identifying conflicts between LCVs and road users in the network. The following sections will outline the methodologies followed to complete these two tasks.

5.1 – Road Network Digitization

In generating a microsimulation model of the study area, multiple aspects of the existing road network must be replicated. This includes defining road geometry, right-of-way rules, and traffic signalization.

5.1.1 – Physical Intersections

The first step in developing the simulation model involves recreating the physical aspects of the network within Vissim. Using the satellite imagery provided within Vissim, the road network is located on the map and is drawn to scale with the real-world dimensions. With each link drawn in the network, the direction of traffic, the number of lanes in each direction, ability to change lanes, and the presence of dedicated turning lanes are customized to replicate existing conditions. Additionally, the widths of each lane in the model are selected to match the existing lanes in the road network. Link dimensions are double-checked throughout this process using Google Earth Pro's measurement tools. From this, the road network is drawn along the entire route previously selected in *Figure 8* in both directions. Next, pedestrian facilities are included as part of geometric roadways, including walking areas adjacent to roads and pedestrian crosswalks by drawing pedestrian areas and connecting them with links. In drawing all links, pavement markings are used to make the model more aesthetically pleasing, including road arrows to identify turning lanes at intersections and zebra crossing patterns on pedestrian links that serve as crosswalks. *Figure 22* below shows an example of the completed intersection drawing in Vissim at Dixie Road and Britannia Road.

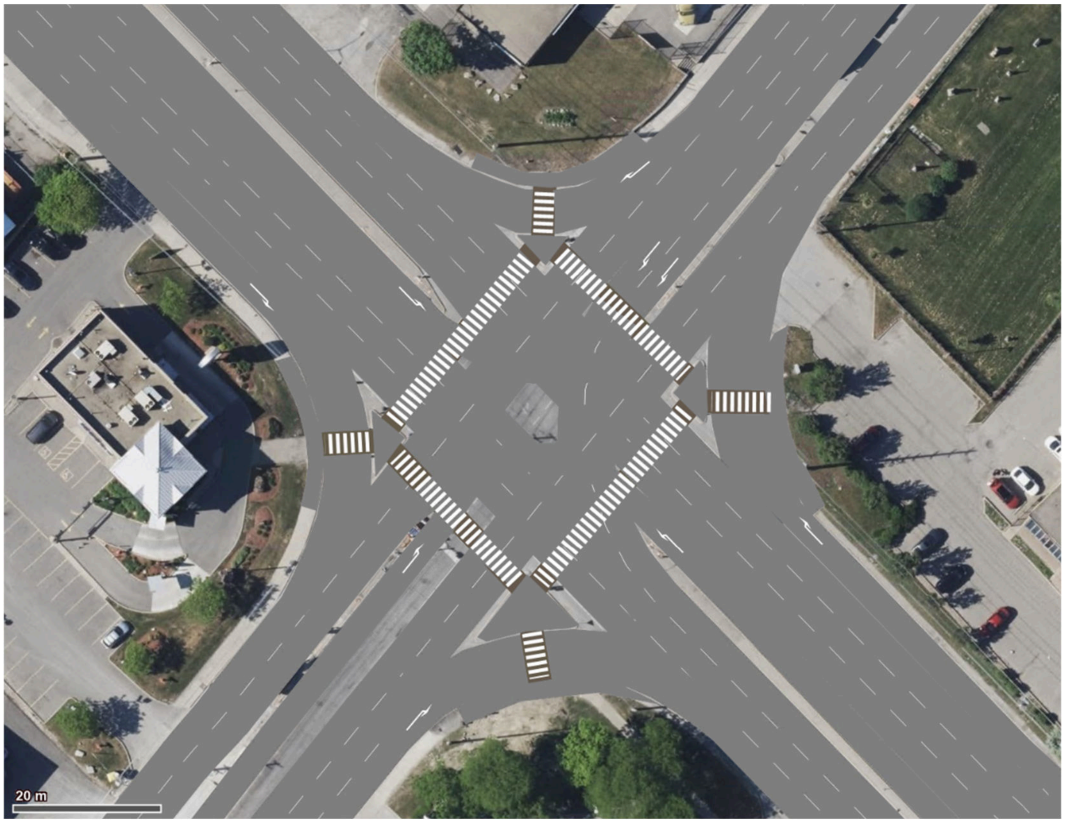

Figure 22: Dixie Rd and Britannia Rd in Vissim

This process was repeated throughout the entire network to accurately reflect the area's existing infrastructure. The only intersection where this process differed was at the Highway 410 ramps at Courtneypark Drive, as the Bing Maps satellite imagery provided within Vissim shows the area under extensive construction. Therefore, the satellite imagery from Google Earth Pro was instead used as a guide for the dimensions and layouts of this intersection.

5.1.2 – Intersection Right-of-Way Priorities

The next key step in developing the microsimulation model is to identify the rights-of-way of vehicles travelling on links that conflict with each other. Vissim automatically identifies potential conflicts occurring between traffic links. Therefore, these rights-of-way must be adjusted to determine which road user has the right-of-way at each conflict and which road user must yield.

In general, vehicles completing through movements always have the right-of-way over vehicles completing turning movements. Additionally, pedestrians on crosswalks always have the right-of-way over vehicles passing over pedestrian links. Furthermore, many additional conflicts identified by the software are left as passive, as it is expected that the implementation of traffic signals will prevent any of these conflicts from occurring. The result of this process at Dixie Road and Britannia Road is shown in *Figure 23*, where each conflict at the intersection has been resolved, with green links having the right-of-way over red links.

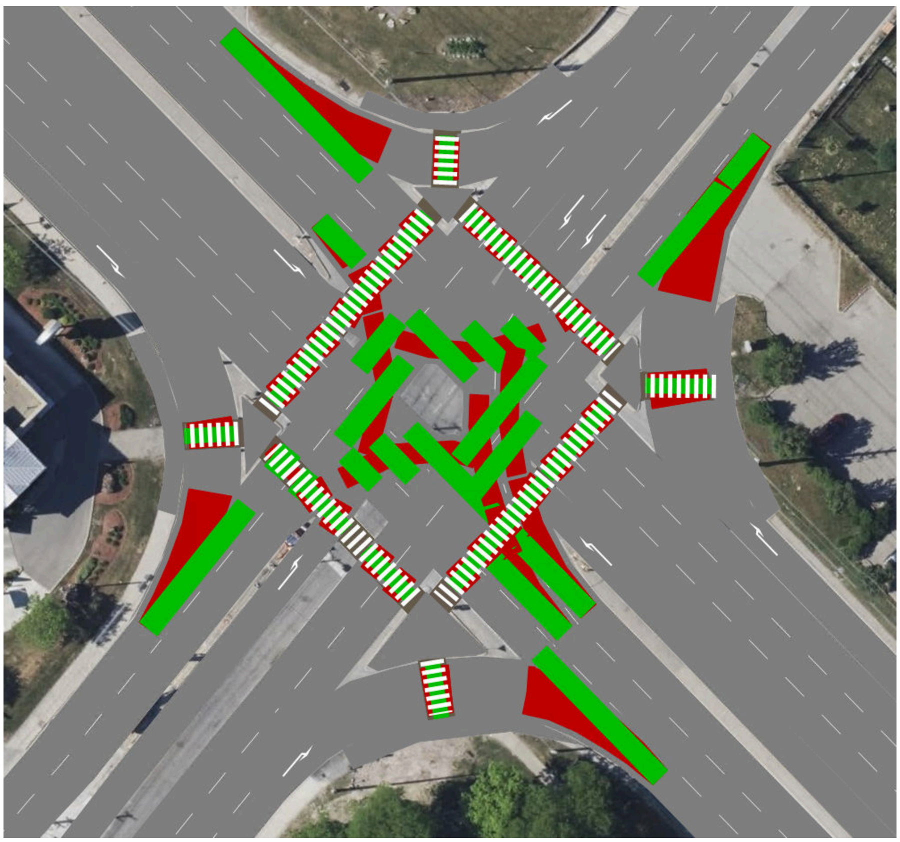

Figure 23: Conflict Resolution at the Britannia Rd - Dixie Rd Intersection

<u>5.1.3 – Traffic Signalization</u>

Another key factor in replicating the existing environment for the base scenario is the implementation of traffic controls at intersections. Throughout the selected route, there are nine signalized intersections, which are controlled by traffic lights in all directions of travel, and four unsignalized intersections, which are controlled by stop signs in either some or all directions of travel. Within Vissim, these two scenarios must be handled using separate network functions.

Starting with the four unsignalized intersections (Tristar Dr-Shawson Dr, Westport Cr N-Shawson Dr, Westport Cr S-Shawson Dr, and Vipond Dr-Britannia Rd), Google Earth Pro is used to review the usage of stop signs at these locations. In each case, stop signs are placed on the appropriate links relative to their locations on Google Earth Pro to allow for the approaching vehicle to stop at an acceptable distance from the intersection to safely assess their next maneuver. At unsignalized intersections controlled by stop signs only, it is assumed that pedestrians crossing the road may enter a crosswalk at any time, so it is the responsibility of vehicles to halt at the stop signs and yield to crossing pedestrians.

On the other hand, signalized intersections are recreated within Vissim using the signalization data provided by the City of Mississauga and the Region of Peel. While provided in different formats between the two government agencies, the data is interpreted to fill in many of the basic signalization requirements, including amber times, red times, maximum and minimum green times, pedestrian green times, and pedestrian "flashing don't walk" (FDW) times for each signal phase. Additionally, these files also provide information on the types of signal phases used at each intersection, such as the inclusion of protected or protected-permitted left turn phases, as well as the level of actuation at each intersection. Furthermore, the Region of Peel provided multiple data sets based on the AM peak, PM peak, and off-peak periods. Therefore, for this thesis, the PM peak values are used from the Region of Peel data sets. In the signal control tab on Vissim, a new signal controller is created for each intersection using a ring-barrier controller and each signal group is defined with their specified timings, phase concurrencies, phase patterns, and associated pedestrian timings. For each signal phase, actuation is implemented by specifying vehicle detectors for signal phases and their resulting timing extensions.

Throughout this process, it should be noted that manual data cleaning was required at times where errors were present in the provided data, such as in situations where concurrent signal cycles contained vastly different cycle lengths or where pedestrian green and FDW times were too short or long relative to their concurrent traffic signal phase. Additionally, another specific scenario that arose was a lack of information available for the signalization at the Courtneypark Dr-Highway 410 Northbound On- and Off-Ramp intersection. Here, a unique situation arose where signalization data was not available for this intersection, but was available for the Courtneypark Dr-Highway 410 Southbound On- and Off-Ramp intersections, which is outside the study area but is almost a mirror image of the northbound ramp intersection. Therefore, a mirrored signal timing is assumed for this intersection.

With the signal controller set up for each intersection, the final step is to draw the signals in the network. This involves placing signal heads on links in the network to represent the locations where the vehicles are required to stop while waiting for the light. Signal heads are also placed on pedestrian links where pedestrians are required to wait when at a signalized crosswalk. For actuated intersections, detectors are placed ahead of each signal head and assigned their corresponding signal controller and phase to extend green times when detecting passing vehicles or pedestrians. An example of a completed intersection signalization is shown in *Figure 24* for the Britannia Rd-Dixie Rd intersection. In this figure, signal heads are shown as red lines and detectors as dark blue rectangles.

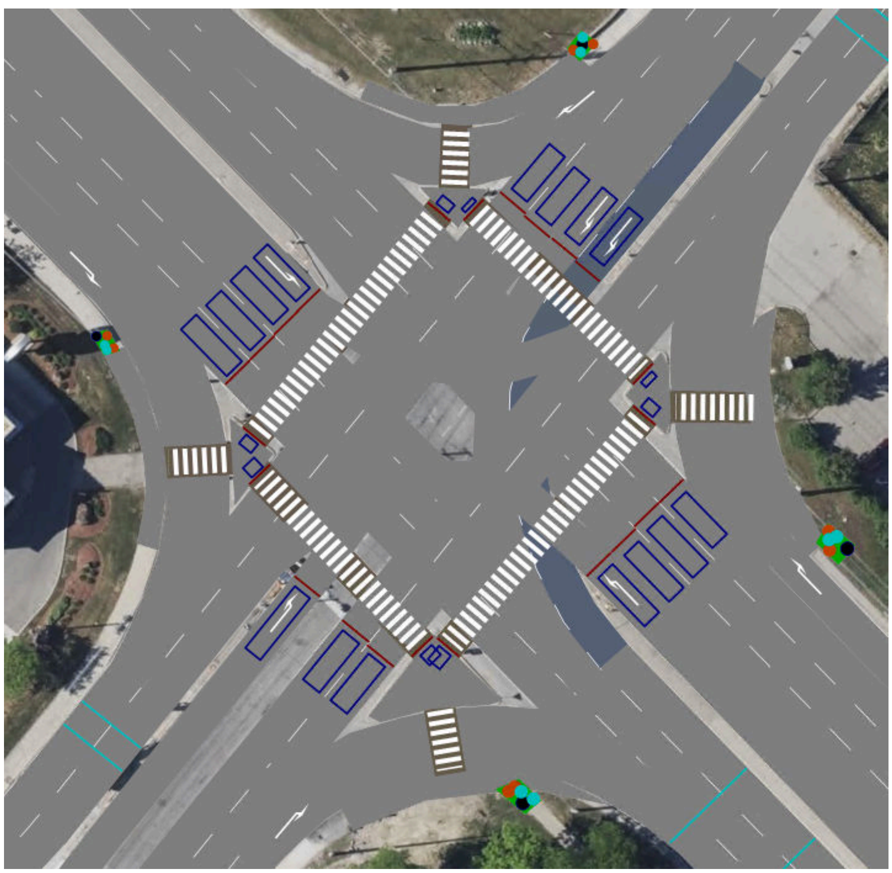

Figure 24: Vissim Signalization for the Britania Rd-Dixie Rd Intersection

5.2 – Traffic Definition

Once the physical environment is established in Vissim, the next step is to define the vehicles that are travelling within the network. This includes defining the volumes and types of vehicles entering the network at each intersection, while also outlining their behavioral properties. Furthermore, pedestrian volumes and behaviors must be defined. Before traffic volumes can be entered into Vissim from the raw data, a volume balancing procedure must first be completed to ensure the discrepancies between different data sets are eliminated.

5.2.1 – Volume Balancing and Turning Proportions

As outlined in *Chapter 3. Data*, the traffic data that will be used in the Vissim simulation has been provided by the City of Mississauga and the Region of Peel. This volume data will be used as the vehicle inputs throughout the traffic network. However, before this data can be applied to the simulation, a volume balancing procedure must be completed. Volume balancing is the process of resolving discrepancies in collected traffic volume data between adjacent locations in a road network (Wisconsin DOT, 2019a). In this thesis, volume balancing is needed for two reasons. First, volume balancing is typically needed in instances where traffic data has been collected across multiple time periods and sources, leading to inconsistencies and errors in traffic volumes between adjacent intersections (Wisconsin DOT, 2019a). The volume data being used here is provided by two independent entities, where Dixie Road data is provided by the Region of Peel and other signalized intersections are provided by the City of Mississauga, as well as across multiple months, years, and times of day. As a result, the provided data when observed together exhibit large discrepancies in traffic volumes between adjacent intersections in the network. Second, the volume data provided by the City of Mississauga and the Region of Peel does not include data pertaining to the four unsignalized intersections in the network, as well as data missing from the Courtneypark Drive East-Highway 410 Northbound Ramp intersection. As a result, volume balancing is applied to ensure that the generated volumes at these intersections are consistent with the known volumes at adjacent intersections.

To balance the traffic volumes obtained from the City of Mississauga and the Region of Peel, five main steps are followed. These are selecting a uniform study period, determining the turning proportions for the 2022 time period, calculating the volume and turning proportions for intersections with missing data, applying the Wisconsin DOT’s volume balancing tool, and manually resolving remaining imbalances in the network. The following text will briefly explain this extensive process, but a more detailed explanation of each step can be found in *Appendix B.1*.

The first step involves selecting a uniform time period for the collected traffic data. The data provided by each jurisdiction ranges across several years as well as times throughout the day. Since several intersections already have data provided for 2022, this year was selected as the study year. Using an annual growth rate (AGR) developed from a local traffic study of Courtneypark Drive East (Soo et al., 2015), the remaining intersections are converted into 2022 traffic data. Furthermore, from the several periods provided for each intersection throughout the day, the PM Peak Period is selected as the study period to be consistent with the signalization time period. The process of converting this data to the 2022-year data using the appropriate AGR can be found in *Appendix B.1.1*.

Next, the turning proportions must be determined for the 2022-year data. The turning proportions refer to the proportion of vehicles turning left, turning right, or travelling through from each leg of the provided intersection. The City of Mississauga and the Region of Peel each provided the corresponding vehicle turning proportions for the intersections they had data on, referred to as the known intersections. Since these values are recorded as proportions of the total traffic volume, these turning proportions are still used for each of the intersections they provided, regardless of whether they had been adjusted to represent 2022 data.

Now that the traffic volumes and turning proportions have been determined for the known intersections for the year 2022, this data must be used to derive the traffic volumes and turning proportions for the five intersections for which no data was provided, referred to as the unknown intersections. Starting with the turning proportions, these are determined for the unknown intersections by calculating the average proportion of turns in each direction across all known intersections, while making adjustments based on the layout and number of legs in each intersection. Then to calculate the traffic volumes at unknown intersections, data from adjacent known intersections is applied. The proportion of traffic volume on the major and minor road in each known intersection is calculated, and these proportions are averaged across all known intersections. These average volume proportions on the major and minor roads, along with the incoming traffic volumes from adjacent known intersections and the average turning proportions calculated earlier in this step, are used to calculate initial traffic volumes in each direction for every unknown intersection. A detailed description of this methodology, examples and figures for the different intersection types, and its limitations, are discussed further in *Appendix B.1.3*.

With initial values for traffic volumes and turning proportions in each direction now provided for both known and unknown intersections, volume balancing can now be completed. The Wisconsin DOT's (2019a) *Traffic Engineering, Operations & Safety Manual* provides a tool to balance traffic volumes in a network and reduce inconsistencies between volumes at adjacent intersections. Within this tool, this thesis' network is replicated by adding in intersection templates and inputting each intersection's respective traffic volumes and turning proportions from the previous steps. By inputting the traffic volume values, the balancing tool calculates the volume imbalance between each set of adjacent intersections. The tool then uses an iterative optimization process to minimize the total volume imbalance across the thirteen intersections. In running the balancing tool, the total network imbalance for this intersection network is reduced from 3730 to 7 vehicles. The few remaining imbalances, which have all been reduced to imbalances of 1 vehicle, are resolved manually by increasing or decreasing a corresponding turning movement by one to eliminate the imbalance. The root normalized square error (RNSE), calculated for each turning movement, is used to assess the difference between the initial traffic volumes and final balanced traffic volumes, as shown in the following equation:

Equation 4: $$RNSE = \sqrt{\frac{(y_1 - y_0)^2}{y_0}}$$

In this equation, y_0 represents the initial traffic volume for a selected turning motion before volume balancing is completed and y_1 is the final balanced traffic volume for the same turning motion. According to the Wisconsin DOT (2019b), a RNSE value less than 5 is generally acceptable. As shown in *Table 5*, only 14.9% of turning movements resulted in RNSE values greater than this threshold, indicating the balanced traffic volumes here for the overall model are acceptable for this thesis. Looking closer at specific intersections, the only two intersections that yielded less favourable RNSE results are the two Dixie Road-Highway 401 Off-Ramp intersections, which each had three out of four of their RNSE values greater than 5. However, since this thesis is focused on a comparative analysis of safety relative to this base scenario model, the traffic performance of the network is less important. Since the overall model yields acceptable RNSE results, this issue with Intersections 1 and 2 can be disregarded. *Section B.1.4* of *Appendix B* showcases the fully balanced traffic volumes in each direction along with their calculated RNSE values.

Table 5: Balanced Volume RNSE Summary

Intersection Number	Intersection	Error Level Acceptable (RNSE < 5)	Error Level Unacceptable (RNSE > 5)
1	Dixie Road & Highway 401 EB Off-Ramp	1	3
2	Dixie Road & Highway 401 WB Off-Ramp	1	3
3	Dixie Road & Shawson Drive	10	2
4	Britannia Road & Dixie Road	12	0
5	Britannia Road & Vipond Drive	6	0
6	Britannia Road & Shawson Drive	12	0
7	Westport Crescent South & Shawson Drive	10	2
8	Westport Crescent North & Shawson Drive	6	0
9	Meyerside Drive & Shawson Drive	12	0
10	Tristar Drive & Shawson Drive	6	0
11	Courtneypark Drive & Shawson Drive	9	3
12	Courtneypark Drive & Tomken Road	9	3
13	Courtneypark Drive & Highway 410 NB Ramps	9	2
Total		103 (85.1%)	18 (14.9%)

Furthermore, the final step of the traffic volume balancing relates to Westport Cr, as the traffic volume imbalance between each end of the crescent is not accounted for by the volume balancing tool. Therefore, these imbalances within Westport Cr are resolved by adding additional traffic sources and sinks within the crescent.

With a fully balanced set of traffic volumes, these volumes can now be applied to the Vissim model as vehicle inputs. These vehicle inputs are placed throughout the network on the links that enter the network. For example, at a 4-legged intersection located between two adjacent intersections, vehicle inputs will only be placed on the links entering that intersection that do not connect with the adjacent intersections.

5.2.2 – Vehicle Compositions

Vehicle compositions in Vissim refers to the proportion of vehicle inputs that are made up of passenger cars, light goods vehicles (LGV), heavy goods vehicles (HGV), and buses. Vissim provides the user with the option to edit vehicle compositions by assigning proportions of each vehicle type to the vehicle inputs. In reviewing the vehicle composition data provided by the City of Mississauga and the Region of Peel, there are some inconsistencies in the data types between the two sources. In the Region of Peel data, vehicle compositions are given for the PM peak hour of traffic, including percentages of passenger vehicles, single-unit trucks, buses, and articulated trucks. In assigning these proportions in Vissim, these are considered passenger cars, LGVs, buses, and HGVs respectively. For the City of Mississauga data, compositions are only provided for passenger cars and trucks, not differentiating between LGVs and HGVs, while also not providing bus data despite transit routes being present on Courtneypark Drive. At these intersections, two modifications are completed to input the data. First, the percentage of trucks are assumed to be equally split between LGVs and HGVs. Second, buses are assumed to be operating in the east-west direction along Courtneypark Drive, so the bus proportions from the Dixie Rd-Highway 401 Eastbound Off-Ramp intersection are copied for the Shawson Dr-Courtneypark Dr intersection and the Tomken Dr-Courtneypark Dr intersection.

For the unknown intersections, for which no data was provided by the Region of Peel or the City of Mississauga, vehicle compositions are determined from adjacent intersections and applied. For each unknown intersection, the average truck percentage is taken from the two adjacent intersections. This average truck percentage is then evenly split between LGVs and HGVs. From this, a final proportion of passenger cars, LGVs, and HGVs is calculated. It is assumed that on these local roads, buses are not operational. For example, consider the vehicle composition calculation for Vipond Dr and Britannia Rd in the southbound direction. The two nearest known intersections, as shown in *Figure 25*, are the Britannia Rd-Dixie Rd intersection and the Britannia Rd-Shawson Dr intersection. To calculate the truck percentage at this intersection in the southbound direction, the eastbound truck percentage taken from Dixie Rd (25.78%) and westbound truck percentage from Shawson Dr (15.08%) are averaged together, providing a southbound truck percentage of 20.43% on Vipond Rd. In evenly splitting this truck percentage between HGVs and LGVs, and accounting for the volume of passenger cars making up the remaining percentage of vehicles, the final vehicle composition for this vehicle input is 79.57%

cars, 10.21% HGVs, and 10.21% LGVs. The full results calculated for all unknown intersection vehicle compositions can be found in *Table 30* of *Appendix B.2*.

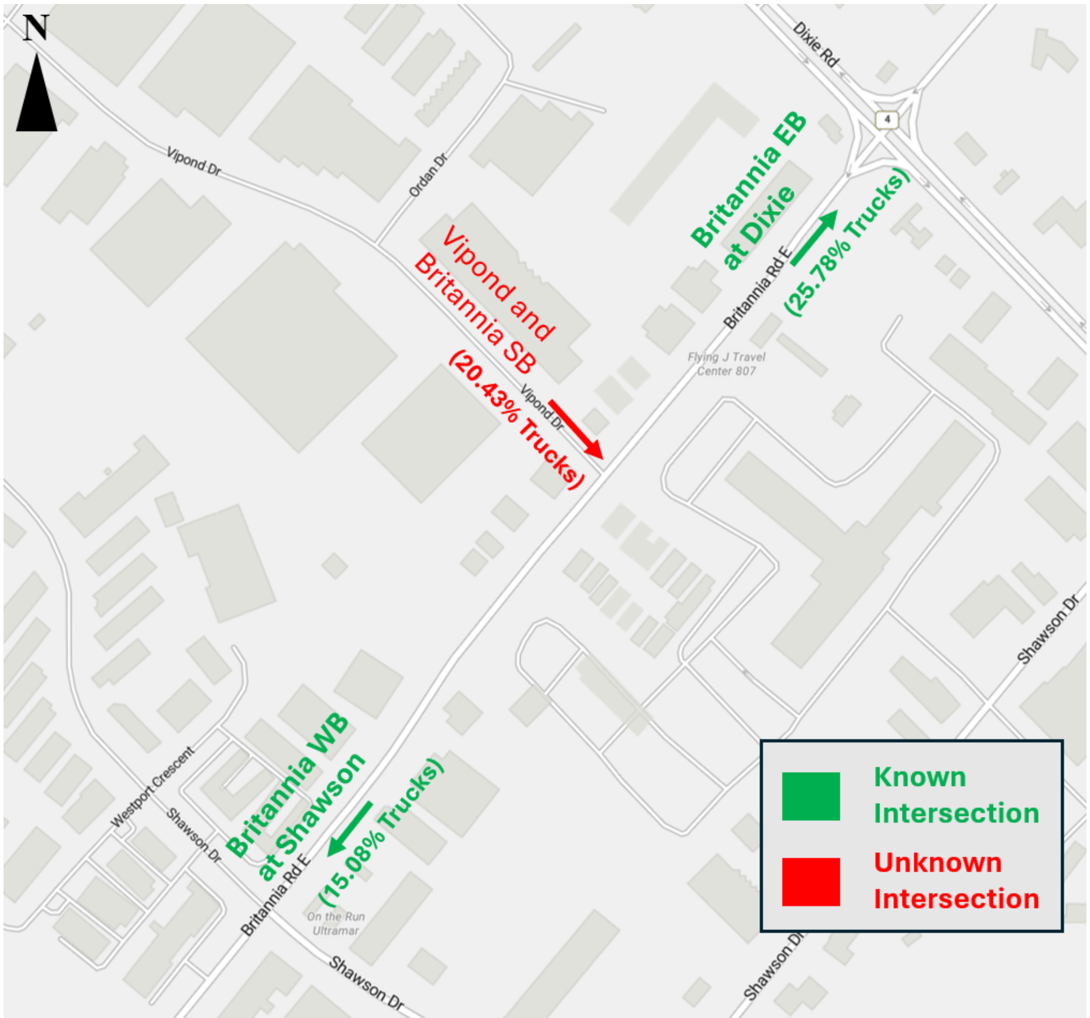

Figure 25: Vipond Dr-Britannia Rd SB Vehicle Composition Calculation

The only unknown intersection that has the vehicle compositions determined differently from this process is the Courtneypark Drive-Highway 410 intersection. This is due to the vastly different characteristics of this intersection compared to the other unknown intersections, as this intersection includes both a carpool parking lot and a highway off-ramp in addition to an arterial road. For this intersection, three vehicle proportions are needed; the Highway 410 Off-Ramp, Courtneypark Drive in the eastbound direction, and the carpool vehicle source. The highway off-ramp is determined by copying the Highway 401 eastbound vehicle compositions at the Highway 401 Eastbound-Dixie Road intersection. For Courtneypark Drive, the Dixie Road northbound vehicle compositions are copied from the Highway 401 Eastbound-Dixie Road intersection. Then for the carpool, it is assumed that passenger vehicles will be the predominant vehicle type here and that no HGVs would use this area on an average day. Therefore, it is assigned an assumed composition of 96% passenger cars, 2% LGVs, and 2% buses.

5.2.3 – Vehicle Routing Decisions

Once these vehicles are input into the network, the next step is to determine their routes upon entering each intersection. The “static vehicle routing decisions” function is used in Vissim, where the start of each route is specified for each leg of an intersection. Each of these route starting points is then assigned all possible end points in the intersection based on the possible turning and through movements. In assigning turning proportions to each route, they are applied using the traffic volume turning proportions previously obtained from the volume balancing processes.

Notably, these turning proportions for all intersections are only applicable for cars, buses, and LGVs. For the routing decisions of HGVs, separate static routing decisions are established to better represent their turning characteristics. In the data provided by the Region of Peel and the City of Mississauga, the turning proportions specific to trucks are also provided for each intersection. Therefore, for the known intersections, additional static routing decisions applicable only to HGVs are drawn and proportioned based on this data. For unknown intersections, where no data was provided by the City of Mississauga or the Region of Peel, the same proportions calculated in the volume balancing process are applied to HGVs, LGVs, buses, and cars alike. Lastly, one final specific detail relating to truck turning movements relates to restrictions on the lanes they can use when turning. In scenarios where an intersection has a double left-turn lane, it is assumed that tractor-trailers will utilize the rightmost left-turn lane (Transportation Association of Canada, 2017). Therefore, links on the left side of a double left-turn lane are edited in Vissim to restrict HGVs from using these lanes. *Figure 26* exhibits the Britannia Rd-Dixie Rd intersection with all vehicle routing decisions implemented, where the start of each route is highlighted with a pink line and the end of each route highlighted with a neon blue line. The leftmost left-turn lane of the double left-turn where the usage of HGVs is restricted is also shown in this figure, shaded in navy blue.

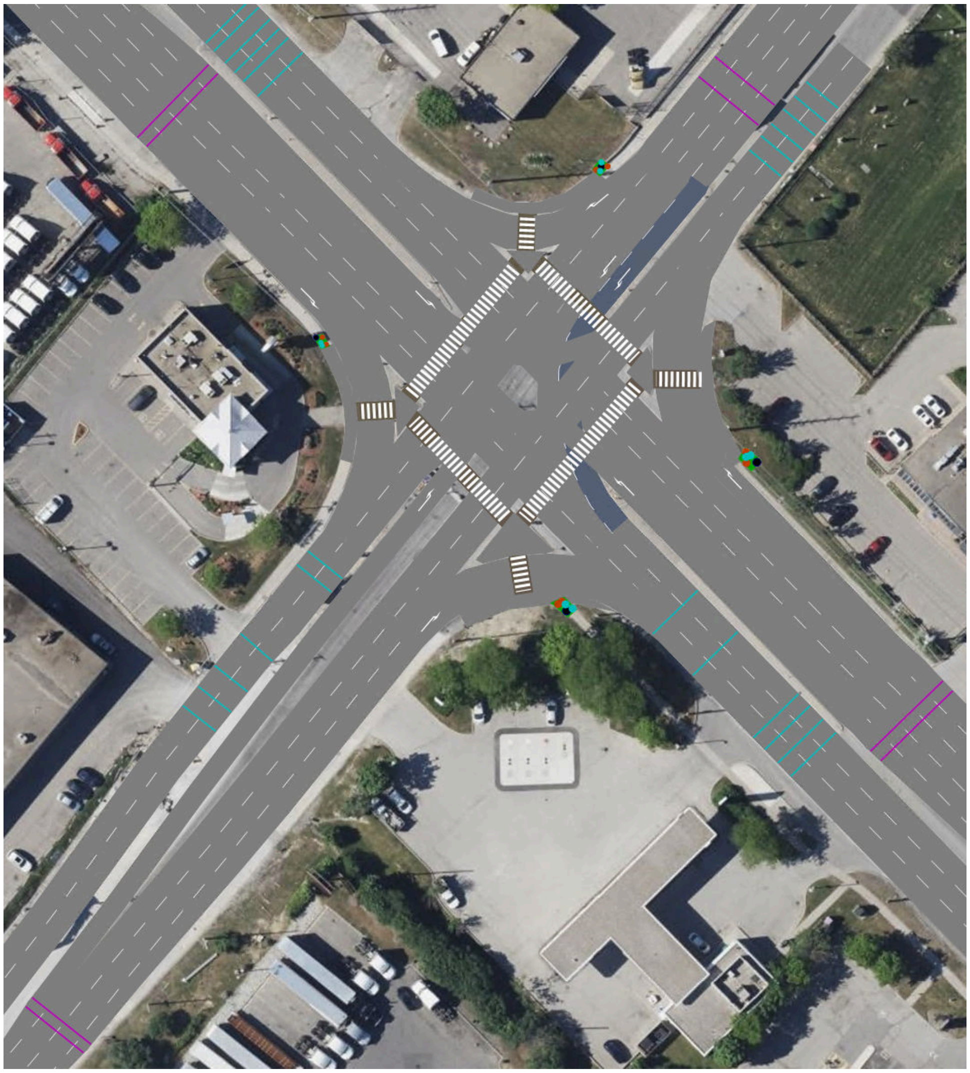

Figure 26: Britannia Rd - Dixie Rd Intersection with Completed Routing Decisions

5.2.4 – Vehicle Behavior

Lastly, setting the vehicle behavior in the Vissim model is vital to ensure vehicles in the simulation behave similar to vehicles in a real-world scenario. One aspect of this is their travel speeds, as many different speed limits are in place on the roads throughout the network. At the same time, it is reasonable to expect that not all drivers will travel at the exact speed limit, with many driving above or below the posted limit. Therefore, speed limits are set in the network along with speed distributions that drivers would be expected to follow. A summary of the speed limits and speed distributions is shown in *Table 6*. Any roads without posted speed limits are assigned assumed speed distributions.

Table 6: Speed Limits and Distributions of Roads in Vissim Model

Road	Travel Direction	Posted Speed Limit (km/hr)	Speed Distribution (km/hr)
Courtneypark Dr	EB	60	58 - 68
	WB	70	68 - 78
Highway 410 Ramps	NB (Off-ramp)	50	48 - 58
	SB (On-ramp)	30	30 - 35
Carpool	NB and SB	40 (assumed)	38 - 48
Tomken Rd	NB and SB	60	58 - 68
Shawson Rd	NB and SB	50 (assumed)	48 - 58
Tristar Dr	EB and WB	50 (assumed)	48 - 58
Meyerside Dr	EB and WB	50 (assumed)	48 - 58
Westport Cr	EB and WB	50 (assumed)	48 - 58
Britannia Rd	EB and WB	60	58 - 68
Vipond Dr	NB and SB	50 (assumed)	48 - 58
Dixie Rd	NB and SB	70	68 - 78
Highway 401 Off-Ramps	WB	60	58 - 68
	EB	70	68 - 78

Aside from driving speed, other characteristics of driving are also configured, including lane changing behavior, lateral driving characteristics, and car-following behaviors. These characteristics are edited by adjusting the settings of urban motorized travel under the driving behaviors tab. For lane changing, free lane selection is allowed as a default setting, along with two adjustments. First, cooperative lane changing is toggled on, meaning that vehicles in the network will work together when detecting that others are changing lanes to complete maneuvers safely (PTV Group, 2023b). The other change to the model's lane changing is setting a diffusion time of 999999 seconds. The diffusion time is the maximum amount of time a vehicle can remain stopped before they are removed from the network (PTV Group, 2023b), so to replicate a realistic scenario, a diffusion time greater than the entire simulation time is specified so no vehicle will ever diffuse from the network. In reviewing the car-following model in this simulation, Wiedemann 74 is assumed, as this car-following model is recommended for urban arterial driving behaviors (PTV Group, 2023b). However, due to the differences in car-following behaviors between different types of vehicles, this vehicle behavior is revisited in detail during the model calibration phase of this project. For lateral driving patterns, the default settings are used, but with two additional parameters. The "observe adjacent lanes" and "consider next turn" settings are both turned on. This means that while driving within their own lane, a vehicle will account for the position and lateral orientation of other vehicles in adjacent lanes to maintain the minimum lateral distance, while also positioning themselves for their next turn (PTV Group, 2023b).

5.2.5 – Pedestrian Volumes

Another key factor pertaining to volumes in the network relates to pedestrians. From the data provided by the Region of Peel and the City of Mississauga, the PM Peak number of pedestrians crossing each leg of the intersection is provided, but no information is provided on their crossing direction. Therefore, it is assumed that half of each provided volume is travelling in each direction. For example, of the twenty-four pedestrians expected to cross the east leg of the Courtneypark Dr-Shawson Dr intersection, it is assumed that twelve are walking northbound and twelve are walking southbound. In Vissim, pedestrian inputs are used to generate these road users, placing these sources within the roadside pedestrian areas. Pedestrian routes are then plotted to reflect the number of pedestrians walking in each direction. It is important to note that these pedestrian volumes are not adjusted to 2022 values, as an AGR between 0.5% and 1% for the pedestrian volumes will generate decimal values of growth, which are simply rounded down to the same value. This process is repeated for all known intersections.

For unknown intersections, all pedestrian counts are filled in based on the pedestrian volumes and directions at adjacent intersections. For instance, for the five pedestrians heading westbound along the northern leg of the Dixie Rd-Britannia Rd intersection, these five pedestrians are also assumed to be walking westbound along the northern leg of the Britannia Rd-Vipond Dr intersection. In the eastbound direction, a pedestrian count is taken from the volume crossing the northern leg of the Britannia Rd-Shawson Dr intersection in the eastbound direction, which is one pedestrian.

5.3 – Model Calibration

In calibrating the base scenario of the Vissim model, multiple steps are taken to ensure that vehicle behavior in the model accurately reflects their real-world behavior, as already discussed in *Section 5.2.4*. However, further calibration is still required beyond the default driver behavior, including defining simulation parameters for the model. Furthermore, given the importance of freight vehicles in this thesis, further calibration of behavioral parameters specific to freight vehicles is necessary. Lastly, a goodness-of-fit test is also performed on the Vissim model to determine how well the real-world traffic conditions are represented in the model.

5.3.1 – Simulation Parameters

In preparing to run the Vissim simulation, parameters must be set so that the simulation behaves as expected. In this thesis, the PM peak hour is selected as the study period. Therefore, all the data input into simulation, from vehicle inputs to traffic signals, should be taking place over a 1-hour timeframe. However, in order to ensure the simulation behavior is accurate, an additional time step is taken to ensure that vehicles have adequately penetrated the network before measurements are recorded. Thus, an additional 900 seconds is added as a warm-up to the 1-hour period, for a total simulation period of 4500 seconds. Data collection within the model is only recorded after this 900-second warm-up period.

5.3.2 – Freight Vehicle Behavior Calibration

Not all vehicle types exhibit the same driving behavior. In particular, freight vehicles often exhibit different driving characteristics from passenger cars due to different engine powers, vehicle weights, acceleration and braking capabilities, and turning characteristics. Since this thesis is heavily focused on freight vehicles, it is crucial that their behavior in the model is reflective of real freight vehicles and not identical to passenger vehicles. In terms of acceleration and deceleration capabilities, Vissim automatically provides default acceleration and deceleration functions tailored to each unique vehicle class. These acceleration functions are relied upon in this thesis to ensure HGVs, including LCVs, accelerate and decelerate as a realistic freight vehicle.

Another adjustment to the acceleration capabilities specific to HGV vehicles is made using the calculated power-weight ratio in Vissim. Within Vissim, an engine power distribution, in kilowatts (kW), and a weight distribution, in kilograms, can each be defined and assigned to the HGV class. Then in generating an HGV at a vehicle input during simulations, a power-weight ratio in kilowatts per tonne (kW/t) is calculated and assigned to the vehicle by randomly selecting an engine power and vehicle weight from the defined power and weight distributions (PTV Group, 2023b). In calculating these power-weight ratios, Vissim also defines a lower limit of 7 kW/t and an upper limit of 30 kW/t to ensure that extreme power-weight ratios are avoided (PTV Group, 2023b). Therefore, to create realistic acceleration and deceleration conditions for HGVs, appropriate vehicle weight and engine power distributions must each be defined.

Beginning with the weight distribution, Vissim defines a default HGV weight distribution of 2800 kg – 40000 kg (PTV Group, 2023b). However, this distribution does not accurately reflect freight vehicle weights in a North American context. A thesis by Chatterjee (2008) adapted a weight distribution for Vissim using reports from the National Cooperative Highway Research Program (NCHRP) and the FHWA, finding an average weight distribution for HGVs of 9080 kg – 36320 kg. Furthermore, trucks in Ontario, including LCVs, are permitted to have total weights of up to 63500 kg (Province of Ontario, 2022a; Province of Ontario, 2022b). Therefore, a weight distribution of 9080 kg – 63500 kg is applied to the Vissim model, as shown in the *Figure 27*.

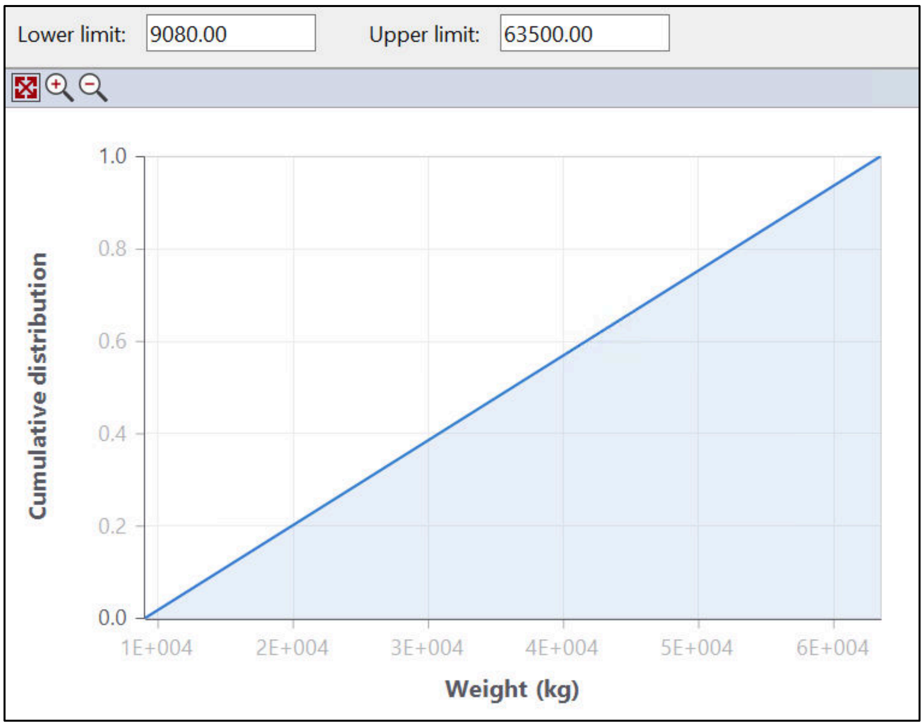

Figure 27: Heavy Goods Vehicle Vissim Weight Distribution

A similar approach is taken with respect to the power distribution. Chatterjee (2008) adapted the weight-to-power ratios from the same NCHRP report, which determined the average freight engine power for the states of Colorado, California, and Pennsylvania. Using the 5th and 95th percentile engine power values for each state, a power distribution is calculated ranging from 198 kW (265 hp) – 517 kW (693 hp) (Chatterjee, 2008). Although, considering the NCHRP report used in this study is dated from 1997, it is important to account for advancements in technology and the accompanying increases in engine power. While no updated version of this study reviewing truck engine powers could be found, this is instead accomplished by reviewing the engine powers available on modern truck models from six prominent truck brands in Ontario: International (2024), Mack (2024), Peterbilt (2024), Kenworth (2024), Volvo (2024), and Freightliner (2024).

Table 31 in *Appendix B.3* summarizes each brand's highway truck model with their available engine models and power. It was found that almost all modern truck models lie within the range developed by Chatterjee (2008), with the only exception being Freightliner's eCascadia with an engine power of 145 kW (195 hp). Therefore, Chatterjee's power range of 198 kW – 517 kW is used in the model, as shown in *Figure 28*. It is important to note that for LCVs specifically, the Ontario LCV program requires vehicles to have an engine power of at least 425 hp (317 kW) (Province of Ontario, 2022b). However, when considering truck driving behavior in this thesis, it is assumed that the power distribution for LCVs is the same as conventional trucks. This is because of a lack of specific information on the vehicle dynamics of LCVs, so driver behavior is assumed to mimic conventional trucks. Both the developed power and weight distributions are assigned to HGVs by editing their vehicle type characteristics in Vissim.

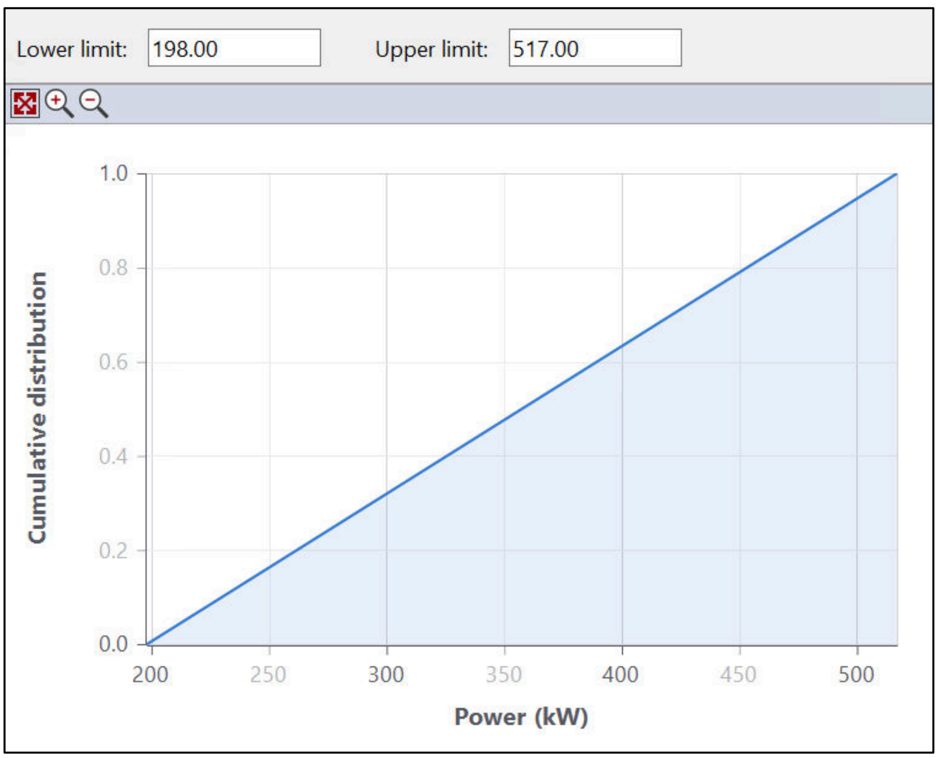

Figure 28: Heavy Goods Vehicle Power Distribution in Vissim

Another key driving behavior that is adjusted for freight vehicles is the car-following behavior of trucks compared to cars. Many studies have observed that car-following characteristics differ between vehicle types when travelling within heterogeneous traffic flows containing cars and trucks (Durrani et al., 2014; Chowdhury et al., 2023). In a study investigating the differences between car and truck movements on freeways, Durrani et al. (2014) found that cars maintain both a higher time headway and spacing when following a truck compared to when they are following another car. Therefore, to reflect this in the microsimulation, the car-following behavior must be adjusted in the model. Within Vissim, the two possible car-following models are available: Wiedemann 74 (W74), which is more appropriate for urban arterial driving behavior, and Wiedemann 99 (W99), which is more appropriate for freeway driving behavior (PTV Group, 2023b). As a result, the W74 car-following model is selected for this project. To implement this car-following model, the W74 parameters developed by Chowdhury et al.'s (2023) study are used, where car-following parameters are developed for cars-following-cars, cars-following-trucks, trucks-following-cars, and trucks-following trucks. *Table 7* shows the W74 parameters used in this thesis adapted from Chowdhury et al.'s study. While Vissim only allows users to define car-following parameters based on the preceding vehicle class, parameters are defined for both the leading and following vehicle by defining separate driving behaviors in Vissim and assigning them to vehicle classes within the link behavior types.

Table 7: Vissim Car-Following Parameters

Input Parameter	Vissim Default	Passenger Vehicle	Freight Vehicle
Average Standstill Distance When Following a Car	2 m	2.87 m	3.77 m
Average Standstill Distance When Following a Truck	2 m	4.07 m	3.37 m
Maximum Deceleration for Cooperative Braking	- 3 m/s^2	- 3 m/s^2	- 1.62 m/s^2

One final freight vehicle driving behavior that must be incorporated into the model occurs at intersections with double left-turn lanes. As previously discussed, it is assumed that at double left-turn lanes, freight vehicles are only permitted to use the rightmost left-turn lane (Transportation Association of Canada, 2017), as this allows these vehicles to perform an oversteering maneuver and avoid conflicts with vehicles travelling in the simultaneous turning lane. However, by default, freight vehicles cannot perform an oversteering maneuver in Vissim, instead completing conventional turns where the tractor remains within the turning lane while the trailer veers into the adjacent left-turning lane, leading to conflicts with vehicles in the simultaneous left-turn lane. Therefore, an adjustment must be made to mimic oversteering maneuvers as effectively as possible. To do this, an additional link driving behavior is defined for freight vehicles, where the "desired lateral position" for freight vehicles in a double left-turn lane is the right side of the lane. Additionally, the "observe adjacent lane" and "consider next turn" lateral driving behaviors are turned off. This link driving behavior is assigned to all double left-turn lanes to force freight vehicles to perform these wide turns, preventing the trailer from passing into the inner left-turn lane.

At the same time, the Vissim network also contains three instances of double right-turn lanes, at each highway interchange, where two lanes of traffic are simultaneously turning right. In this case, the same lateral driving behavior adjustment is made, but instead changing the freight vehicles' "desired lateral position" to the left side of the turning lane to force an oversteering maneuver in the leftmost right-turn lane.

5.3.3 – Goodness-of-Fit Assessment

The final step of the model calibration is to perform a goodness-of-fit test to assess how well the developed model represents the observed traffic volumes. As recommended by the Wisconsin DOT (2019c) and the FHWA (2004), a traffic volume calibration is performed using the Geoffrey E. Havers (GEH) statistic. The GEH statistic test compares local observed traffic volumes in the field with traffic volumes generated by the microsimulation model. The GEH metric is calculated using the following equation,

Equation 5: $$GEH = \sqrt{\frac{(E-V)^2}{(E+V)/2}}$$

where E represents the modelled traffic volume of a turning movement in the network and V represents the observed PM peak hour traffic volumes at each intersection that have been adjusted to 2022 values for each specific turning movement (FHWA, 2004; Wisconsin DOT, 2019c), both measured in vehicles per hour. This GEH statistic is calculated for every turning movement at each intersection in the network to provide a measure of whether the model appropriately reflects field observations. In general, a GEH value of less than 5 indicates a good fit for a turning movement, a GEH value between 5 and 10 warrants further investigation to determine its goodness-of-fit, and a GEH value greater than 10 indicates the model is not a good fit for the corresponding turning movement (FHWA, 2004; Soo et al., 2015; Wisconsin DOT, 2019c). Additionally, for the model to be considered a good fit for the network, a threshold is set where at least 85% of turning movements across the entire network should have a GEH value less than 5 (FHWA, 2004; Soo et al., 2015; Wisconsin DOT, 2019c). Three 1-hour simulations, excluding the 900 second warm-up periods, are run in the model and the collected vehicle volume results are averaged across the three runs. A summary of the GEH results with respect to the count of turning movements within each GEH threshold interval is displayed in *Table 8*. The full results of the GEH calculations can be found in *Table 32* in *Appendix B.4*, which includes the GEH value calculated for every turning movement at each intersection.

Table 8: GEH Results Summary

GEH Threshold	Count	Percentage
GEH < 5	115	95.1 %
5 < GEH < 10	5	4.1 %
GEH > 10	1	0.8 %

In completing this goodness-of-fit test, it was found that 95.1% (115 out of 121) of the turning movements in the network fall within the GEH < 5 threshold, indicating that the Vissim model developed here is an excellent fit for representing the observed field conditions. At the same time, of the remaining six turning movements, only one movement provides a GEH value greater than 10, indicating a poor fit for that motion. Upon closer investigation, this GEH value of 11.09 occurs at the Courtneypark Dr-Tomken Rd intersection, in the northbound through direction along Tomken Rd. In reviewing the simulation, this occurs due to extreme congestion occurring on the south side of the Courtneypark Dr-Tomken Rd intersection. With the existing PM peak hour traffic volume and signalization at this intersection, the traffic becomes congested on the entire link to the point that not all input vehicles are able to pass through the intersection before the end of the 3600-second simulation period. As a result, the number of vehicles recorded going through the intersection in the northbound direction is underestimated compared to observed field conditions. Considering this northbound through movement is not a key part of the study's LCV route, as the primary route runs in the east-west direction and this motion has no impact on any other movement in the network, this individual GEH value is disregarded.

5.4 – LCV Application

To assess the LCV's effects on last-mile road safety, a sensitivity analysis is performed where these vehicles are added into the simulated road network. The following sections will outline the steps taken to perform this sensitivity analysis, including the development of an LCV model vehicle in Vissim, the process of adding LCVs into the network, the calibration of this LCV model, and the development of scenarios that will be assessed in the sensitivity analysis.

5.4.1 – Vehicle Model Development

To perform the sensitivity analysis, the first step is to develop a vehicle model for an LCV within Vissim. Vissim's vehicle library does not include any LCV models, so segments from existing vehicle models are repurposed to create an LCV model for this thesis. To remain consistent with the swept-path analysis in AutoTurn, a turnpike double is replicated in Vissim, which includes a tractor pulling two freight trailers attached together with a connector. The lengths of the trailers and joint positions are manually edited to ensure that the total length of the turnpike double is 38 meters to be consistent with the AutoTurn vehicle model. *Figure 29* shows the final model of the turnpike double used in Vissim, alongside the dimensions of each vehicle component in *Table 9*. It should be noted that the trailer components of this model contain triple axles, which differs from the model selected in AutoTurn. However, this triple axle is only a cosmetic component of the Vissim 3D model and does not have any impact on the driving mechanics of the LCV.

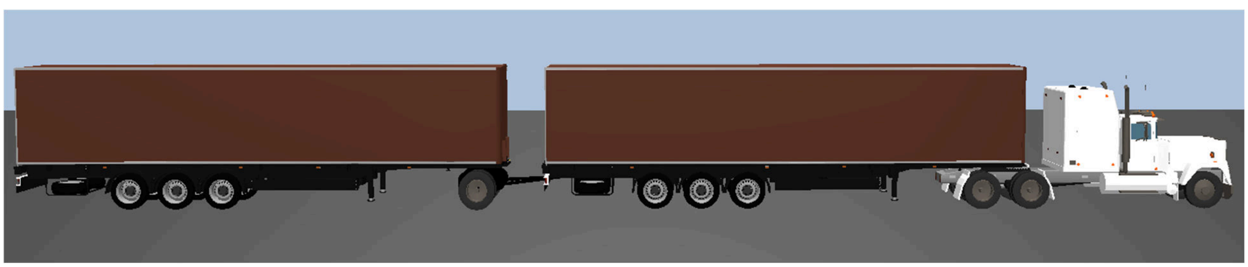

Figure 29: Turnpike Double Model in Vissim

Table 9: Turnpike Double Vissim Model Dimensions

Turnpike Double Model Segment	Length (m)	Width (m)
US AASHTO WB-65 Tractor	8.174	2.543
Semi-Trailer Box 3-axles	15.138	2.300
US AASHTO WB-67D Trailer Connector	4.000	2.616
Semi-Trailer Box 3-axles	15.138	2.300

5.4.2 – Implementation and Calibration

Once the LCV model is formulated, the next step of the sensitivity analysis is to implement the LCVs within the model and calibrate their behavior. First, to implement LCVs in Vissim, LCVs are assigned as their own vehicle class. The characteristics of this vehicle class are then redefined following the same process previously used for passenger vehicles and conventional freight vehicles. Starting with the weight distribution, LCVs in Ontario are subject to the same weight restrictions as conventional freight vehicles (Province of Ontario, 2022a; Province of Ontario, 2022b), so the weight distribution of 9080 kg – 68500 kg previously used for conventional freight trucks is applied again for LCVs. For the power distribution, Chatterjee's (2008) conventional power distribution of 198 kW – 517 kW is also assumed for the LCV vehicle class. It should be noted that the Province of Ontario's (2022b) LCV guidelines require that LCVs have engine powers of at least 425 hp (313 kW). However, when applying this minimum value to the LCV engine power range, this results in the LCVs having unrealistic acceleration rates compared to conventional freight vehicles. As a result, the conventional freight power distribution is also adopted for LCVs.

The LCV's characteristics in terms of how it interacts with the road network components is also redefined. In terms of the routing decisions at each intersection, the LCV routing decisions replicate the routing decisions previously used by the HGV class. At the same time, the freight-free lanes that previously prohibited HGVs are expanded in scope to also exclude LCVs. Additionally, all previously defined driving behaviors developed and calibrated for freight vehicles in the base model are also adopted by the LCV vehicle class. This includes adopting the previously defined car-following model characteristics for cars-following-cars, trucks-following-cars, trucks-following-trucks, and cars-following-trucks from Chowdhury et al.'s (2023) study. Lastly, the lateral driving behaviors previously defined for conventional freight vehicles to force them into oversteering maneuvers at double left turns and double right turns are also assigned to the LCV vehicle class.

The final step to calibrate the behavior of LCVs is to validate the turning paths of the LCVs within Vissim. This is achieved by superimposing the Vissim simulation onto a to-scale AutoTurn drawing of the turnpike double's swept path to assess how well the Vissim turning path replicates the path previously analyzed in AutoTurn. For this exercise, an AutoTurn drawing of the left turn at Shawson Drive and Westport Crescent South is added into the Vissim simulation and adjusted to match the scale of the simulation model. Links are then drawn over the photo and vehicle inputs of LCVs added to simulate an LCV left turn over the AutoCAD drawing.

As can be seen in the sequence of five images in *Figure 30*, a burgundy LCV in Vissim completed the left turn overlapped with the AutoTurn swept path at Shawson Drive and Westport Crescent South. In using the calibrated driving characteristics described earlier in this section, the turning path of the LCV follows the swept path established in AutoTurn moderately well. While the Vissim model vehicle may not perfectly trace the entire swept path, this model still does well to trace the innermost side of the swept path envelope, which can be considered more important than the outside of the turning envelope due to the tendency of the trailers to drag as the tractor makes its turn. It should be noted that an adjustment to the lateral turning characteristics was attempted to force the Vissim vehicle to oversteer at this intersection. However, this oversteering resulted in the LCV model's turning characteristics deviating significantly from the swept path envelope in AutoTurn. Therefore, the calibrated driving characteristics previously established are acceptable for moving forward with the LCV simulation modelling.

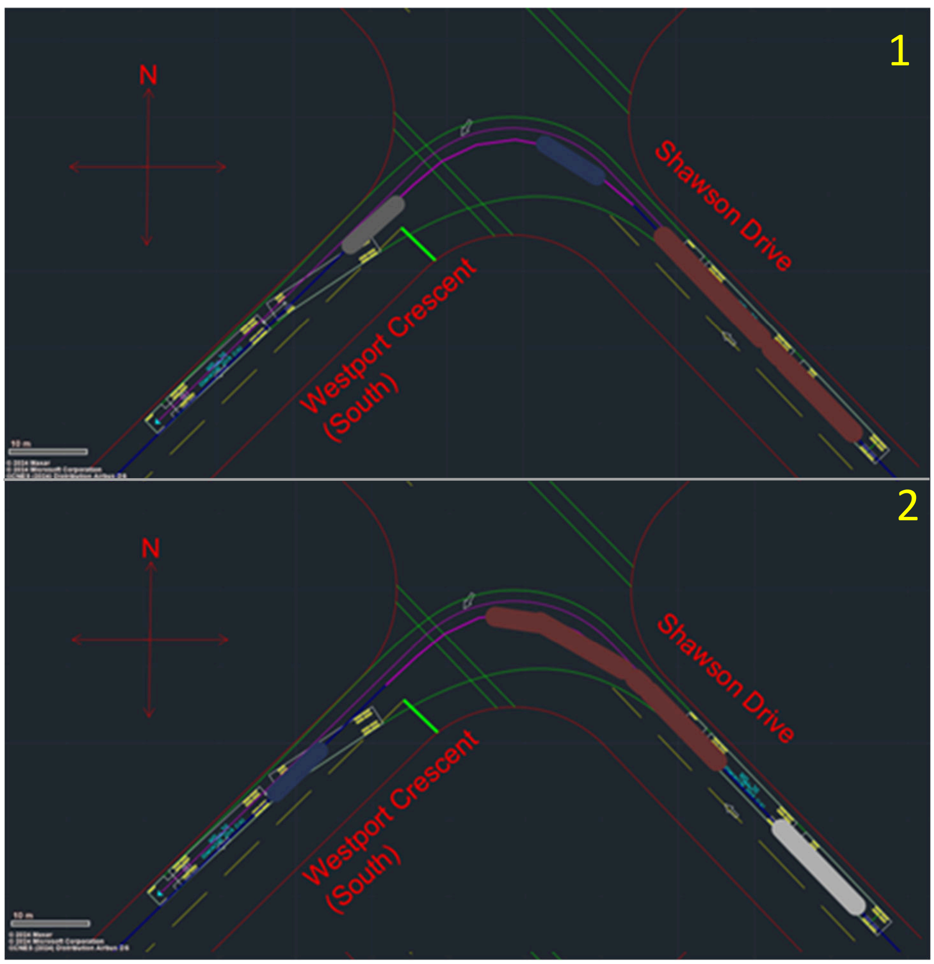

Figure 30: Vissim Simulation Superimposed Over AutoTurn

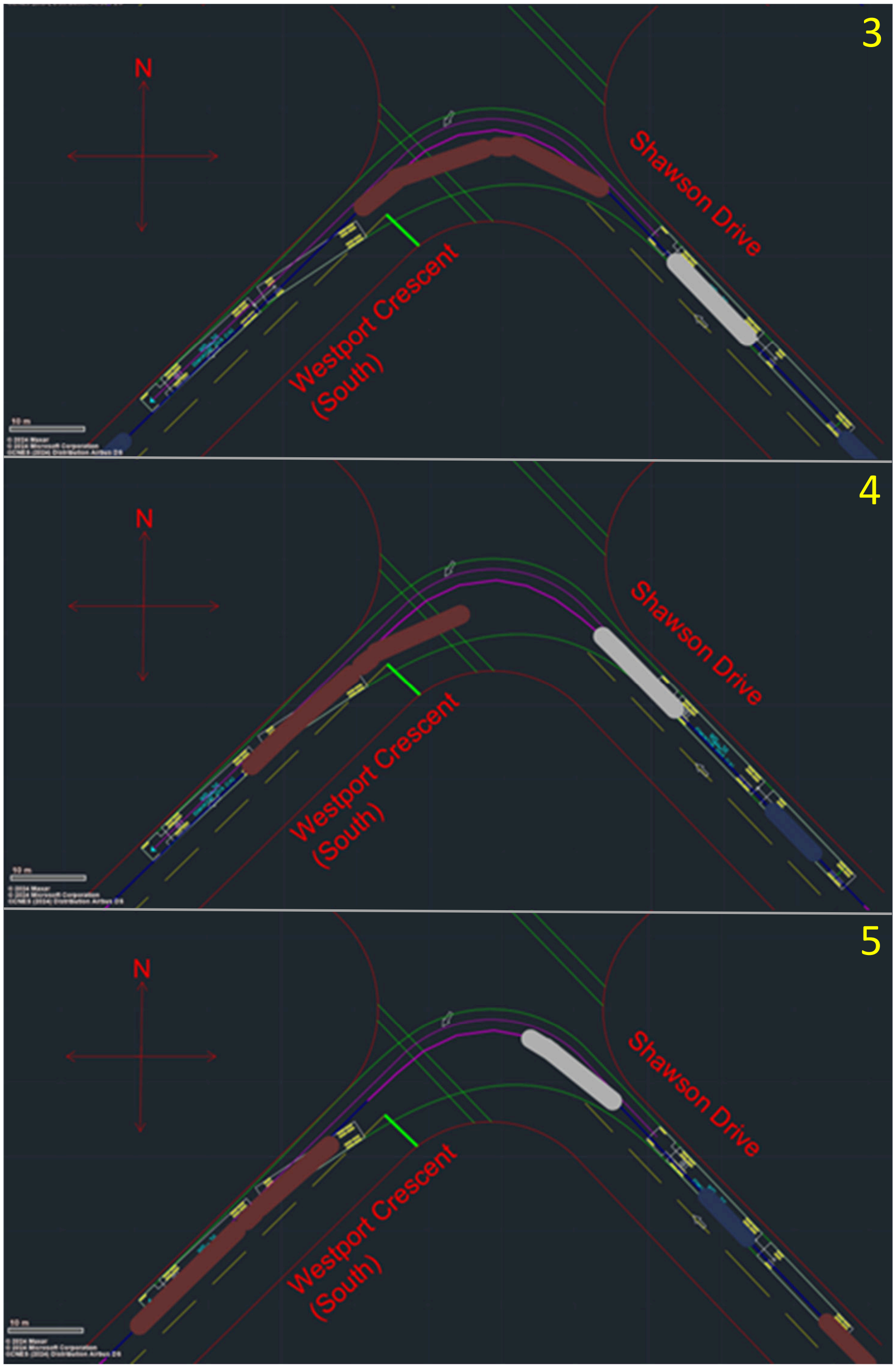

Figure 30: Vissim Simulation Superimposed Over AutoTurn (Continued)

5.4.3 – Sensitivity Analysis of LCV Penetration

To analyze the safety implications of LCVs in last-mile areas during left turns, a sensitivity analysis is performed with respect to the percentage of LCVs replacing conventional freight vehicles, otherwise known as the LCV penetration level. In total, five scenarios are devised for different LCV penetration levels: a base scenario with 0% LCVs, 25% LCVs, 50% LCVs, 75% LCVs and 100% LCVs. For example, in the 25% LCV level, 25% of the conventional freight vehicles from the base scenario will be replaced with turnpike doubles, while the remaining 75% will still be composed of conventional freight vehicles. It should be noted that in reality, LCV penetration levels of 75% or 100% may not be practical, but these scenarios are still included in the study to provide additional context for the safety analysis of LCVs. Another fact to consider is the ratio at which LCVs replace normal freight vehicles. Due to their size and cargo capacity, a direct replacement of one freight vehicle with one LCV does not seem logical. Therefore, it is assumed that one LCV effectively replaces two conventional freight vehicles. For example, in the 25% scenario, while 25% of conventional freight vehicles are removed from the network, only half of the number of trucks removed are replaced by an LCV.

Taking into account the different penetration levels and the 2:1 LCV replacement ratio, both the vehicle compositions and vehicle inputs used in the Vissim model are recalculated. Using the vehicle proportions and inputs originally calibrated from the City of Mississauga's and Region of Peel's data collections, the new vehicle proportions are calculated by replacing 25% of the freight vehicles at each penetration level and accounting for the 2:1 ratio. Many of the new vehicle compositions include decimal values, which is acceptable given that these are relative proportions. The vehicle totals are also calculated accordingly for each level. However, in applying these vehicle totals in Vissim, these vehicle input values will need to be rounded up to the nearest integer. The final values from these calculations for vehicle compositions and totals at each penetration level can be found in *Appendix B.5*. To input these scenarios in Vissim, four copies of the 0% base scenario are created. In each new scenario, the corresponding vehicle compositions and vehicle inputs are applied throughout the network.

The final step of the sensitivity analysis is to generate trajectory files for each scenario, achieved by selecting the SSAM option under the evaluation configuration tab in Vissim. Each of the five LCV level simulations is run three times, generating three trajectory files for each penetration level. These trajectory files are used in the next step of the methodology to evaluate safety conflicts in the network.

5.5 – Surrogate Safety Assessment Model

The FHWA's SSAM is used to assess safety conflicts that occur during freight left turns within the developed Vissim simulation. This is achieved by analyzing the trajectory files generated during each of the five LCV penetration level scenarios. Due to the fact that this analysis is limited to traffic conflicts involving freight vehicles during left turns, only conflicts involving these vehicle types occurring on left-turning links are included in the analysis. This is achieved by first sorting through all the links in Vissim and isolating the ones that are involved in left turns at each intersection. In this analysis of the left-turning links, there may be some specific links that inadvertently include additional conflicts that are not actually taking place within the left turn, such as links that are involved in the left turns but also extend far beyond the area of the left turn. As a result, these links that introduce additional conflicts are classified into three categories based on their potential to add additional conflicts that do not occur during left turns. Amongst these categories, Category 1 links have the highest potential to add additional conflicts and Category 3 links marginally introduce extra conflicts. Using SSAM's link filter, the analysis is run for each of the five scenarios where all non-left-turning links are removed from the analysis. Additionally, the analysis is run three more times by further removing the Category 1, 2, and 3 links.

Furthermore, a filter is also applied to remove conflicts that do not involve freight vehicles. While SSAM's filter function does not include an option to filter the results based on vehicle type, it does provide information on the lengths of the two vehicles involved in each conflict. Since the lengths of the three freight vehicle types are known from the model vehicle lengths in Vissim, with LCVs at 38 meters, single-unit box trucks at 10.26 meters, and conventional tractor-trailers at 16.50 meters, all conflicts not involving any of these three vehicle lengths are disregarded in the analysis.

In reviewing the number of left-turning freight conflicts for each penetration level after removing links classified in different categories, the results of each LCV penetration level are displayed in the *Table 10*. As can be seen in this table, the removal of the Category 1 links results in a considerable decrease in the number of freight collisions, while further removing the Category 2 and Category 3 links yields very little to no change in the number of freight conflicts. This indicates that the Category 1 links add in a considerable number of conflicts that are not actually occurring during left-turning maneuvers. Therefore, all SSAM analyses moving forward exclude Category 1 Vissim links, with the remaining freight conflicts after removing Category 1 links marked in bold font in the following table.

Table 10: Number of Freight Conflicts by Category

LCV Penetration Level	# of Freight Conflicts			
	All Left-Turning Links	**Removing Category 1**	Removing Category 1 and 2	Removing Category 1, 2, and 3
0%	250	**236**	236	236
25%	599	**584**	584	584
50%	895	**847**	845	845
75%	1219	**1182**	1176	1175
100%	1351	**1290**	1284	1281

With these appropriate links and vehicle types selected, SSAM provides information on the severity, frequency, type, and location of freight conflicts in the network. In terms of severity, the surrogate measures of safety used in this analysis are time-to-collision (TTC) and post-encroachment time (PET). Based on the default thresholds provided by SSAM, conflicts involving a TTC between 0 and 1.5 seconds or a PET between 0 and 5 seconds are identified, where a lower PET or TTC indicates a more severe conflict. In each conflict, the type of conflict is identified, including whether it is a rear end conflict, lane changing conflict, or a crossing conflict. The location of each freight conflict is also conveyed by providing the links in which the conflicts occurred.

In this step of this project, an aggregate-level analysis is first conducted. In this aggregate analysis, the freight conflicts across all left turns at each of the thirteen intersections are observed together to identify trends in the conflict frequency, TTC, PET, and conflict type as the LCV penetration level is increased. Once the aggregate-level analysis of the network is finished, an intersection-level analysis is completed, where the aggregate trends are compared with the conflict trends occurring at each specific intersection. Amongst the thirteen intersections in this network, a closer analysis is conducted for the highest-conflict intersections, investigating whether the trends at these intersections are consistent with the findings of the aggregate-level analysis. A detailed examination of these intersections will also be conducted to postulate why these trends in conflict type, PET, or TTC are occurring at each intersection for each LCV penetration level by reviewing the specific links and vehicles involved in these conflicts.

Chapter 6. Swept Path Analysis Results

As discussed in the swept path analysis methodology section, results are generated for both the existing road infrastructure, as well as for the sensitivity analyses pertaining to lane widths and setback distances. The following sections will summarize the findings of these analyses and will include a discussion on the typical problems LCVs encountered during these assessments.

6.1 – Results for Current Intersection Geometry

In total, twenty-two swept path simulations are completed for the existing intersection geometry. This includes running the analysis for the six left turns along the proposed route, assessing Scenarios A through D for each intersection. The only exception to this is the Courtneypark Dr – Highway 410 intersection, where only one receiving lane is available. As a result, only two scenarios are assessed for this intersection. The results of these analyses are shown in the table below. This table summarizes the passing or failing grade received for each scenario. For passing scenarios, the maximum speed determined is noted. For failing scenarios, commentary is made on the types of conflicts that are occurring. Images of the full AutoTurn swept paths can be found in *Appendix A.2*.

Table 11: Swept Path Analysis Results for Existing Intersections

Int. #	Intersection	Scenario A	Scenario B	Scenario C	Scenario D
1	Highway 401 EB Off-Ramp onto Dixie Road	Fail: Conflicts with adjacent lanes on both departing and destination roads	Pass: $Speed_{MAX}$ = 19 km/hr	Fail: Veers into right lane, but still interferes with adjacent lanes on both roads	Pass: $Speed_{MAX}$ = 19 km/hr
4	Dixie Road onto Britannia Road	Fail: Conflicts with medians and other lanes of traffic	Pass: $Speed_{MAX}$ = 19 km/hr	Fail: Conflicts with medians and other lanes of traffic	Pass: $Speed_{MAX}$ = 19 km/hr
7	Shawson Drive onto Westport Crescent South	Fail: Trailer infringes into oncoming lane of receiving road despite extra wide lanes	Pass: $Speed_{MAX}$ = 19 km/hr	Fail: Trailer infringes into oncoming lane of receiving road despite extra wide lanes	Pass: $Speed_{MAX}$ = 21 km/hr
8	Westport Cr North onto	Fail: Conflicts with oncoming lanes and	Pass: $Speed_{MAX}$ = 19 km/hr	Fail: Conflicts with oncoming	Pass: $Speed_{MAX}$ = 21 km/hr

Int. #	Intersection	Scenario A	Scenario B	Scenario C	Scenario D
	Shawson Drive	curb of receiving road		lanes and curb of receiving road	
11	Shawson Drive onto Courtneypark Drive East	Fail: Conflicts with oncoming lanes on departing road and median on destination road	Fail: Conflicts with adjacent driving lane, curbs, and medians	Fail: Conflicts with oncoming lanes, adjacent lanes, and medians	Fail: Conflicts with curb and median
13	Courtneypark Drive East onto Highway 410 NB On-Ramp	Pass: $Speed_{MAX}$ = 21 km/hr	Not applicable since there is only one receiving lane	Pass: $Speed_{MAX}$ = 20 km/hr	Not applicable since there is only one receiving lane

One trend that stands out from the swept path analysis pertains to the results with Scenario A, where the LCV is restricted to one lane on both the departing and destination roads. In this scenario, the WB-36 received a failing grade for five out of six intersections, as the left turns almost always lead to a combination of conflicts with oncoming lanes of traffic, adjacent lanes, raised medians, and roadside curbs. This occurs across both signalized and unsignalized roads and amongst arterial, collector, and local roads, even at the low turning speed of 5 km/hr. Amongst the most common problems in Scenario A is conflicts with oncoming lanes. This occurs when the tractor of the turnpike double lands within the correct destination lane, but as the vehicle moves forward in this lane, the two trailers drag through the adjacent oncoming lane closest to the center of the road. Therefore, these findings indicate that existing road geometry is not well equipped for the large body envelopes of LCVs when restricted to one lane on both roads, regardless of the intersection type or road class.

The results for Scenarios B, C, and D, where additional lanes are used, highlight that many urban and suburban roads are designed with the expectation that freight vehicles may have permission to use the additional lanes available to them, in which case LCVs must use these lanes with extra caution. In comparing the results for Scenarios B and C, another trend that stands out is that Scenario B, where an additional lane is permitted for use on the destination road, results in more passing grades (4/5 intersections) than Scenario C (1/6 intersections), where an additional lane is allowed on the departing road. It can be concluded that allowing an additional lane for use on the destination road in Scenario B holds more value for safety than an additional lane on the departing road in Scenario C. In reviewing the swept paths in Scenario B, it is noted that the additional destination lane helps to significantly mitigate the issue of the trailers dragging into the oncoming lane adjacent to the destination lane. Instead, the LCV makes use of the additional destination lane by positioning its tractor in the additional lane when making the left turn. Consequently, as the tractor continues in the extra lane on the destination road, the semi trailers are able to pass into the second destination lane without infringing on the oncoming traffic lanes. On the other hand, the additional departing lane in Scenario C is not as advantageous for LCVs, as the swept path envelope still results in the trailers passing into oncoming traffic.

In Scenario D, where two lanes are available on both roads, this scenario results in the joint most passing grades with Scenario B (4/5 intersections), which is not surprising given the additional space available during the turns. Overall, in the scenarios resulting in a passing grade, the maximum speed ranged from 19 km/hr to 21 km/hr, representing a major increase from 5 km/hr. At high speeds, freight vehicles require more space to maneuver comfortably, so this major speed increase shows that the additional space provided by the extra lanes allows much more leeway in the LCV's ability to make left turns in Scenario D compared to Scenarios A through C.

In reviewing the failing scenarios, another pattern that stands out is at the Shawson Drive-Courtneypark Drive intersection, where the turnpike double cannot complete the turn in any of the four scenarios. Due to the narrow right-of-way between the road edge and the center median, the swept path envelope of the WB-36 prevents the vehicle from completing the turn without conflict. In attempting a conventional left turn, the trailers of the LCV will conflict with the center median and oncoming lanes on the departing road. If a wider turn is attempted, the front of the tractor conflicts with the curb at the road edge and the trailer still collides with the center median. Above all, these conflicts are a major concern for future LCV use, as it shows that even with all lanes available to LCVs, some intersections are still unable to accommodate these large freight vehicles.

One final area of interest in these results relates to the left turns involving the Westport Crescent North and South intersections. These left turns take place on local roads, with one lane in both directions, so the additional lane permitted for use is the oncoming lane. Due to these local roads being in an industrial area, these lane widths are exceptionally wide to accommodate freight vehicles, with lane widths of 6 meters on Shawson Drive and Westport Crescent. Additionally, some of this width is used as on-street parking along Westport Crescent. In these local intersections, Scenarios A and C result in failing grades, with the trailers infringing into the oncoming lane on the destination roads. However, the argument could be made that these traffic lanes are so wide that the trailer partially infringing into the oncoming lane may not be as problematic since other vehicles in the oncoming lane will have ample width remaining to continue safely. However, with some vehicles parked on the road along Westport Crescent, the concern of infringing into oncoming lanes remains high, as less space in the receiving lane due to a parked car leads to more space being occupied in the destination road's oncoming lane as the LCV's trailer drags through these lanes.

In conclusion, the results of the swept path analysis on the existing roads are mixed. As intersections are currently designed, LCVs are not able to use these intersections safely if restricted to one lane on both the origin and destination roads. However, when permitted to use an additional lane on the destination road, the intersection geometry can accommodate these LCVs, as the turnpike double is able to complete the turn without conflict in most cases. Moving forward, sensitivity analyses are performed to assess available the buffer space LCVs have to turn at each intersection.

6.2 – Sensitivity Analysis Results

In assessing the buffer space for LCVs at each of the six intersections, sensitivity analyses are completed for both the departing and destination widths of the lanes at each intersection and the setback distance of oncoming lanes. The following sections summarize the results of these two procedures.

6.2.1 – Lane Width Sensitivity Analysis

In completing the sensitivity analysis of the lane widths, adjustments are made to the departing road and destination road of each intersection. The analyses of the departing and destination roads are completed separately. For each sensitivity analysis, the widths of specific lanes are changed until the grade status changes, assuming a turning speed of 5 km/hr. This is the lane width threshold (LWT), at which a passing grade changes to a failing grade or vice versa. The before-and-after lane widths as well as the change in lane width are recorded for each scenario at each intersection. The original widths of Lane W, X, Y, and Z for each road are recorded in *Table 12*.

Table 12: Original Lane Widths in AutoTurn Scenarios

Int. #	Intersection	Original Lane Widths (m)			
		W	X	Y	Z
1	Dixie Road and Highway 401	4	3.8	3.5	4.2
4	Dixie Road and Britannia Road	3.7	3	3.5	5*
7	Shawson Drive and Westport Crescent South	6	6	6	6
8	Shawson Drive and Westport Crescent North	6	6	6	6
11	Shawson Drive and Courtneypark Drive	3	3.3	3.4	3.8
13	Courtneypark Drive and Highway 410	3	3.5**	8.6*	-

* Includes shoulder width
** Lane is a painted median

Beginning with the sensitivity analysis of the departing road lane widths, the results of each scenario can be found in *Table 13*. Note that the lanes denoted W, X, Y, and Z are consistent with those shown previously in *Figures 19* and *20*. In the sensitivity analysis of the departing lane width in *Table 13*, Lane W is the primary lane adjusted.

Table 13: Lane Width Sensitivity Analysis for Departing Lanes

<table>
<tr><th>Int #</th><th>Intersection</th><th>Scenario</th><th>Base Grade</th><th>Original Lane W Widths (m)</th><th>Threshold Lane Width (m)</th><th>Change in Width (m)</th></tr>
<tr><td rowspan="8">1</td><td rowspan="8">Dixie Road and Highway 401</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>4</td><td>Does Not Exist (DNE)</td><td>-</td></tr>
<tr><td colspan="3">Receiving lane not wide enough to accommodate LCV, regardless of departing lane; never changes to pass.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>4</td><td>3.1</td><td>- 0.9</td></tr>
<tr><td colspan="3">Right-of-way is reduced since the Lane X width remains the same.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>4</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Receiving lane not wide enough to accommodate LCV, regardless of departing lane; never changes to pass.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>4</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Reducing Lane W to the minimum 2.6m still allows for a pass (Right-of-way is reduced since the Lane X width remains the same).</td></tr>
<tr><td rowspan="8">4</td><td rowspan="8">Dixie Road and Britannia Road</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>3.7</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Receiving lane not wide enough to accommodate LCV, regardless of departing lane; never changes to pass.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>3.7</td><td>3.2</td><td>- 0.5</td></tr>
<tr><td colspan="3">Right-of-way is reduced since the Lane X width remains the same.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>3.7</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Receiving lane not wide enough to accommodate LCV, regardless of departing lane; never changes to pass.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>3.7</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Reducing Lane W to 2.6m still allows for a pass (Lane X width is unchanged, right-of-way adjusted).</td></tr>
<tr><td rowspan="2">7</td><td rowspan="2">Shawson Drive and Westport Crescent South</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>6</td><td>9</td><td>+ 3.0</td></tr>
<tr><td colspan="3">Increasing from 6m to 9m, an extreme width, brings the scenario to a pass (in this case, Lane X is the oncoming lane and is unchanged, right-of-way adjusted).</td></tr>
</table>

<table>
<tr><th>Int #</th><th>Intersection</th><th>Scenario</th><th>Base Grade</th><th>Original Lane W Widths (m)</th><th>Threshold Lane Width (m)</th><th>Change in Width (m)</th></tr>
<tr><td rowspan="6"></td><td rowspan="6"></td><td rowspan="2">B</td><td rowspan="2">Pass</td><td>6</td><td>3.8</td><td>- 2.2</td></tr>
<tr><td colspan="3">Right-of-way is adjusted and Lane X is unaffected.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>6</td><td>9</td><td>+ 3.0</td></tr>
<tr><td colspan="3">Identical to Scenario A.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>6</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Reducing Lane W to minimum width 2.6 m still allows for a pass (Right-of-way is reduced and Lane X is unaffected).</td></tr>
<tr><td rowspan="8">8</td><td rowspan="8">Shawson Drive and Westport Crescent North</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>6</td><td>12</td><td>+ 6.0</td></tr>
<tr><td colspan="3">Right-of-way is increased and Lane X is unaffected.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>6</td><td>4.7</td><td>- 1.3</td></tr>
<tr><td colspan="3">Right-of-way is reduced and Lane X is unaffected.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>6</td><td>12</td><td>+ 6.0</td></tr>
<tr><td colspan="3">Same as Scenario A; second departing lane is not used (Right-of-way is increased and Lane X is unaffected).</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>6</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Reducing Lane W to minimum 2.6m still allows for a passing grade (Right-of-way is reduced, Lane X is unaffected).</td></tr>
<tr><td rowspan="8">11</td><td rowspan="8">Shawson Drive and Courtneypark Drive</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>3</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Receiving lane not wide enough to accommodate swept path of LCV, regardless of departing lane width.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Fail</td><td>3</td><td>7.6</td><td>+ 4.6</td></tr>
<tr><td colspan="3">Increasing Lane W to 7.6m, completely taking over second lane, changes scenario to a passing grade (Right-of-way is increased, Lane X completely overtaken).</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>3</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Identical to Scenario A, where destination lane is too narrow.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Fail</td><td>3</td><td>7.6</td><td>+ 4.6</td></tr>
<tr><td colspan="3">Lane W increased to 7.6m wide to change grade to a pass (Lane X width is unaffected and shifted accordingly, total right-of-way increased).</td></tr>
<tr><td rowspan="4">13</td><td rowspan="4">Courtneypark Drive and Highway 410</td><td rowspan="2">A</td><td rowspan="2">Pass</td><td>3</td><td>2.6</td><td>- 0.4</td></tr>
<tr><td colspan="3">Lane W width going below minimum width of 2.6m leads to a fail.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Pass</td><td>3</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Reducing Lane W to minimum 2.6m still allows for a pass since painted median is used.</td></tr>
</table>

Meanwhile, the results of the sensitivity analysis pertaining to changes in the destination lane widths are shown in *Table 14*. In this sensitivity analysis of the destination lane width, Lane Y is the primary lane adjusted.

Table 14: Lane Width Sensitivity Analysis for Destination Lanes

<table>
<tr><th>Int. #</th><th>Intersection</th><th>Scenario</th><th>Base Grade</th><th>Original Lane Y Widths (m)</th><th>Threshold Lane Width (m)</th><th>Change in Width (m)</th></tr>
<tr><td rowspan="8">1</td><td rowspan="8">Dixie Road and Highway 401</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>3.5</td><td>7.1</td><td>+ 3.6</td></tr>
<tr><td colspan="3">Lane Y is increased by taking space from Lane Z, right-of-way is unaffected.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>3.5</td><td>2.9</td><td>- 0.6</td></tr>
<tr><td colspan="3">Reducing the receiving Lane Y, while leaving Lane Z as it is and adjusting the total right-of-way, results in a fail.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>3.5</td><td>7.1</td><td>+ 3.6</td></tr>
<tr><td colspan="3">Lane Y takes up space from Lane Z and the right-of-way is not changed. Note it is the same value as in Scenario A.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>3.5</td><td>2.9</td><td>- 0.6</td></tr>
<tr><td colspan="3">Reducing Lane Y while Lane Z width is unaffected and the right-of-way is adjusted leads to a fail. Consistent with Scenario B.</td></tr>
<tr><td rowspan="8">4</td><td rowspan="8">Dixie Road and Britannia Road</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>3.5</td><td>8.1</td><td>+ 4.6</td></tr>
<tr><td colspan="3">Lane Y increases and overtakes Lane Z (right-of-way is increased) changes to a pass.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>3.5</td><td>3.2</td><td>- 0.3</td></tr>
<tr><td colspan="3">Right-of-way is adjusted, including road edges and pedestrian island.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>3.5</td><td>7.9</td><td>+ 4.4</td></tr>
<tr><td colspan="3">Lane Y overtakes Lane Z and increases right-of-way width, leading to a pass.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>3.5</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Minimum Lane Y width of 2.6 m still allows for a pass.</td></tr>
<tr><td rowspan="8">7</td><td rowspan="8">Shawson Drive and Westport Crescent South</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>6</td><td>7.1</td><td>+ 1.1</td></tr>
<tr><td colspan="3">Right-of-way is adjusted, Lane Z is the oncoming lane and is not affected.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>6</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Minimum width of 2.6m still allows for a pass. Note that a significant amount of the oncoming Lane Z is used, which poses a safety risk.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>6</td><td>7.1</td><td>+ 1.1</td></tr>
<tr><td colspan="3">Identical to Scenario A since Lane X is not used.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>6</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Same as Scenario B. Minimum width of 2.6m still allows for a pass. Note that a significant amount of the oncoming Lane Z is used, which poses a safety risk.</td></tr>
<tr><td rowspan="8">8</td><td rowspan="8">Shawson Drive and Westport Crescent North</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>6</td><td>7.3</td><td>+ 1.3</td></tr>
<tr><td colspan="3">Right-of-way is widened and Lane Z is unaffected.</td></tr>
<tr><td rowspan="2">B</td><td rowspan="2">Pass</td><td>6</td><td>3.3</td><td>- 2.7</td></tr>
<tr><td colspan="3">Right-of-way is reduced.</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Fail</td><td>6</td><td>7.3</td><td>+ 1.3</td></tr>
<tr><td colspan="3">Same as Scenario A, where it changes to a pass at 7.3m and does not utilize Lane Z or Lane X. Right-of-way is widened.</td></tr>
<tr><td rowspan="2">D</td><td rowspan="2">Pass</td><td>6</td><td>DNE</td><td>-</td></tr>
<tr><td colspan="3">Minimum width of 2.6m still results in a pass. Note that the entire Lane Z is used in the turn, and right-of-way is also reduced.</td></tr>
<tr><td rowspan="3">11</td><td rowspan="3">Shawson Drive and Courtneypark Drive</td><td rowspan="2">A</td><td rowspan="2">Fail</td><td>3.4</td><td>9.6</td><td>+ 6.2</td></tr>
<tr><td colspan="3">Lane Y overtakes Lane Z entirely and expands the right-of-way. Note that the main issue causing conflicts at this intersection is the median.</td></tr>
<tr><td>B</td><td>Fail</td><td>3.4</td><td>5.8</td><td>+ 2.4</td></tr>
</table>

Int. #	Intersection	Scenario	Base Grade	Original Lane Y Widths (m)	Threshold Lane Width (m)	Change in Width (m)
				Lane Y expands, while Lane Z shifts over and the right-of-way is adjusted. Note that the combined widths of Lanes Y and Z here is equal to the SBDT in Scenario A.		
		C	Fail	3.4	9.1	+ 5.7
				Changes to a pass at a Lane Y width of 9.1m. Lane Y expands into Lane Z and right-of-way expands.		
		D	Fail	3.4	4.9	+ 1.5
				Lane Y is increased, while Lane Z just shifts to the right and the right-of-way increases.		
13	Courtneypark Drive and Highway 410	A	Pass	8.6 - including shoulders	3.5	- 5.1
				Shoulder width disregarded and included within total width, changes to a fail when the width reaches 3.5 m, right-of-way adjusted.		
		C	Pass	8.6 - including shoulders	3.5	- 5.1
				Exactly the same as Scenario A.		

In reviewing the results of the sensitivity analysis, one conclusion that can be drawn is that adjusting lane widths on the departing road is less effective in changing a failing grade to a passing grade compared to adjusting lane widths on the destination road. In adjusting the lane widths to result in a passing grade, a larger width adjustment is often required on the departing road compared to the destination road for the same intersection and scenario. For example, across the six intersections, there are twelve instances where a failure occurs in the base scenario. In adjusting the departing lane widths, only six of the twelve failing grades are converted into passing grades. The remaining six scenarios yielded DNE solutions, where it was not possible to convert the failing scenarios to passing grades regardless of the departing lane width. Meanwhile, in adjusting the destination lane widths, all twelve of the failing grades are successfully improved to a passing grade with lane width increases. One of the main reasons behind this is due to the shape of the LCV swept path envelope as it makes a left turn, as the envelope starts very wide at the beginning of the turn and requires a long horizontal distance in the receiving lane for the envelope to narrow as the truck recovers its position in the destination lane. This can be seen in *Figure 31*, showing the Scenario A left turn at Westport Crescent North and Shawson Drive with an adjusted destination lane. Here, the swept path envelope is very wide throughout the start of the turn and remains very wide as the LCV enters the receiving lane. It is only after some distance in the receiving lane that the envelope narrows back to the width of the truck to comfortably fit within the receiving lane.

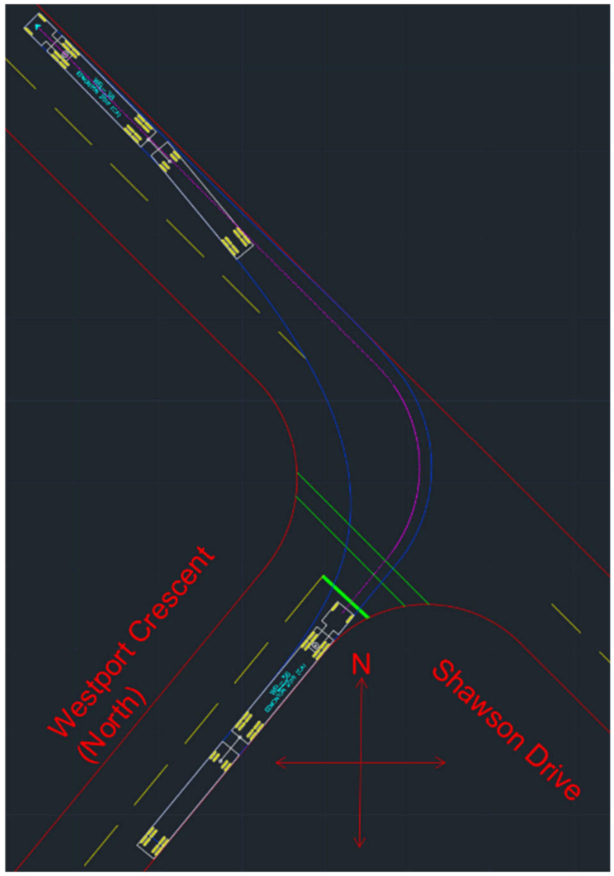

Figure 31: Scenario A at Westport Cr N and Shawson Dr - Adjusted Destination Lane

When the destination lane widths are increased, this provides more space for the left turn to take place with the wider envelope at the beginning of the turn. On the other hand, when the departing lane widths are increased, this does not help in providing the additional space needed for the turn, as the additional envelope width at the start of the turn still struggles with the limited width of the unchanged receiving lane. While the swept path envelope becomes narrower as the horizontal distance along the receiving lane increases, increasing the lane width of the departing lane is often insufficient to allow the envelope to become narrow enough for the LCV to enter the receiving lane without conflict.

Additionally, in comparing the effectiveness of adjusting the departing or destination lanes in creating passing grades, another fact that stands out is the magnitude of the lane width increases in each instance. In these cases, the lane width increase for the departing lane is always larger than the lane width increase needed for a passing grade for the destination lane. To observe this, an example can be seen in Scenario A at Shawson Drive and Westport Crescent North. In adjusting the departing Lane W, a width increase of 6 meters is required to allow the LCV to turn safely, for a total lane width of 12 meters. On the other hand, in adjusting the destination Lane Y at the same intersection and scenario, a width increase of only 1.3 meters is required to allow for a passing grade, for a total lane width of 7.3 meters. This pattern is consistent across the six scenarios where a failing grade was successfully converted to a passing grade, further reinforcing the conclusion that departing lane width adjustments are less effective for resolving failing scenarios than destination lane width adjustments.

Another key finding in the results of this analysis is the LWT in comparison to the base scenario lane widths. For many of the LWTs, for changes to either the departing or destination lane width, the final passing lane width is an unreasonably large value. Across the departing LWTs, some scenarios result in values of 9 meters, 12 meters, and 7.6 meters, representing large increases from their base scenario. Meanwhile, destination lane width adjustments result in some threshold values of 7.9 meters, 8.1 meters, and 9.1 meters. Each of these instances represents a significant width increase from their base scenarios to widths that would not be reasonable in an urbanized area. Taking into consideration the width of roadside amenities and right-of-way allowances between private properties, these lane widths would not be able to fit within these rights-of-way. Many of these larger lane widths primarily occur during Scenarios A and C, which further exacerbates these right-of-way issues when considering an additional lane not permitted for use will also take up space within the right-of-way allowance. Furthermore, this is also concerning when observing that at the unsignalized intersections at each end of Westport Crescent, Scenarios A and C each require significant lane width increases to their already large widths of 6 meters to result in a passing grade. With changes to the departing lane, extreme widths of at least 9 and 12 meters are required for the southern and northern intersections respectively. With changes to the destination lane, widths of at least 7.1 and 7.3 meters are needed in the southern and northern intersections respectively. Therefore, based on this, Scenarios A and C are increasingly unacceptable in accommodating LCVs in a last-mile intersection.

On the other end of this spectrum, where the LWT is identified when a grade changes from a pass to a failure by reducing lane widths, another alarming trend is how close existing lane widths are to their corresponding thresholds. In reducing lane widths in passing scenarios, there are several instances in both the departing and destination lane widths where the threshold lane width leading to a failure is within 1.5 meters of the existing lane width. For instance, the worst of these phenomena occurs in Scenario B at Dixie Road and Britannia Road when changing the destination lane width, with the threshold being only 0.3 meters less than the existing width. As a result, the fact that the failing scenario width is such a small difference from the existing lane width shows that there is very little leeway space for the LCV to maneuver and that existing dimensions can just barely support the left-turning space requirements of the turnpike double.

Looking more closely at specific scenarios, another pattern that stands out is that across both changes to the departing and destination lanes, the LWTs are very close or identical between Scenarios A and C. These two scenarios both involve having only one receiving lane available for use, but Scenario C has an extra lane available on the departing road. Across the six intersections with changes to either departing or destination lanes, nine instances occur where Scenarios A and C result in the same threshold width, while two more instances occur where the threshold widths are within 0.5 meters or less of one another. From this, the previous conclusion is further reinforced that the additional lane on the departing road holds very little value in improving the ability of an LCV to complete left turns. The main reason for this is because in making the left turn in Scenario C, the LCV path generated by IntelliPath often opts not to use the extra departing lane, as there is a limited need for the LCV to move to the right before turning to the left. An example of this can be viewed in *Figure 32*, which shows the Scenario C left turn at the Dixie Road-Highway 401 Off-Ramp intersection after the destination lane was adjusted. In this instance, the LCV starts out in the left-turning Lane W. Despite also having permission to use Lane X to the right on the departing road, IntelliPath does not make use of the second permissible lane, as veering to the right does not contribute to the LCV's ideal left turn. This results in a destination lane width threshold of 7.1 meters, which is identical to the threshold in Scenario A.

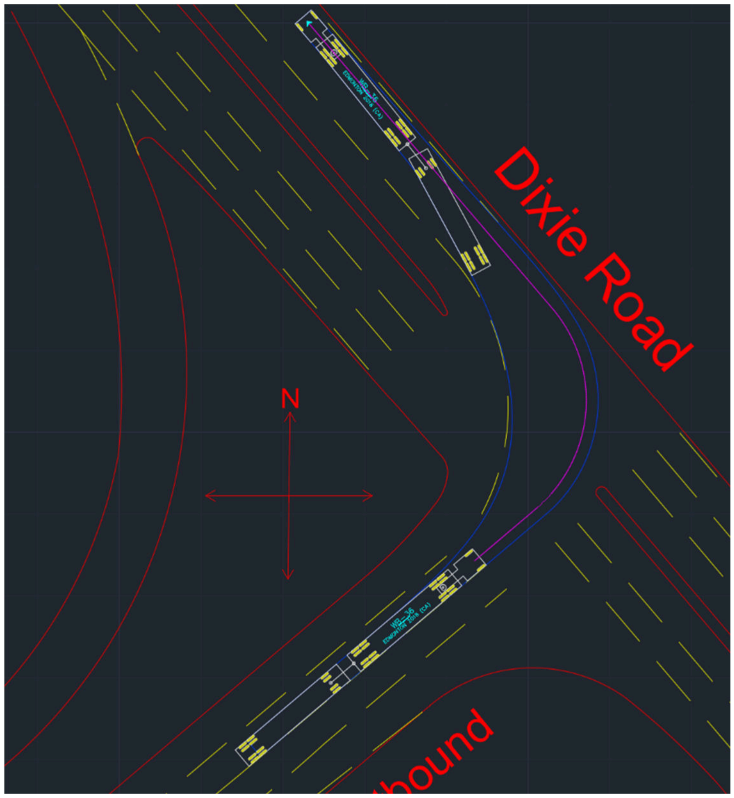

Figure 32: Dixie Rd - Hwy 401 Off-Ramp - Scenario C with Adjusted Destination Lane

One last fact identified in the sensitivity analysis of lane widths is that in Scenario D, where multiple lanes are allowed on both roads, this scenario often resulted in DNE thresholds when changing from a pass to a fail at multiple intersections. In the base scenario, there are five instances where Scenario D resulted in a passing grade, with the Shawson Drive-Courtneypark Drive intersection the only intersection that fails in Scenario D. In the ten attempts to change the grade from a pass to a fail using either departing or destination lanes, eight of them result in DNE solutions, meaning that a minimum lane width of 2.6 meters still results in a passing grade. This is the only scenario where this occurs, with the exception of one instance in Scenario B. This occurrence indicates that Scenario D has the greatest buffer space for LCVs when maneuvering, as a significant reduction in lane widths in these scenarios still results in a passing grade. This is unsurprising, as Scenario D is expected to have the most space for maneuvering due to the availability of extra lanes for use on both roads.

6.2.2 – Setback Distance Sensitivity Analysis

Upon completion of the setback distance sensitivity analysis, the findings of the analyses are recorded in the *Table 15*. Here, changes to the setback distance are evaluated to determine the setback distance threshold (SBDT), which is the setback distance at which the grade status of the left turn changes from a passing grade to a failing grade or vice versa. However, it is important to note that of the six intersections selected in the study area, analyses are only performed on four of them, leaving out the two highway ramp intersections for various reasons. Starting with the Highway 401 Off-Ramp-Dixie Road intersection, this intersection involves the LCV turning left onto Dixie Road using a double left-turn lane. It has been previously assumed that when approaching a double left-turn lane that the LCV will use the lane closest to the centre of the road to prevent encroachment into the inner turning lane. As a result, when the turnpike double is making the turn here, it is assumed that the leftmost left-turn lane is not permitted for use under Scenarios A through D. Due to this assumption, there will never be a case where the swept path envelope passes into the oncoming lanes or over the median on Dixie Road without interfering with the simultaneous left-turn lane. Thus, applying a setback distance in the oncoming lane and cutting back the median will not make any difference since the LCV will not be able to use it.

Similarly, the Courtneypark Drive-Highway 410 On-Ramp intersection is also left out of the sensitivity analysis. Recall that in the layout of this intersection, there is only one receiving lane on the highway on-ramp, with a total width of 8.6 meters including the shoulders. This extra-wide lane is then separated from oncoming lanes of the off-ramp by an 8-meter-wide raised median. As a result, there is no situation in which the trailer of the LCV ever conflicts with either the raised median or the oncoming lanes. Therefore, there is no reason to implement a setback distance on the oncoming road.

Lastly, another unique situation arose at the northern Westport Crescent intersection along Shawson Drive. In this intersection, Shawson Drive always has the right-of-way, as the only stop sign present is on Westport Crescent. Therefore, with no vehicles stopping at the intersection along Shawson Drive, there would be no stopping line to assign a setback distance to, making this sensitivity analysis moot. However, for the sake of conducting analyses on multiple unsignalized intersections, it is assumed that there is a stopping line along the oncoming lane of Shawson Drive. This stopping line is manually assigned, aligned with the end of the centerline of the road before the intersection, and is subject to changes in the setback distance sensitivity analysis.

Therefore, the analysis results of the remaining four intersections can be found in *Table 15*. For each intersection, Scenarios A through D are assessed, with the initial (S_0) and final setback distances (S_f) each recorded. The grade status and changes to the medians are noted as the adjustments are made and the change in setback distance is also calculated for each scenario.

Table 15: Setback Distance Sensitivity Analysis Results

<table>
<tr><th>Int. #</th><th>Intersection</th><th>Scenario</th><th>Base Grade</th><th>Existing Setback Distance (m) [S_0]</th><th>Final Setback Distance (m) [S_f]</th><th>Change in Distance (m)</th><th>Median Cut or Removed?</th></tr>
<tr><td rowspan="2">1</td><td rowspan="2">Dixie Road and Highway 401 Off-Ramp</td><td rowspan="2">-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td colspan="5">Not applicable since LCV is turning from a dual left-turn lane</td></tr>
<tr><td rowspan="8">4</td><td rowspan="8">Dixie Road and Britannia Road</td><td rowspan="2">A</td><td>Fail</td><td>7.6</td><td>DNE</td><td>-</td><td>-</td></tr>
<tr><td colspan="5">Increasing the setback to 43m, IntelliPath still provides a failing grade. In this case, the trailer conflicts with the median on the departing road. Oversteering to avoid this leads to conflicts with Lane Z.</td></tr>
<tr><td rowspan="2">B</td><td>Pass</td><td>7.6</td><td>7</td><td>- 0.6</td><td>Median Extended</td></tr>
<tr><td colspan="5">Changes to a fail once the median and stop line are moved forward 0.6m. At this point, the trailer drags over the curb.</td></tr>
<tr><td rowspan="2">C</td><td>Fail</td><td>7.6</td><td>31.9</td><td>+ 24.3</td><td>Removed</td></tr>
<tr><td colspan="5">Changes to a pass only after the median is removed completely. Still produces an unreasonable setback distance.</td></tr>
<tr><td rowspan="2">D</td><td>Pass</td><td>7.6</td><td>DNE</td><td>-</td><td>Median Extended</td></tr>
<tr><td colspan="5">Extending the median and stopping line towards the pedestrian crosswalk still results in a pass. Final setback distance tested is 5.2m, for a change of -1.9m.</td></tr>
<tr><td rowspan="8">7</td><td rowspan="8">Shawson Drive and Westport Crescent South</td><td rowspan="2">A</td><td>Fail</td><td>15</td><td>20.6</td><td>+ 5.6</td><td>N/A</td></tr>
<tr><td colspan="5">Changes to a pass at 20.6m (No median to begin with).</td></tr>
<tr><td rowspan="2">B</td><td>Pass</td><td>15</td><td>Not Feasible</td><td>-</td><td>N/A</td></tr>
<tr><td colspan="5">Because Lane Z, the oncoming lane, is already permitted for use, any setback distance should be acceptable, up until the crosswalk.</td></tr>
<tr><td rowspan="2">C</td><td>Fail</td><td>15</td><td>20.4</td><td>+ 5.4</td><td>N/A</td></tr>
<tr><td colspan="5">Changes to a pass at 20.4m (No median to begin with).</td></tr>
<tr><td rowspan="2">D</td><td>Pass</td><td>15</td><td>Not Feasible</td><td>-</td><td>N/A</td></tr>
<tr><td colspan="5">Identical to Scenario B.</td></tr>
<tr><td rowspan="8">8</td><td rowspan="8">Shawson Drive and Westport Crescent North</td><td rowspan="2">A</td><td>Fail</td><td>12.6</td><td>20</td><td>+ 7.4</td><td>N/A</td></tr>
<tr><td colspan="5">Changes to a pass when the setback is moved back 7.4m. (This road originally has no stop on Shawson Drive, but is adjusted for this analysis)</td></tr>
<tr><td rowspan="2">B</td><td>Pass</td><td>12.6</td><td>Not Feasible</td><td>-</td><td>N/A</td></tr>
<tr><td colspan="5">Because Lane Z, the oncoming lane, is permitted for use, any setback distance should be acceptable, up until the crosswalk. (This road originally has no stop on Shawson Drive, but is adjusted for this analysis)</td></tr>
<tr><td rowspan="2">C</td><td>Fail</td><td>12.6</td><td>20</td><td>+ 7.4</td><td>N/A</td></tr>
<tr><td colspan="5">Changes to a pass when setback is moved back 7.4m. (This road originally has no stop on Shawson Drive, but is adjusted for this analysis)</td></tr>
<tr><td rowspan="2">D</td><td>Pass</td><td>12.6</td><td>Not Feasible</td><td>-</td><td>N/A</td></tr>
<tr><td colspan="5">Identical to Scenario B</td></tr>
<tr><td rowspan="4">11</td><td rowspan="4">Shawson Drive and Courtneypark Drive</td><td rowspan="2">A</td><td>Fail</td><td>12</td><td>DNE</td><td>-</td><td>-</td></tr>
<tr><td colspan="5">Even after moving the stopping line back, LCV trailer still drags through the second oncoming lane on the receiving road and through the oncoming lane on the departing road.</td></tr>
<tr><td rowspan="2">B</td><td>Fail</td><td>12</td><td>15.9</td><td>+ 3.9</td><td>Cut</td></tr>
<tr><td colspan="5">Changes to a pass when moved back 3.9m.</td></tr>
</table>

Int. #	Intersection	Scenario	Base Grade	Existing Setback Distance (m) [S_0]	Final Setback Distance (m) [S_f]	Change in Distance (m)	Median Cut or Removed?
		C	Fail	12	32	+ 20.0	Removed
			With the median intact, the setback distance threshold is assigned as DNE, as the lane is still not wide enough for the turning LCV. But removing the median, a setback of 32m is acceptable since there is extra space for the LCV to turn. Nevertheless, a 32m setback distance is unreasonable.				
		D	Fail	12	12.7	+ 0.7	Cut
			Changes to a pass when the setback is pushed back 0.7m. It should be noted that most of the improvement actually comes from cutting the median, which is cut back by 1.8m. This is because the median extends further into the intersection beyond the stopping line.				
13	Courtneypark Drive and Highway 410 On-Ramp	-	-	-	-	-	-
			Not applicable since oncoming lanes are nowhere near the swept path envelope.				

In reviewing the results of this sensitivity analysis, there are multiple trends that stand out. First, in reviewing the efficacy of setbacks in changing a failing grade into a passing grade, assigning additional setback distances as a solution performs moderately well. In the ten base scenarios that resulted in failing grades, eight of them are resolved into a passing grade using a setback distance. At first glance, this 80% success rate appears promising. However, a closer review of the results reveals some significant concerns considering the magnitude of the setback distance changes and the presence or removal of raised medians. In comparing Scenarios A through D, Scenarios A and C require much larger increases to the setback distance in order to change from a failing grade to a passing grade than Scenarios B and D. Across Scenarios A and C, four out of eight setback distance increases, including DNE setback distances, reach values of over 20 meters, resulting in final setback distances of over 30 meters. These increases are particularly concerning, as such setback distances are unrealistic to be implemented on densely designed arterials. For context, *Figure 33* showcases the results of the setback distance determined in Scenario C at the Dixie Road-Britannia Road intersection. Here, an unrealistically large setback distance of 31.9 meters is required.

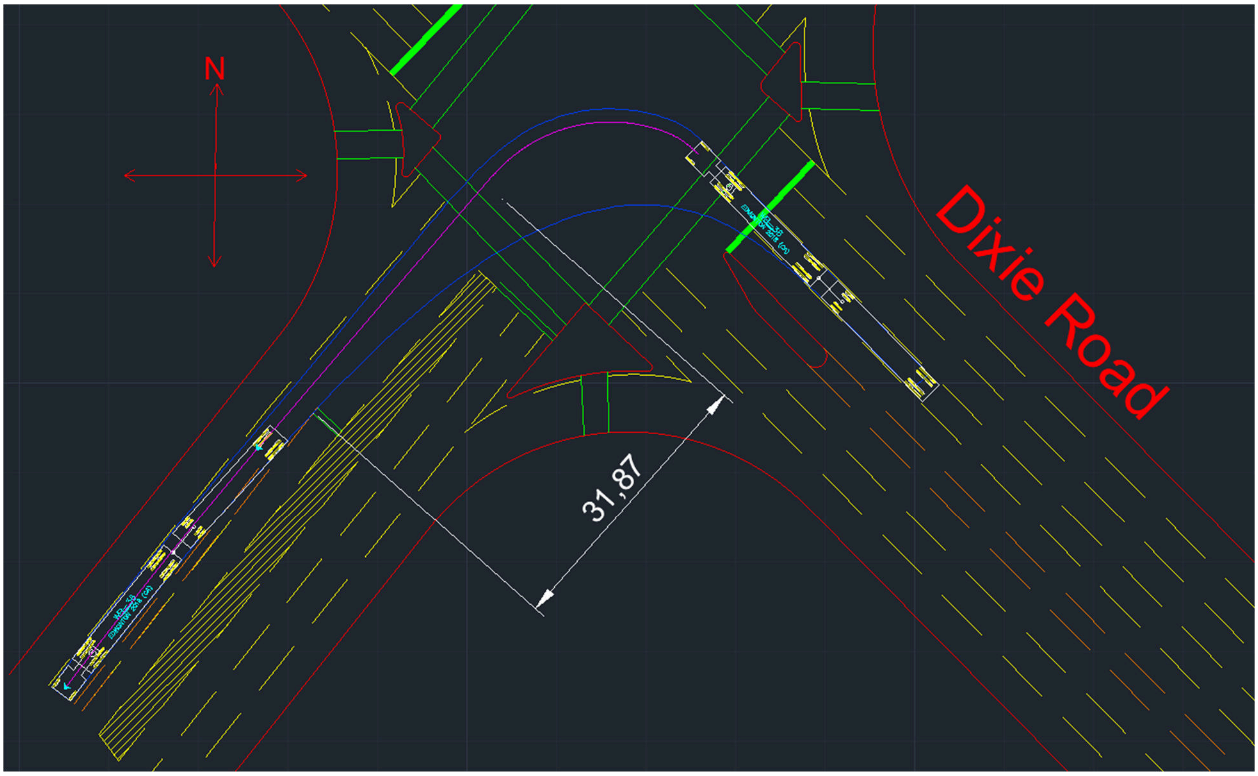

Figure 33: Passing-Grade Setback Distance for Scenario C of Dixie Rd-Britannia Rd

It should be noted that in these Scenarios A and C requiring unreasonably large setback increases, they each require the median to be removed entirely. This means that no setback distance yielded a passing grade while the median was still present in the right-of-way, resulting in a DNE setback. Instead, IntelliPath could only determine these passing grade setback distances after the median is removed, which essentially widens the receiving lane for the LCV. Furthermore, in two of the eight Scenarios A and C, a DNE setback distance is yielded. This means that even after the removal of the median in these two cases, a passing grade cannot be generated even as the setbacks are increased to the limits of the AutoCAD drawing, with setback distances of over 40 meters in some cases still yielding failing grades. Therefore, in situations where only one receiving lane is available for use by the LCV, these results show that last-mile intersections are not well suited to accommodate the swept paths of LCVs during left turns. Similar to the lane width sensitivity analysis, the reason for this is due to the large size of the swept path envelope as the LCV makes a left turn, as the width of the envelope is very wide at the beginning of the turn and requires a considerable horizontal distance to return to a narrower path. In Scenarios A and C, limited space is available for LCVs since only one receiving lane is permitted for use. To make space for the width of the swept path envelope, the oncoming lane and median must be set back a large distance until the swept path is narrow enough to fit within the existing receiving lane. Referring back to *Figure 33*, this phenomenon can be observed, as it requires 31.9 meters of horizontal distance along Britannia Road before the swept path of the LCV is narrow enough to fit within the receiving Lane Y.

Another notable occurrence in changing a failing grade to a passing grade in Scenarios A and C can be observed at the minor intersections at both ends of Westport Crescent. For both Scenarios A and C, the northern and southern Westport Crescent intersections require setback distance increases of 7.4 and 5.4 meters respectively, for final setback distances of 20 and 20.4 meters respectively. These setback distance increases are much smaller than those proposed at the arterial intersections and thus are more realistic to be implemented. These smaller setback distances may be an indication that these two intersections are better prepared to accommodate LCVs in scenarios where only one lane is permitted for use on the destination road. This can also be attributed to the fact that the lanes along Shawson Drive and Westport Crescent, with their widths of 6 meters, already provide a considerable amount of space for the LCV swept path envelope to make the turn, with only a small horizontal distance needed in the oncoming lane to provide extra space for the turning envelope. Additionally, the identical SBDT values for Scenarios A and C in both intersections is due to the fact that the ideal path selected in Scenario C does not make use of the additional departing lane, ultimately producing results identical to Scenario A.

Contrarily, in changing a failing grade to a passing grade in Scenarios B and D, these scenarios require much smaller setback distance increases. The only intersection where failing grades occurred in Scenario B and D was at the Courtneypark Drive-Shawson Drive intersection. Here, Scenarios B and D require setback distance increases of 3.9 and 0.7 meters respectively, for final setback distances of 15.9 and 12.7 meters respectively. Again, with increased maneuvering space available in Scenario B from a second receiving lane on Courtneypark Drive, much less space from a setback of the oncoming lane and median is required. This is because most of the swept path envelope's width is contained within the two existing receiving lanes, only needing extra space from the median and oncoming lanes for a short horizontal distance. For example, this can be observed in *Figure 34*, which shows Scenario B of the Shawson Drive-Courtneypark Drive intersection. For a passing grade in this case, a much smaller setback is required of the destination road's oncoming lane due to extra space available from both Lanes Y and Z. Scenario D goes one step further in providing extra space for maneuvering, with a second lane available on both Shawson Drive and Courtneypark Drive, resulting in the lowest setback distance of this intersection.

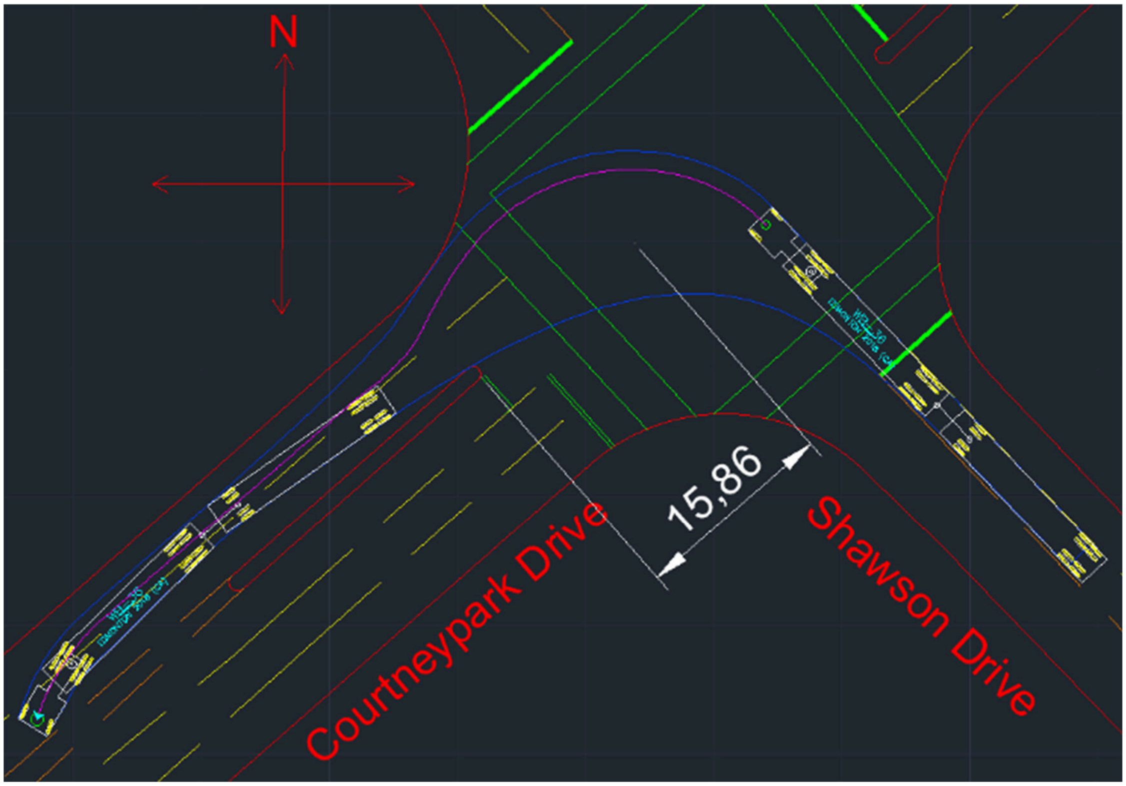

Figure 34: Passing-Grade Setback Distance for Scenario B of Shawson Dr-Courtneypark Dr

On the other hand, findings are somewhat limited when attempting to change a passing grade to a failing grade using adjustments to the setback distance. In the base scenario, there are six instances where a passing grade occurred, taking place during Scenarios B and D of the Dixie Road-Britannia Road intersection and the two Westport Crescent-Shawson Drive intersections. However, before this analysis could take place, it should be noted that a unique situation occurs at the Westport Crescent intersections, which makes up four of the six scenarios that could be analyzed for pass-to-fail setbacks. In each of these Westport Crescent scenarios, the oncoming lane serves as the additional lane available on the destination road in Scenarios B and D. This means that the change to the setback distance occurs within this oncoming lane that is already permitted for use in Scenarios B and D. As a result, since the whole lane is already permitted for use, technically any setback distance should be acceptable up until the stopping line is at the crosswalk. This situation never results in a failing grade, as the permissions to use the oncoming lane renders the position of the setback distance irrelevant. Thus, these four scenarios are not feasible to provide an accurate reflection of statistics regarding changing a passing grade to a failing grade. Excluding these two intersections, only the Scenarios B and D at the Dixie Road-Britannia Road intersection are available for an analysis of changing a passing grade to a failing grade.

In limited opportunities to change a passing grade to a failing grade at the Dixie Road-Britannia Road intersection, adjusting the setback distance failed to change the grade status once out of the two scenarios. In moving the stopping line of the oncoming lane closer to the pedestrian crosswalk at each intersection, Scenario D results in a DNE setback distance. This means that the stopping line in the oncoming lane was moved forward to be in line with the pedestrian crosswalk and still results in a passing grade from IntelliPath. Moving the stopping line forward beyond the crosswalk is not logical, as the vehicles would be stopped within the crosswalk, posing a safety risk to pedestrians, so this setback that still results in passes at the crosswalk is assigned a DNE value. This inability to change from a passing grade to a failing grade in this scenario indicates that the extra lanes assigned on the departing and destination roads provides a significant amount of buffer space for the LCV to maneuver. On the other hand, in Scenario B at the Britannia Road-Dixie Road intersection, moving the stopping line 0.6 meters closer to the intersection results in a failing grade. In this instance, the trailers of the turnpike double begin to conflict with the median on Britannia Road. Unlike in the lane width sensitivity analysis, there are very limited opportunities to apply a setback distance sensitivity analysis to change a passing grade to a failing grade, inhibiting the analysis from generating clear conclusions.

6.2.3 – Logit Model Results

Four binomial logit models are shown in *Table 16*, with each variable assigned to the utility of the passing grade. Positive parameter coefficients therefore indicate a higher likelihood of successful left-turn maneuvers. Variables included in these models are the scenario type, the average departing and destination road widths, the setback distance, and the intersection. The variable for Scenario A is left out of the model to represent a baseline comparison with the other scenarios. Positive parameters for other scenarios therefore indicate a higher likelihood of success compared Scenario A. A similar approach is also used for Intersection 13, leaving this intersection variable out of the model. A description of each intersection analyzed in this model is provided in

Chapter 3. Data. The full NLOGIT code for each model, along with the NLOGIT outputs, are showcased in *Appendix A.3*.

Model 1 is estimated using the lane width variables, scenarios, and setback distance variable, while excluding the intersection-specific dummy variables. There are 78 data points for this model based on the performance of existing road geometry, the LWTs (embodied within the average departing and destination widths), and the SBDTs where a grade status change occurs. This can be compared with the same configuration of variables for Model 2, but Model 2 also includes additional intermediate data points, as discussed previously in *Chapter 4. Swept Path Methodology*. The results slightly differ between Models 1 and 2, but the general relationships for significant parameters remain consistent. The goodness of fit results, as denoted by the Adjusted ρ^2, improve slightly from 0.301 to 0.334 from Model 1 to Model 2. Therefore, the remaining models use the larger dataset that includes intermediate data points.

Model 3 presents results for a model using only the intersection-specific variables to isolate the impact of intersection type on the results. The Adjusted ρ^2 of Model 3 is 0.079 and therefore reflects a lower data fit compared with the other variables. This further indicates that the results do not depend greatly on the intersection type and are more dependent on other variables.

Lastly, Model 4 utilizes the larger input data set and includes all tested variables. This model also provides the best results based on the Adjusted ρ^2 goodness of fit measure equal to 0.453. This is a very strong result for a logit model and will be used as the selected model to review individual variable results.

In analyzing specific variables, the Scenario B, Scenario C, and Scenario D variables for Model 4 all exhibit positive parameters, indicating a higher likelihood of success compared to Scenario A, with all other things being equal. However, Scenario C is not statistically significant and exhibits a very minor improvement compared to Scenario A. Scenario B is statistically significant and exhibits a much larger parameter value compared to Scenario C. This result suggests that enabling two lanes at the destination leg of the turn provides much more benefit compared to additional lanes at the departing leg. Scenario D provides the largest improvement based on the highest parameter value among the scenario variables by allowing two lanes at both ends of the turn. Similarly, the average destination width per lane is shown to be more statistically significant when compared with the average departure width per lane. The elasticities shown for these variables indicate that a 1% change in average destination width per lane results in a 3.86% increase in the likelihood of successful maneuvering, which is highly elastic. The elasticity for average departure width per lane suggests a somewhat elastic result of 1.035%. The setback distance variable also exhibits a statistically significant parameter, with an elasticity of 1.331%. The Model 4 results for the intersection dummy variables are all shown to have negative parameters that are statistically significant. These intersections are therefore less likely to encounter a successful maneuver when compared with the baseline intersection specified as Intersection 13. Intersection 11 exhibits the lowest likelihood of successful maneuvers based on the most negative parameter when compared to the other intersections. It should be noted that Model 4, which includes the intersection dummy variables, risks becoming an overfit model. As a result, this model is not transferrable to other road networks due to the inclusion of variables that capture intersections specific to this study area.

Beyond the variables observed in the logit results presented here, several other intersection characteristics were also tested for statistical significance but were ultimately excluded from the models. These included factors such as the individual lane widths of Lanes W, X, Y, and Z, whether any of the lane widths are assigned the minimum width of 2.6 meters, the status of the median being cut, removed, extended, or non-existent during the setback sensitivity, whether the intersections are considered a major or minor intersection, or whether the departing or destination road is considered a major or minor road. In the case of the individual lane widths, the effect of these variables is embedded within the average departing and destination widths, and therefore, these variables become redundant. Meanwhile, the other variables that are left out of the logit model were too inconsistent or limited in sample size between the different intersections.

Table 16: NLOGIT Software Analysis Results

Variable		Model 1	Model 2	Model 3	Model 4
Constant	Parameter	-10.047***	-8.551***	1.466**	-9.053***
Scenario B	Parameter	2.580**	3.517***		4.796***
	Direct Effect	0.356	0.483		0.515
Scenario C	Parameter	1.533	-0.207		0.316
	Direct Effect	0.211	-0.028		0.034
Scenario D	Parameter	5.179***	4.735***		6.682***
	Direct Effect	0.715	0.651		0.718
Avg. Departing Width (m)	Parameter	0.401**	0.175*		0.331*
	Elasticity	0.752	0.547		1.035

Variable		Model 1	Model 2	Model 3	Model 4
Avg. Destination Width (m)	Parameter	1.068***	0.861***		1.263***
	Elasticity	1.831	2.632		3.86
Setback Distance (m)	Parameter	0.075*	0.067**		0.134***
	Elasticity	0.486	0.671		1.331
Intersection 1	Parameter			-2.255***	-4.225***
	Direct Effect			-0.455	-0.454
Intersection 4	Parameter			-2.220***	-2.893***
	Direct Effect			-0.448	-0.311
Intersection 7	Parameter			-1.584**	-4.519***
	Direct Effect			-0.320	-0.486
Intersection 8	Parameter			-1.663**	-4.616***
	Direct Effect			-0.336	-0.496
Intersection 11	Parameter			-3.276***	-5.537***
	Direct Effect			-0.661	-0.595
Number of Observations		78	264	264	264
ρ^2		0.364	0.352	0.100	0.447
Adjusted ρ^2		0.301	0.334	0.079	0.453

*, **, *** represent a statistical significance of 90%, 95%, and 99% respectively. No asterisk assigned represents a statistical significance less than 90%.

Chapter 7. Microsimulation Results

In assessing the safety of LCVs in last-mile settings using PTV Vissim and the Surrogate Safety Assessment Model (SSAM), aggregate results are first generated for the study area as a whole. This is then followed by an analysis of specific intersections within the network to compare the safety trends at an intersection-level with the aggregate results. The following sections detail the aggregate-level and intersection-level results generated. As previously discussed, all results moving forward exclude the Category 1 links.

7.1 – Aggregate-Level Analysis of Left-Turning Freight Conflicts

In assessing the aggregate results of the SSAM analysis, combining the total results for all three simulation runs for each penetration level, four main measures are used to evaluate the safety of the network at each LCV penetration level; the conflict frequency, time-to-collision (TTC) of each conflict, post-encroachment time (PET) of each conflict, and conflict type.

7.1.1 – Aggregate Conflict Frequency Results

In sorting through the aggregate SSAM results, the number of conflicts occurring at each LCV penetration level is determined. More specifically, this included results for the total number of conflicts, the number of freight conflicts, and the number of non-freight conflicts. These results are recorded in *Table 17*, along with the percent change between the base scenario and each LCV penetration level.

Table 17: Aggregate Conflict Frequency Results

LCV Penetration Level	# of Safety Conflicts					
	Total Conflicts	% Change from Base Scenario	Freight Conflicts	% Change from Base Scenario	Non-Freight Conflicts	% Change from Base Scenario
0%	675	-	236	-	439	-
25%	1015	+ 50%	584	+ 147%	431	- 2%
50%	1299	+ 92%	847	+ 259%	452	+ 3%
75%	1610	+ 139%	1182	+ 401%	428	- 3%
100%	1712	+ 154%	1290	+ 447%	422	- 4%

In reviewing the aggregate conflict frequency, one fact that stands out is that as the LCV penetration level increases, the number of freight conflicts increases significantly. In increasing the percentage of LCVs to the first level of 25%, the number of freight conflicts more than doubles from the base scenario, representing a major increase. Further increases to 50%, 75%, and 100%

LCVs, while not always practical in reality but still serving as a reference point for safety analyses, still leads to even more extreme increases in freight conflicts, yielding a 259%, 401%, and 447% increase respectively from the base scenario. Since LCVs are part of the heavy goods vehicle (HGV) class, this increase in freight conflicts is not surprising considering these arterial roads are not well equipped for freight vehicles of this size. Furthermore, in observing the changes in total safety conflicts, which includes passenger cars, buses, and light goods vehicles (LGV) in addition to HGVs, an increase in LCVs also leads to an increase in total conflicts, although at much smaller proportions compared to the freight conflicts. Conversely, in observing the number of non-freight conflicts, which is limited to passenger cars, buses, and LGVs, the number of conflicts remained stagnant throughout each LCV penetration level, experiencing miniscule increases or decreases from the base scenario. From this, it can be concluded that the increase in LCVs has little to no impact on non-freight conflicts, as behavior between the passenger cars, buses, and LGV classes remain unchanged in the presence of LCVs. In addition, while total safety conflicts increase with each LCV penetration level, these conflict increases are accounted for by the increase in conflicts among freight vehicles, as freight conflicts experience significant increases and non-freight conflicts remain largely unchanged with the introduction of LCVs.

Another interesting trend occurring here is that despite freight vehicles being only a small portion of the entire vehicle composition of the network, the number of freight conflicts are disproportionately represented in the results. Consider that in the base scenario, where no LCVs are present, the number of non-freight conflicts is almost double the number of freight conflicts occurring. However, this proportion is not consistent when LCVs are added into the network. At each LCV penetration level from 25% to 100%, the number of freight conflicts drastically outnumbers the number of non-freight conflicts, more than tripling the number of non-freight conflicts at the 100% penetration level. From this, it can be concluded that the number of potential collisions caused per LCV introduced into the network is far greater than the number of potential collisions caused per conventional freight vehicle or non-freight vehicle in the network.

7.1.2 – Aggregate Time-to-Collision Results

For the three simulation runs for each LCV penetration level, the TTC values for the freight collisions are recorded in seconds. Using the default SSAM threshold for TTC, any freight conflict with a TTC between 0 and 1.5 seconds is recorded, with a lower TTC indicating a more severe conflict. The conflicts are further ranked into intervals of 0.5 seconds to separate the conflicts into different levels of severity. The results of this analysis can be found in the table below.

Table 18: Aggregate Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	140 (59.3%)	37 (15.7%)	5 (2.1%)	54 (22.9%)
25%	465 (79.6%)	66 (11.3%)	11 (1.9%)	42 (7.2%)
50%	685 (80.9%)	116 (13.7%)	14 (1.7%)	32 (3.8%)
75%	970 (82.1%)	150 (12.7%)	14 (1.2%)	48 (4.1%)
100%	1094 (84.8%)	124 (9.6%)	12 (0.9%)	60 (4.7%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Amongst the aggregate TTC results, one obvious trend that can be identified is that in the presence of LCVs, the majority of conflicts are very severe, with a TTC equal to 0 seconds. In the base scenario, with no LCVs present in the network, the conflict severity is distributed between the four severity levels, with 59.3% at the TTC = 0 level, followed by 22.9% at the lowest severity level between 1 and 1.5 seconds, 15.7% at the 0 to 0.5 second level, and 2.1% at the 0.5 to 1 second level. In increasing the LCV penetration level to 25%, this distribution changes drastically. At the first LCV penetration level, the conflict severity increases significantly, with the TTC distribution skewed heavily towards the TTC = 0 interval. The addition of LCVs in the first LCV penetration level significantly increased conflicts with a TTC equal to 0 seconds from 59.3% to 79.6%, all while reducing the number of conflicts in each TTC interval over 0 seconds. Furthermore, with each subsequent LCV penetration level, the severity of conflicts increases marginally, with the TTC = 0 severity level increasing from 80.9% at 50% LCVs to 84.8% at 100% LCVs. Therefore, the introduction of LCVs at any penetration level results in a major increase in the conflict severity from the base scenario.

7.1.3 – Aggregate Post-Encroachment Time Results

As an alternative surrogate measure of safety, freight conflict PETs are also analyzed for this network. Similar to the aggregate TTC analysis, PET values for each freight conflict are recorded by SSAM for the three simulation runs in Vissim. SSAM's default threshold for PET is used, with the program recording any conflict with a PET between 0 and 5 seconds. Likewise, the freight conflicts are also sorted into PET ranges based on severity, with a PET equal to 0 seconds being the most severe conflicts and 1-second intervals being used between 0 and 5 seconds. The results of the aggregate PET investigation are exhibited in *Table 19*.

Table 19: Aggregate Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	146 (61.9%)	31 (13.1%)	20 (8.5%)	32 (13.6%)	5 (2.1%)	2 (0.9%)
25%	468 (80.1%)	65 (11.1%)	21 (3.6%)	20 (3.4%)	8 (1.4%)	2 (0.3%)
50%	694 (81.9%)	93 (11.0%)	37 (4.4%)	11 (1.3%)	9 (1.1%)	3 (0.4%)
75%	978 (82.7%)	116 (9.8%)	43 (3.6%)	24 (2.0%)	13 (1.1%)	8 (0.7%)
100%	1096 (85.0%)	107 (8.3%)	37 (2.9%)	24 (1.9%)	22 (1.7%)	4 (0.3%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

From these aggregate PET results, it can be seen that the trends exhibited in the aggregate TTC results are repeated for the PET results. Starting with the base scenario, a fair majority of the freight conflicts at 61.9% are ranked at the most severe PET, with a PET of 0 seconds. The remaining conflicts in the base scenario are then distributed between the other severity intervals, with a PET between 2 and 3 seconds being the next most prominent, followed by a PET between 0 and 1 second and between 1 and 2 seconds. Meanwhile, very few freight conflicts rank within the least severe intervals between 3 and 4 seconds and between 4 and 5 seconds. However, in increasing the number of LCVs from the 0% base scenario up to 25%, a significant increase in the conflict severity is observed, with the PET distribution heavily skewed in favour of a PET of 0 seconds. Correspondingly, the other five PET intervals experience decreases from their base scenario proportions. Furthermore, in increasing LCV penetration levels by 25% intervals from 25% LCVs to 100% LCVs, only marginal increases occur in the most severe collisions with PETs of 0 seconds, maxing out at 85.0% of conflicts. This is the exact same pattern identified with the TTC results, where a large increase in the most severe conflicts occurs when LCVs are initially introduced at the 25% level, followed by marginal increases at each subsequent level. Again, this concludes that LCVs at any penetration level result in a major increase in the conflict severity compared to the base scenario.

7.1.4 – Aggregate Conflict Type Results

The final key piece of information generated for the aggregate results by SSAM is the conflict type. Based on the trajectories of the vehicles in Vissim, SSAM classifies each conflict into three main categories. The first category is a crossing conflict, where the paths of two vehicles travelling in different directions on different links meet, resulting in a collision. The second type is a rear end collision, where one vehicle's actions causes it to collide with another vehicle travelling in the same direction, often on the same link, while following or being followed by the second vehicle. Lastly, the third category is a lane changing conflict, where a collision with another vehicle is caused while changing lanes in the road network. In classifying each conflict across each LCV penetration level, the following table of results is generated.

Table 20: Aggregate Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	123 (52.1%)	93 (39.4%)	20 (8.5%)
25%	209 (35.9%)	310 (53.1%)	65 (11.1%)
50%	367 (43.3%)	380 (44.9%)	100 (11.8%)
75%	491 (41.5%)	534 (45.2%)	157 (13.3%)
100%	535 (41.5%)	589 (45.7%)	166 (12.9%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

As with the TTC and PET analyses, many trends in the conflict type can be identified by observing differences between each LCV level in comparison to the base scenario. In reviewing the base scenario, with 0% of freight vehicles composed of LCVs, crossing collisions are the most common type of freight conflict, making up just over half of all conflicts at 52.1%. The next most common conflict type is rear end conflicts at 39.4%. Lastly, very few lane changing conflicts factored into the safety analysis, with only 8.5% of conflicts occurring during a lane change. With the introduction of LCVs at the first penetration level, the proportion of rear end collisions increases to 53.1%, with this type becoming the most prominent collision type at 25% LCVs. At the same time, the proportion of crossing conflicts decreases to 35.8%, while the proportion of lane changing conflicts increases slightly, but remains the least common. Upon further introduction of LCVs at the 50%, 75%, and 100% levels, the proportions of rear end conflicts and crossing conflicts gradually level off, with an approximately even split between these two conflict types across these three penetration levels. At the same time, lane changing conflicts make up the remaining proportion of conflicts and are generally consistent. Based on these aggregate results, it would appear that LCVs have the propensity to increase the occurrence of rear end conflicts across the network, making them one of the most prominent conflict types, while also maintaining the levels of crossing conflicts across the network. The next step before drawing any further conclusions is to observe conflicts at the intersection level to assess the trends at each intersection in relation to the aggregate conflict trends.

7.2 – Intersection-Level Analysis of Left-Turning Freight Conflicts

To assess the safety trends of specific intersections in comparison to the aggregate results, the intersections with the most conflicts throughout the network are first identified. Once these intersections are identified, a detailed analysis of the TTC, PET, and conflict type trends is completed, along with an investigation into the reason behind these trends at each intersection. The following sections will outline the results of this procedure.

7.2.1 – High-Conflict Intersections

The first step of the intersection-level analysis is to conduct a comparison between the thirteen intersections to determine which intersections contain the largest proportion of safety conflicts in the network. Two methods are used to conduct this assessment. First, conflicts at each intersection based on frequency is used, where the number of freight conflicts at each intersection is determined for the five LCV penetration levels. Then, the conflicts at each intersection based on exposure is used, where the ratio of freight conflicts per freight left turn at each intersection is calculated for the five LCV penetration levels.

Starting with the intersections based on conflict frequency, each intersection's total number of freight conflicts is recorded together in *Table 21* for each of the five LCV penetration levels. Additionally, based on the total freight conflicts occurring across the entire road network, the percentage of freight conflicts that occur at each intersection is calculated. In each LCV level, the six intersections with the highest percentages of conflicts are highlighted with bold font. Additionally, each of the thirteen intersections are numbered in the chronological order they appear in the route starting at the Highway 401 EB Off-Ramp and Dixie Road intersection, consistent with *Table 4*.

Table 21: Intersection Conflict Summary by Conflict Frequency

LCV Penetration Level	0%	25%	50%	75%	100%
Intersection 1	**11 (4.7%)**	28 (4.8%)	22 (2.6%)	54 (4.6%)	54 (4.2%)
Intersection 2	1 (0.4%)	**49 (8.4%)**	41 (4.8%)	**92 (7.8%)**	**111 (8.6%)**
Intersection 3	**27 (11.4%)**	**115 (19.7%)**	**141 (16.7%)**	**210 (17.8%)**	**181 (14.0%)**

LCV Penetration Level	0%	25%	50%	75%	100%
Intersection 4	**19 (8.1%)**	**79 (13.5%)**	**134 (15.8%)**	**210 (17.8%)**	**253 (19.6%)**
Intersection 5	**8 (3.4%)**	11 (1.9%)	23 (2.7%)	23 (2.0%)	42 (3.3%)
Intersection 6	3 (1.3%)	20 (3.4%)	**54 (6.4%)**	68 (5.8%)	70 (5.4%)
Intersection 7	1 (0.4%)	7 (1.2%)	6 (0.7%)	16 (1.4%)	18 (1.4%)
Intersection 8	3 (1.3%)	4 (0.7%)	7 (0.8%)	10 (0.9%)	10 (0.8%)
Intersection 9	4 (1.7%)	12 (2.1%)	26 (3.1%)	24 (2.0%)	34 (2.6%)
Intersection 10	1 (0.4%)	1 (0.2%)	8 (0.9%)	5 (0.4%)	12 (0.9%)
Intersection 11	**57 (24.2%)**	**32 (5.5%)**	**91 (10.7%)**	**88 (7.5%)**	**73 (5.7%)**
Intersection 12	**97 (41.1%)**	**122 (20.9%)**	**132 (15.6%)**	**147 (12.4%)**	**180 (14.0%)**
Intersection 13	4 (1.7%)	**104 (17.8%)**	**162 (19.1%)**	**235 (19.9%)**	**252 (19.5%)**

*Each entry is written as the number of freight conflicts followed by the percentage it makes up of all conflicts in brackets
**Bold indicates the intersection is amongst the six highest-conflict intersections for that LCV penetration level

In comparing the thirteen intersections across the LCV penetration levels, several trends with respect to the base scenario start to emerge. In reviewing the base scenario, Intersection 12, at Courtneypark Drive and Tomken Road, is the intersection with the most conflicts by far, with 41.1% of the freight conflicts in the network, followed by Intersection 11, at Shawson Drive and Courtneypark Drive, Intersection 3, at Dixie Road and Shawson Drive, and Intersection 4, at Dixie Road and Britannia Road, with 24.2%, 11.4%, and 8.1% of the freight conflicts respectively. Aside from these four intersections, not many intersections in the base scenario experience a significant number of freight conflicts, each with less than 5% of the total freight conflicts. In increasing the LCV penetration level, Intersections 3, 4, 11, and 12 are reinforced as four of the highest-conflict intersections, as they always rank within the top six most conflicting intersections across each LCV penetration level. Furthermore, the other two intersections ranked as the next two most concerning intersections are Intersection 2, at Dixie Road and the Highway 401 Westbound Off-Ramp, and Intersection 13, at Courtneypark Drive and the Highway 410 Northbound Ramp, as these two intersections, while not ranked within the top six intersections in the base scenario, frequently ranked in the top six once LCVs are introduced. In fact, Intersection 13 surpassed all other intersections and experienced the most freight conflicts at the 50%, 75% and 100% LCV penetration levels.

Interestingly, one trend amongst the high-conflict intersections is that they mimic the aggregate trends seen in the TTC and PET analyses. In the TTC and PET analyses, large increases in the proportion of the most severe collisions are seen when increasing the LCV penetration level from 0% to 25%, followed by marginal increases in the most severe conflict proportions at 50%, 75%, and 100%. In some of the highest-conflict intersections, a similar trend is observed, where Intersections 2, 3, 4, and 13 see a large increase in their respective share of the conflicts between 0% and 25% LCVs. Following the 25% LCV level, only marginal increases or decreases in conflict proportions are seen for these four intersections. Meanwhile, Intersections 11 and 12 do not follow this trend. Intersection 12, which is the intersection with the biggest share of conflicts in the base scenario, sees a general increase in conflict frequency but a decrease in conflict proportion as other intersections become more unsafe. Lastly, Intersection 11 sees inconsistent increases and decreases in conflict frequency and conflict percentage when LCVs are incorporated into the freight traffic. These decreases are surprising when taking into consideration that this intersection is one of the worst-performing intersections in the AutoTurn analysis. Recall that at the Shawson Drive-Courtneypark Drive intersection, the turnpike double fails to complete the left turn at this intersection regardless of the number of lanes available on the departing or destination road. Therefore, it is surprising that an increase in the LCV penetration level did not yield consistent increases in conflict frequency or percentage and thus requires closer analysis.

Another interesting trend based on conflict frequency is the fact that the six highest-conflict intersections are all major arterial intersections. These intersections each contain a combination of highway off-ramps (Highway 401 and Highway 410), arterial roads (Dixie Road, Courtneypark Drive, Britannia Road) or collector roads (Shawson Drive, Tomken Road). This is expected, as arterial intersections are the areas of the road network where road users are the most concentrated, while road lanes are tightly packed into the available right-of-way. Meanwhile, the four minor intersections in the network, namely Intersections 7, 8, 9, and 10 which are only composed of a collector road (Shawson Drive) and three local roads (Westport Crescent, Tristar Drive, and Meyerside Drive), never rank in the top six across any of the LCV penetration levels. Of course, because this analysis is solely based on conflict frequency, much of this can be attributed to the exposure of these intersections. In other words, the minor intersections are not expected to have a high frequency of conflicts simply because fewer freight left turns are occurring on these roads, while the major arterial intersections have the highest volumes of freight traffic and experience the most freight left turns. Therefore, before any final decisions can be made regarding the high-conflict intersections in the network, the intersections must also be analyzed with respect to the exposure of freight traffic at each intersection.

In addition to finding the most concerning intersections based only on conflict frequency, this same process was followed but instead using exposure as the primary indicator. This means that the ratio of freight conflicts per freight left turn at each of the thirteen intersections is determined using the following equation,

Equation 6: $$\lambda_{Int} = \frac{C_{freight}}{L_{freight}}$$

where λ is the exposure ratio of an intersection (freight conflicts per freight left turn), C is the number of freight left-turning conflicts taking place at that intersection, and L is the total number of freight left turns completed at that intersection. This freight conflict-to-left-turn ratio is calculated individually for each of the thirteen intersections across all five LCV penetration levels, as shown in *Table 22*. In keeping consistent with the previous steps, the six intersections with the highest conflict-to-left-turn ratio are highlighted with bold text. The number of freight left turns at each intersection can be located in *Appendix B.6*.

Table 22: Intersection Conflict Summary by Exposure

LCV Penetration Level	0%	25%	50%	75%	100%
Intersection 1	11 (0.080)	28 (0.241)	22 (0.220)	**54 (0.600)**	54 (0.740)
Intersection 2	1 (0.009)	**49 (0.533)**	41 (0.554)	**92 (1.195)**	**111 (2.265)**
Intersection 3	**27 (0.126)**	**115 (0.608)**	**141 (0.844)**	**210 (1.005)**	**181 (1.708)**
Intersection 4	**19 (0.083)**	**79 (0.387)**	**134 (0.788)**	**210 (0.991)**	**253 (2.219)**
Intersection 5	**8 (0.131)**	11 (0.193)	23 (0.479)	23 (0.274)	42 (1.500)
Intersection 6	3 (0.033)	20 (0.253)	54 (0.761)	68 (0.425)	70 (1.273)
Intersection 7	1 (0.022)	7 (0.149)	6 (0.214)	16 (0.127)	18 (0.720)
Intersection 8	3 (0.068)	4 (0.105)	7 (0.269)	10 (0.152)	10 (0.500)
Intersection 9	**4 (0.114)**	12 (0.333)	**26 (0.867)**	24 (0.194)	34 (1.545)
Intersection 10	1 (0.031)	1 (0.029)	8 (0.333)	5 (0.057)	12 (1.000)
Intersection 11	**57 (0.934)**	**32 (0.653)**	**91 (1.820)**	88 (0.478)	**73 (2.355)**
Intersection 12	**97 (0.614)**	**122 (0.992)**	**132 (1.200)**	**147 (0.710)**	**180 (2.278)**
Intersection 13	4 (0.013)	**104 (0.395)**	**162 (0.743)**	**235 (0.867)**	**252 (1.726)**

*Each entry is written as the number of freight conflicts followed by the freight-conflict-to-left-turn ratio, λ, in brackets
** Bold indicates the intersection is amongst the six highest-conflict intersections for that LCV penetration level

It should be noted that the calculated decimals seen here are no longer percentages, as there are situations where one freight left turn results in multiple traffic conflicts. Instead, these values are represented as a ratio of the number of freight conflicts that occur during each freight left turn.

For example, at 100% LCV penetration in Intersection 11, each freight left turn results in 2.355 freight traffic conflicts.

Based on freight conflicts per freight left turn, many trends previously identified during the conflict frequency analysis are repeated. Using this metric, the analysis once again points to the arterial Intersections 2, 3, 4, 11, 12, and 13 as the highest-conflict intersections, as these intersections rank most often amongst the six intersections with the highest conflict-to-left-turn ratios when LCVs are gradually introduced into the network. However, unlike in the previous analysis where a large increase in conflicts was observed between 0% and 25% LCVs followed by marginal increases at subsequent LCV levels, this exposure analysis sees gradual growth of the conflict-to-left-turn ratio across all intersections. Furthermore, in comparison to their base scenario, Intersections 2 and 4 saw the largest increase in their ratios, increasing from 0.009 and 0.083 freight conflicts/left turn to 2.265 and 2.219 freight conflicts/left turn respectively. In reinforcing the idea that Intersections 2, 3, 4, 11, 12, and 13 are the most concerning intersections with respect to conflicts as LCVs are introduced into the network, a closer analysis of each intersection is performed to determine the reasoning behind these safety issues. To closely analyze these intersections, the TTC, PET, and conflict types across each LCV penetration level are reported in tables similar to those used in the aggregate results. These tables can be found in *Appendix B.7* of this report.

7.2.2 – Intersection 2 Analysis: Dixie Road and Highway 401 Westbound Off-Ramp

Intersection 2, at Dixie Road and the Highway 401 Westbound Off-Ramp, is amongst the most conflicting intersections, ranking within the top six in six out of ten instances between the frequency and exposure analyses. In reviewing the trends seen at Intersection 2, it can be observed that the TTC and PET trends at this intersection are generally consistent with the aggregate trends. While it should be noted that the sample size of conflicts at the 0% LCV penetration level limits the relevancy of that level's results, the severity of conflicts with PETs and TTCs of 0 seconds still experience a major increase from the 0% LCV level to the 25% LCV level, followed by very marginal changes at the subsequent levels. However, the conflict type trend sees a different trend from the aggregate results. Recall that in the aggregate results, as LCVs are added into the network, rear end conflicts initially increase significantly and then generally level out with crossing conflicts as more LCVs are added to the network. Additionally, both base scenario crossing and rear end conflicts significantly outweigh the remaining proportion of lane changing conflicts. However, in Intersection 2, the collision type distribution is skewed heavily in favour of rear end conflicts, with proportions of at least 64% across all LCV penetration levels. This dominant conflict type is then followed by lane changing conflicts, which make up most of the remaining conflict proportion, while almost no crossing collisions occur in the network at all LCV penetration levels.

Reviewing the freight conflict data from SSAM and filtering by link, the most common network links that are involved in conflicts are determined and located in Vissim. By narrowing down the locations in Vissim, the simulation is observed in real time to determine the reasoning behind these conflicts. In doing this, it is found that the vast majority of conflicts are occurring in the double left-turn lanes, which turn from the westbound direction on the Highway 401 Off-Ramp to the southbound direction on Dixie Road. This phenomenon is not surprising, as conflicts are occurring as freight vehicles, including LCVs, are required to use the outer left-turning lane. The lane changing conflicts are typically occurring in the lanes leading up to the double left turn, as the LCVs are attempting to move out of the inner turning lane. However, many freight vehicles eventually run out of space and time to change lanes safety without conflicting with another vehicle. Then as the vehicles are in the outer left-turn lane, rear end conflicts occur as a result of the LCVs encroaching into the inner turning lane due to the shape of their swept path envelope, causing conflicts with the vehicles travelling in that lane. Ultimately, these rear end conflicts during the double left turn occur because the LCVs fail to make use of the additional road space available to them when turning. Despite the fact that the freight vehicle’s lateral driving behaviors are adjusted in the Vissim simulation during the double left turn to maintain additional distance from the inner lane, the LCVs are still incapable of oversteering to completely avoid the inner turning lane.

7.2.3 – Intersection 3 Analysis: Dixie Road and Shawson Drive

The next intersection to be analyzed in comparison to the aggregate trends is Intersection 3, which takes place at Dixie Road and Shawson Drive. This intersection ranked as one of the worst intersections at every LCV penetration level, regardless of frequency or exposure. Similar to Intersection 2, the results for both the TTC and PET measures are consistent with the aggregate trends. For both of these measures, the most severe conflicts, with TTC and PET values equal to 0 seconds, see a major increase as LCVs are incorporated into the network for the first time at the

25% penetration level. Following the 25% LCV penetration level, each following level results in marginal increases or decreases for the most severe conflicts. Additionally, in observing the trends for the type of conflict occurring, Intersection 3 is also generally consistent with the aggregate results, where the addition of LCVs into the network gradually leads to rear end collisions becoming the dominant type of collision, followed closely by crossing collisions. Meanwhile, lane changing conflicts are the least common type of conflict. In Intersection 3, the same trend occurs, but with rear end collisions being slightly more dominant at this intersection than in the aggregate scenario.

In examining the links involved in each freight conflict in SSAM, it appears that the most common conflicts are again occurring at a double left-turn lane, which takes place when turning left from the westbound direction along Shawson Drive to the southbound direction on Dixie Road. In these instances, the LCVs are required to use the outer left-turn lane of the double left turn. However, due to the LCV's large turning envelope and inability to oversteer, the body of the LCV encroaches into the inner turning lane, resulting in rear end collisions with other vehicles in that lane. While this accounts for the rear end conflicts, the other three single left-turn lanes account for the high proportion of crossing conflicts in the intersection. In these cases, an LCV attempts a left turn at any of the single left-turn lanes. However, due to the size of the LCV's turning envelope and inability to effectively oversteer in Vissim, the vehicle's body cannot complete the turn without encroaching into other lanes of traffic. As a result, the body of the LCVs crosses into the oncoming lanes adjacent to the receiving lanes, leading to the high proportion of crossing conflicts with the oncoming vehicles.

7.2.4 – Intersection 4 Analysis: Dixie Road and Britannia Road

Intersection 4, at the intersection between Dixie Road and Britannia Road, serves as another intersection to be closely studied, ranking within the top six most conflicting intersections at every LCV penetration level based on conflict frequency and exposure. Based on the two surrogate measures of safety, TTC and PET, this intersection's results closely follow the results generated for the aggregate data. While the 0% LCV level results for Intersection 4 indicate that this intersection is safer than the 0% aggregate results due to much lower proportions of conflicts with TTC and PET values of 0 seconds, the same trend takes place where a sharp increase in conflict severity occurs at the 25% LCV level. This is again followed by marginal increases at each subsequent LCV penetration level. On the other hand, the conflict type trend at Intersection 4 differs from the aggregate trends. At this intersection, crossing collisions are clearly the dominant type of conflicts, making up at least 61% of the conflicts across all LCV penetration levels. This is followed by rear end conflicts as the next most common type, followed by lane changing conflicts as the least common type.

In reviewing the links where conflicts are occurring during the simulation, it appears that the crossing conflicts occur most often when LCVs encroach into oncoming lanes during left turns. Due to the size of the swept path envelope of the LCV during left turns, crossing collisions occur when the body of the LCV passes into the oncoming traffic lane that is adjacent to the receiving lane, conflicting with the oncoming vehicles in those lanes. The exaggerated proportion of crossing collisions at this intersection is likely caused by the tight dimensions of these arterial lanes, which leave much less room for these large vehicles to maneuver without interfering with other road users. At the same time, the inability of these vehicles to effectively complete an oversteering maneuver prevents them from utilizing additional space available in the intersection that would prevent the body of the vehicle from dragging into oncoming lanes. Occasionally, some crossing conflicts occur when an LCV begins their left turn at an intersection, but because longer times are needed for the LCV to clear the intersection, oncoming through vehicles are forced to a stop as the LCV is still turning. While many of these crossing conflicts are much less severe with higher PETs and TTCs, they do point to a potential issue with the time needed for LCVs to clear larger intersections, as well as the need to adjust signal timings to accommodate these longer clearance times.

Furthermore, the rear end conflicts recorded in this intersection are largely due to the presence of a double left-turn lane. Again, the body of the LCV travelling in the outer left-turn lane conflicts with the road users travelling in the inner left-turn lane, leading to numerous rear end conflicts. Although, it should be noted that the double left-turn lane leads to fewer crossing conflicts during these turns, as the vehicle of the body does not pass into oncoming lanes where it can conflict with oncoming road users.

7.2.5 – Intersection 11 Analysis: Shawson Drive and Courtneypark Drive

Intersection 11, which encompasses the intersection between Shawson Drive and Courtneypark Drive, is another intersection of interest analyzed, as this intersection ranks in the top six most conflicting intersections nine out of ten times based on either exposure or conflict frequency. Interestingly, this is the only intersection reviewed where each of the PET, TTC, or conflict type trends are inconsistent with the aggregate results. Starting with the TTC and PET results, both surrogate measures of safety are heavily skewed towards the highest conflict severity level, as over 90% of the conflicts have a PET or TTC equal to 0 seconds across all LCV penetration levels. As a result, the inclusion of LCVs at this intersection has very little impact on the freight conflict severity. With respect to the conflict type, the vast majority of conflicts are crossing conflicts, with at least 89% of the conflicts classified as crossing conflicts throughout all LCV penetration levels. Meanwhile, rear end conflicts make up the remaining proportion of conflicts, while lane changing conflicts do not factor into the final proportions. With the introduction of LCVs from 0% to each subsequent LCV penetration level, the proportion of conflict type hardly changes, with crossing conflicts remaining dominant. This could indicate that the presence of LCVs does not significantly impact the left-turning safety of this intersection.

Looking more closely at the intersection links where the conflicts are taking place as identified by SSAM, some conclusions can be reached regarding the reasoning behind these conflicts. In this intersection, only two left turns are possible; northbound on Shawson Drive onto westbound on Courtneypark Drive, and westbound on Courtneypark Drive onto southbound on Shawson Drive. This is because in the data provided by the City of Mississauga and Peel Region, no freight vehicles were recorded travelling into or out of the north leg of this intersection. Therefore, no freight traffic turns left into or out of this road, leaving only two possible left turns. In reviewing the conflicts occurring, almost all of the conflicts are occurring when turning from the northbound direction on Shawson Drive onto the westbound direction of Courtneypark Drive. In these turning scenarios, the left-turning freight vehicles are involved in crossing conflicts with vehicles in the oncoming lane adjacent to the receiving lanes. Again, this could be due to the limited maneuvering space available in last-mile arterial intersections. As previously determined in the AutoTurn analysis, the size of freight vehicle swept paths requires a significant amount of additional space on the receiving road to avoid conflicts between the trailer of the vehicle and other road lanes. In arterial roads designed with tight geometric dimensions, this often leads to freight conflicts as the vehicle passes through the lanes adjacent to their destination lane. In reviewing the simulation, another key issue that leads to these conflicts at this left turn is the signal timing of the intersection. In the northbound direction, the amber time is very short and does not allow enough time for the large LCV to clear the intersection during its turn. As a result, this can leave the LCV stranded momentarily within the intersection, causing conflicts with oncoming traffic. Lastly, another issue leading to conflicts during this left turn is traffic congestion on Courtneypark Drive in the eastbound direction. Using the 2022 adjusted traffic volumes combined with the provided signal timings, the simulation experiences traffic congestion from Intersection 12, at Courtneypark Drive and Tomken Road, backing up into Intersection 11. Because of this, vehicles turning left onto Courtneypark Drive, including LCVs, are again becoming stranded in the intersection and conflict with through vehicles.

7.2.6 – Intersection 12 Analysis: Courtneypark Drive and Tomken Road

Another intersection analyzed more closely is Intersection 12, at Courtneypark Drive and Tomken Road, which ranks amongst the high-conflict intersections across all LCV penetration levels based on both conflict frequency and exposure. In reviewing the TTC and PET results, both results are consistent with the aggregate results, as a large increase in conflict severity, where TTC or PET are equal to 0 seconds, occurs between the 0% and 25% LCV penetration levels. At each subsequent increase in LCV penetration, marginal increases are seen for the most severe TTC and PET values. Meanwhile, with respect to conflict type, the Intersection 12 conflict type results are somewhat consistent with the aggregate results, where rear end conflicts are more prominent than crossing conflicts, followed by lane changing conflicts. However, in this specific intersection, the proportion of rear end conflicts is much more prominent than in the aggregate results.

According to the SSAM analysis, the majority of the rear end collisions occur at this intersection's double left-turn lanes, which turn from the northbound direction on Tomken Road to the westbound direction on Courtneypark Drive. Like in the previous intersections, the encroachment of the LCVs into the inner left-turn lane leads to numerous rear end collisions with the vehicles in that lane. Furthermore, at the conventional left-turn lanes in this intersection, many of the crossing conflicts are again accounted for by the left-turning LCVs crossing into the adjacent oncoming lanes on the destination road due to the shape of the LCV's swept path envelope. Some other crossing conflicts are also due to the LCV taking longer to clear the intersection during their left turns, causing oncoming through vehicles to conflict with the LCV blocking the intersection. This is particularly an issue at the left turns that are protected-permitted and not fully protected, as the LCV elects to start its turn without effectively considering the oncoming vehicles. Lastly, the small number of lane changing conflicts occur in the links preceding left-turn lanes, as the LCVs moving into the left-turn lanes conflict with other vehicles in the same lane.

7.2.7 – Intersection 13 Analysis: Courtneypark Drive and Highway 410 Northbound

Lastly, the final intersection analyzed further is Intersection 13, which takes place between Courtneypark Drive, the Highway 410 Northbound Ramp, and a carpool parking lot. In comparing this intersection's TTC and PET trends to the aggregate trends, the trends for these surrogate measures of safety are consistent. Although the base scenario experiences few freight conflicts and ultimately limits the sample size of the data, the same general trend is observed where a significant increase in the most severe conflicts occurs between the 0% and 25% LCV penetration levels, followed by marginal increases at each subsequent LCV level. Meanwhile, the conflict type trend at this intersection is inconsistent with the aggregate trend. In this intersection, the most dominant collision type is rear end conflicts, with at least 73% of conflicts made up of rear end conflicts across each LCV level. Lane changing conflicts are then the next most common type of conflict, making up the remaining proportion of conflicts at greater than 23% for each LCV level. Meanwhile, crossing conflicts are very sparce and are mostly insignificant to the conflict type distribution. Furthermore, with the addition of LCVs between each LCV penetration level, the proportions of each conflict type remains largely unchanged.

Reviewing the links involved in each conflict, the double left-turn lane from the Highway 410 Off-Ramp onto Courtneypark Drive accounts for the majority of conflicts. Most of the conflicts at this double left-turn are rear end conflicts, where the body of the freight vehicle in the outer turning lane conflicts with the vehicles in the inner turning lane. At the same time, in the build up to the double left-turn lane, lane changing conflicts then occur when the LCVs attempt to move into the outer left-turn lane but run out of space to do so without conflicting with other vehicles. It is important to note that the findings of the AutoTurn analysis determined that this intersection was among the safer intersections for LCVs due to the additional space available to them, as the wider shoulders on the off-ramp and the painted medians provided additional space that could be used as the LCVs completed their left turn. However, the AutoTurn analysis only analyzed the left turn from the westbound direction onto the southbound direction. While very few conflicts are detected in this specific turn during the SSAM analysis, many conflicts are detected in the turn from the northbound direction to the westbound direction, which is not tested in AutoTurn.

7.2.8 – Concluding Remarks

In closely examining the six high-conflict intersections, it is clear that each individual intersection is subject to its own unique conditions that lead to the conflict patterns experienced at each location. However, there appears to be some similarities between the conflict trends and the reasons behind them. First, one clear trend that exists amongst these intersections is the increase in conflict severity as LCVs are added to the network. In each of the intersections studied, the conflict severity based on TTC or PET experienced a major increase between the 0% and 25% LCV penetration level. At higher LCV penetration levels, this high conflict severity is maintained and marginally increases further. This conflict severity can be directly attributed to the use of LCVs due to such a drastic increase from the 0% base scenario. The next trend amongst the high-conflict intersections is the prevalence of rear end conflicts when LCVs utilize double left-turn lanes. In five of the six intersections considered for closer analysis, a double left-turn lane is present in at least one leg of the intersection. As LCVs are added to these intersections, both the frequency of rear end conflicts and proportion of total conflicts made up of rear end conflicts increase significantly. This is as a result of the trailers of the LCVs in the outer turning lanes encroaching into the inner turning lanes, causing rear end conflicts with the other vehicles in those lanes. In each of these intersections, this leads rear end conflicts to become one of the dominant conflict types. At the same time, another trend amongst the high-conflict intersections is the prevalence of crossing conflicts where conventional single-lane left turns are attempted. In these situations, the body of LCVs attempting a left turn will partially pass through the oncoming lane adjacent to their destination lane. In doing so, crossing conflicts occur with the vehicles in those lanes, increasing the frequency and proportion of crossing conflicts at these intersections. Furthermore, crossing conflicts also occur in some intersections where the signal timings or simply driver behavior leave the LCVs limited time to clear the intersection before oncoming traffic arrives at the same location.

In reviewing these trends between the high-conflict intersections, there appears to be one common theme uniting these conflict trends. That is, the underlying issue amongst these trends is the inability of the existing infrastructure to accommodate the size of the LCV during a left turn. In each of these conflict scenarios where LCVs lead to a significant increase in different types of conflicts, the size and shape of the LCV's swept path envelope in relation to the available infrastructure plays a key role in causing the conflicts. In the double left-turn lanes, the width of the two left-turn lanes is not adequate for a left-turning LCV, as the width of the swept path envelope throughout the left turn causes the trailers of the LCV to inadvertently pass into the inner turning lane. This leads to the rear end conflicts between the vehicles in the inner lane and the rear trailers of the LCVs. Meanwhile, as the LCV completes left turns at conventional single left-turn lanes, the increased width of the turnpike double's turning envelope as it enters the destination lane causes the body of the vehicle to pass through adjacent oncoming lanes. This then causes the crossing conflicts identified by SSAM between the LCV and the oncoming vehicles. Thus, geometric design not only leads to LCV conflicts with infrastructure, but can also have a direct impact on LCV conflicts with other road users. Overall, these findings reinforce the conclusions derived by the swept path analysis previously completed in AutoTurn, where it was determined that the road infrastructure as it is currently designed fails to accommodate the turning envelope of LCVs. In that previous analysis, the same issues occurred within the double left-turn lane and with the oncoming lane at conventional left turns. It was only after significant increases to the dimensions of the lanes and changes to the medians that LCVs could safely complete their left turns.

Of course, it is important to consider the limitations of Vissim in this specific analysis. In particular, the inability of Vissim to effectively replicate an oversteering maneuver and make use of additional space in the intersections could have contributed to the results. By not simulating a wider turn at the double left turns, it is not known whether the conflicts with the inner turning lane could have effectively been avoided. Meanwhile at the conventional left turns, wider left turns have the potential to avoid the oncoming lane entirely. In the AutoTurn simulation, making use of additional space in the intersections did allow for the LCV to potentially avoid the inner left-turning vehicles and the adjacent oncoming lanes. However, in many of the scenarios in AutoTurn, avoiding these conflicts typically resulted in other conflicts arising on the outside of the left turns, such as conflicts with other adjacent lanes on the destination road or the curb of the road due to an inadequate right-of-way width. Therefore, the results generated here should still have merit, as oversteering during left turns in the AutoTurn simulations did not necessarily eliminate the conflicts experienced when navigating the existing road infrastructure.

Chapter 8. Conclusion

8.1 – Summary

In conclusion, this thesis studied the safety considerations of LCVs as they complete left-turning maneuvers in last-mile settings. Due to the many benefits LCVs offer for the environment and the economy, many jurisdictions are considering expanding permissions to use LCVs on their road networks. However, the unique size and driving characteristics of LCVs calls into question their safety when operating alongside other road users on major arterial roads. Therefore, extensive research into the safety implications of these vehicles is required before expanding their usage in urban areas.

Upon a review of previous literature on the subject, it was found that limited research has been performed on LCV safety in last-mile urban intersections. Due to heavy restrictions on urban arterial LCV usage, much of the existing research focused instead on areas where LCVs have previously been allowed, including on highways and rural roads. Meanwhile, a previous study into LCV turning safety in urban intersections focused solely on right-turning maneuvers and related particularly to conflicts between LCVs and the existing urban infrastructure geometry, while leaving out safety considerations involving conflicts between LCVs and other road users. Past literature often relied on historical collision data, which tends to suffer from sample size and contextual issues. As a result, this thesis fills previous research gaps by focusing on left-turning maneuvers of LCVs in last-mile areas. Using different types of simulations, safety conflicts are investigated between LCVs and infrastructure as well as with other road users.

A study area was selected in a freight-heavy section of the Region of Peel, bordered by two highways and two major arterial roads. Within this study area, a sample LCV route was devised composed of thirteen intersections, including six intersections where the LCV will complete a left turn. For these intersections, traffic volume and signalization data were provided by the City of Mississauga and the Region of Peel, while missing data points were determined using various methods. Additionally, the turnpike double, which is amongst the largest of the LCV types, was selected as the control vehicle in this thesis.

The first of the two analysis methods attempted is the swept path analysis, which analyzes the conflicts experienced between the body of the LCV and the available infrastructure geometry during left turns. AutoTurn software is used to observe the swept path envelope of the turnpike double as it turns within the intersections drawn in AutoCAD. At each intersection, four scenarios were tested based on permissions to use additional lanes on either of the departing or destination roads, with IntelliPath determining the optimum turning path and assigning a passing or failing grade in each scenario. An additional sensitivity analysis was also performed for each scenario, where the passing grade status was monitored when changes to the lane widths and setback distances were completed. Lastly, a statistical significancy test was performed using NLOGIT software to determine the significancy and elasticity of several intersection variables with respect to each scenario's grade status.

Meanwhile, a safety performance evaluation was performed using PTV Vissim. A microsimulation model of the thirteen-intersection network was developed and calibrated. LCVs were then added into the network, where 0%, 25%, 50%, 75%, and 100% of existing freight vehicles in the network were replaced by turnpike doubles. In running the network simulation at the five LCV penetration levels, the generated trajectory files were transferred into SSAM, which assesses conflicts between road users in the network. SSAM was then used to analyze trends in freight conflicts as the LCV penetration level increases in the network, reporting on the frequency, type, TTC, and PET of conflicts. The six highest-conflict intersections were then analyzed further to determine their intersection-specific conflict trends.

From the AutoTurn analyses, it was found that LCVs are often unable to use existing intersections safely, especially when restricted to one lane on both the departing and destination roads. In the most restrictive scenario, where only one departing and destination lane are permitted for use, the turnpike double failed to safely complete the turn almost every time due to conflicts with adjacent road lanes, curbs, and raised medians. Meanwhile, the scenarios where additional lanes were available for use resulted in more passing grades, with an additional lane on the destination road being an important factor in increasing passing grades. Furthermore, in completing a sensitivity analysis of the lane widths, the importance of the destination lane widths was further reinforced. Increases to the destination lane widths were more successful in increasing the number of passing grades than lane width increases on the departing roads due to the shape of the LCV's swept path envelope. At the same time, many scenarios required unrealistically large lane width increases to result in passing grades, further indicating that major changes are required to allow existing intersections to accommodate LCVs. Setback distance sensitivity analyses also resulted in similar results, where setback distance increases were successful in changing failing grades into passing grades, but these successes often required unrealistically large setback distances. This was particularly true for the scenarios that restricted access to an additional lane on the destination road.

Using Vissim and SSAM to analyze freight conflicts, results were first generated as an aggregate of the entire network, and then as results for the six highest-conflict intersections. For the aggregate results, it was found that the addition of LCVs into the network led to a major increase in freight conflicts, with extreme increases at each LCV penetration level. Both aggregate TTC and PET values experience a major increase in freight conflict severity at the 25% LCV penetration level, followed by marginal increases at each subsequent level. Additionally, it was found that rear end conflicts became the most prominent conflict type when LCVs were added into the network, followed closely by crossing conflicts. Further analyses were conducted on the six intersections with the highest conflict levels. A closer review of where the specific conflicts were occurring for each intersection further validated the findings of the AutoTurn analyses. One of the most common issues took place during double left turns, where the body of the LCV in the outer turning lane inadvertently passes into the inner turning lane, leading to a significant number of rear end conflicts. The other most common issue was at conventional left-turning lanes, where an LCV would encroach into the adjacent oncoming lane on the destination road during a left turn, leading to crossing conflicts with vehicles in the oncoming lane. Both of these intersection-level trends further reinforce the idea from the AutoTurn analysis that intersection geometry as it is currently designed is not equipped to effectively accommodate LCVs. Each of these common issues between intersections is rooted in the fact that the size and width of the LCV’s swept path are too large to maneuver within the limits of last-mile intersections. This indicates that insufficient geometric designs not only lead to conflicts with the road infrastructure, but also increases the frequency and severity of conflicts with other road users.

8.2 – Limitations

In reviewing the results of this study, there are some limitations to this thesis that should be kept in mind. One key limitation relates to the limitations of the software programs used in this project. In using AutoTurn, and in particular IntelliPath, one issue relates to the sensitivity of the grade status based on the selected starting position of the model vehicle. In that analysis, slight changes in the starting position and orientation of the vehicle could impact the optimum route selected by IntelliPath and ultimately change the grade status of each attempted turn. Between each intersection and each of the four scenarios assessed in this thesis, the starting positions within each of the lanes were kept moderately consistent while the orientation was always selected as parallel to the nearest curb or median. Additionally, each intersection and scenario was tested multiple times with different starting positions to find the most common optimum path and grade status. These choices were ultimately made to minimize the volatility of IntelliPath's results and overcome this limitation. Future assessments using IntelliPath should be weary of this limitation and attempt to follow suit in minimizing this volatility.

With respect to the Vissim simulation, a main limitation of this software is its ability to accurately represent a real-world traffic network. In a realistic traffic network, there are many additional external factors that can affect driver behavior and safety. Driver visibility and vehicle braking capabilities can be impacted by various weather and lighting conditions. Meanwhile, other factors like human error, driver fatigue, impaired driving, and illegal driving maneuvers each have adverse impacts on traffic network safety. Unfortunately, the model produced here does not address all of these factors, as Vissim's ability to replicate these additional factors is limited, oversimplified, or non-existent. There are further questions to be asked about the safety of LCVs in a last-mile area when in the presence of poor weather conditions or unsafe human characteristics that are not addressed in this thesis. However, many of these additional factors remain outside the scope of this thesis and are not investigated as a result. Furthermore, it was previously discussed that the Vissim software does not accurately replicate the path of an LCV if an oversteering maneuver is attempted. As a result, the freight vehicles may have had additional space in different parts of the intersection that they were not utilizing and instead limited themselves to the Vissim links that were drawn manually. Even through Vissim was superimposed onto an AutoTurn drawing of an LCV left turn to validate the default turning maneuver of the LCV, oversteering still would have provided an alternative for the LCVs to attempt while completing their turns. While it is not guaranteed that this additional space would have improved the safety conflicts recorded in SSAM, a more accurate representation of the LCV's swept path in Vissim would likely benefit the results of future endeavours simulating LCVs in Vissim.

Lastly, another limitation of the thesis completed here relates to the geographical context of the study. As discussed in the scope, the entirety of this thesis relates to the context of Ontario, with design guidelines referenced, data collected, and assumptions made all pertaining to the Province of Ontario. Therefore, the findings of this study may be relevant within a Canadian context, but one must be weary of applying the findings of this thesis in vastly different traffic contexts, such as in Asia or Europe, without further study into how these contexts may change the safety considerations of LCVs.

8.3 – Applications

In completing this thesis, there are many real-world implications of the findings reported here. This thesis sheds light on the many safety concerns LCVs pose when turning left on last-mile arterial and collector roads, including conflicts with infrastructure and with other road users. With many jurisdictions considering expanding permissions for LCVs to operate on their roads, the information provided by this thesis can assist with the future planning of these roads to accommodate these vehicles. An examination of the existing road infrastructure could be completed along the lines of this thesis' methodology to analyze potential conflicts that may arise at different intersection types. With the increased prominence of crossing conflicts and rear end conflicts during certain intersection layouts when LCVs are added into a road network, jurisdictions can plan their roads to avoid these dangerous contexts. This can be achieved by planning for roads in areas expected to have high levels of LCV activity in order to provide additional space for LCVs to maneuver. This additional space can be provided by increasing road right-of-way widths, implementing further setback distances for oncoming vehicles stopped at an intersection, or expanding permissions of LCVs to utilize additional lanes at different parts of the intersection. Overall, these findings relating to conflicts with infrastructure and other road users could contribute to the development of updated road design guidelines to ensure that future road designs are in line with the latest developments in LCV safety research.

Furthermore, with an increased understanding of LCV safety in last-mile trips, another application of this thesis relates to the education of prospective LCV operators. As businesses consider expanding their freight operations to rely on LCVs, the potential operators should be educated about the unique maneuvering characteristics and safety concerns posed by LCVs.

Ultimately, these drivers should be trained on the effects of using additional lanes of the road to safely complete left turns while being aware of possible safety conflicts with other road users.

8.4 – Areas of Future Research

Moving forward, there are many areas in which this research should be continued to further develop an understanding of the safety implications of LCV operation in last-mile contexts. First, one area of future works could relate to understanding the impacts that traffic signalization has on the last-mile safety of LCVs. Recall that in the intersection-level SSAM analysis, there were specific intersections where the signal timings at the intersection appeared to play a role in leading to conflicts between LCVs and other road users. The size of LCVs and their maneuvering characteristics inevitably cause them to take much longer to clear an intersection, putting them at risk of conflicting with other users at the intersection. As a result, further study should be conducted into the impacts of signalization on LCV safety and perhaps determine an approach to optimize traffic signals to minimize conflicts.

Another potential area of future research relates to further exploration into conflicts between LCVs and vulnerable road users. This includes an investigation into how LCVs impact the safety of cyclists and pedestrians when travelling alongside one another in a last-mile area. As discussed in the literature review, pedestrians and cyclists are the most vulnerable users when involved in a conflict with a motorized vehicle. This is because freight-pedestrian and freight-bicycle collisions are often very severe due to the minimum safety protections equipped by these transportation modes and the relative speeds of each mode. Moreover, with the extensive blind spots in freight drivers' field of view, and the further exaggerated blind spots of LCVs, smaller modes of travel are easily hidden within these blind spots, leading to a higher risk of collision with cyclists or pedestrians. The study area selected for this thesis had very low levels of pedestrian and bicycle activity, so opportunities were scarce to assess the safety impacts on these modes based on such a limited sample size. Therefore, further research must be conducted into the safety of pedestrians and cyclists in the presence of LCVs, especially as major arterials continue to be transformed to meet the needs of safety initiatives like complete street designs or Vision Zero.

Chapter 9. References

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Appendix

Appendix A – Swept Path Modelling

A.1 – AutoCAD Drawings of Each Intersection

The following figures are images of the AutoCAD drawings of each left-turning intersection in chronological order.

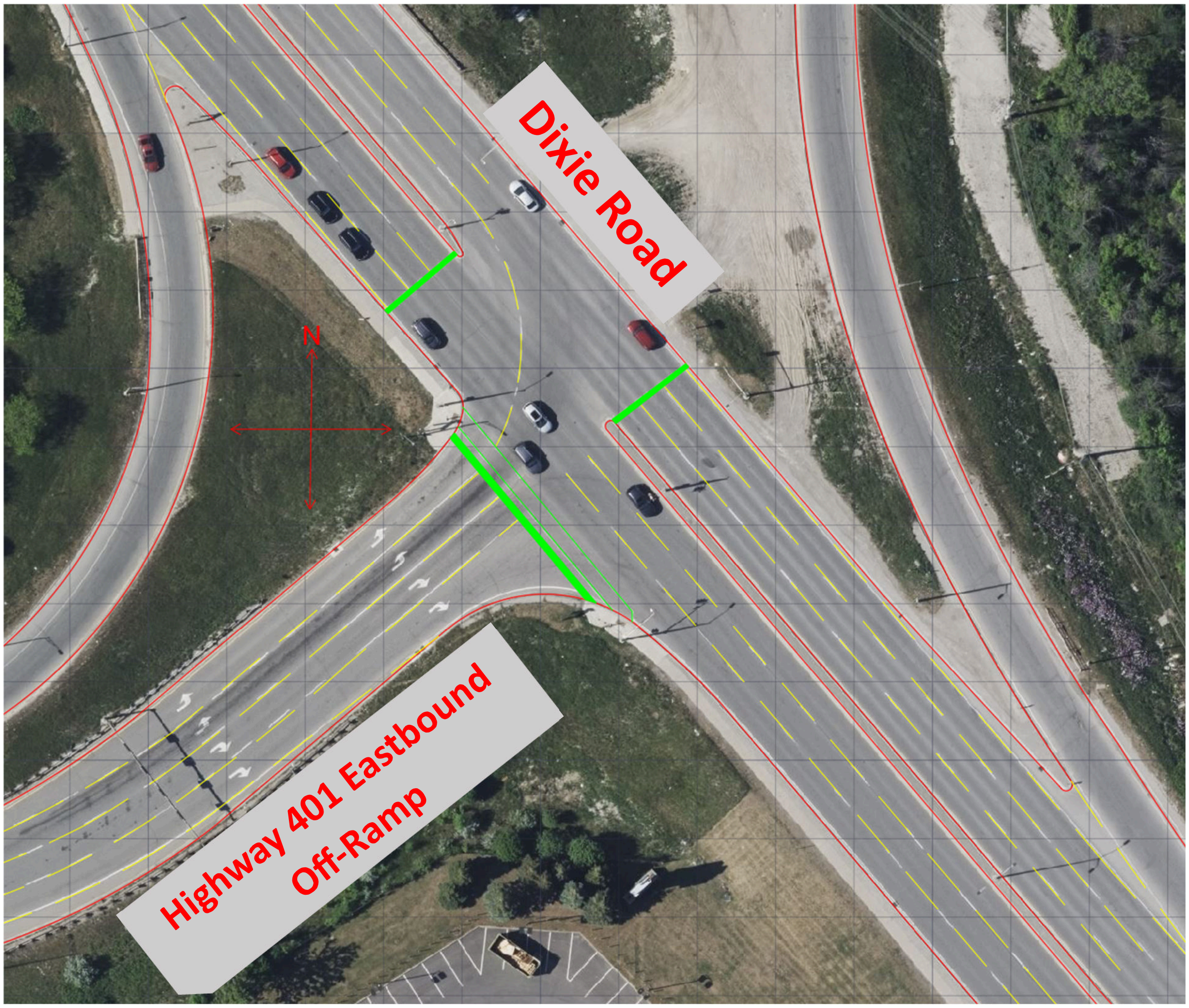

Figure 35: AutoCAD Drawing of Dixie Rd - Hwy 401 Off-Ramp

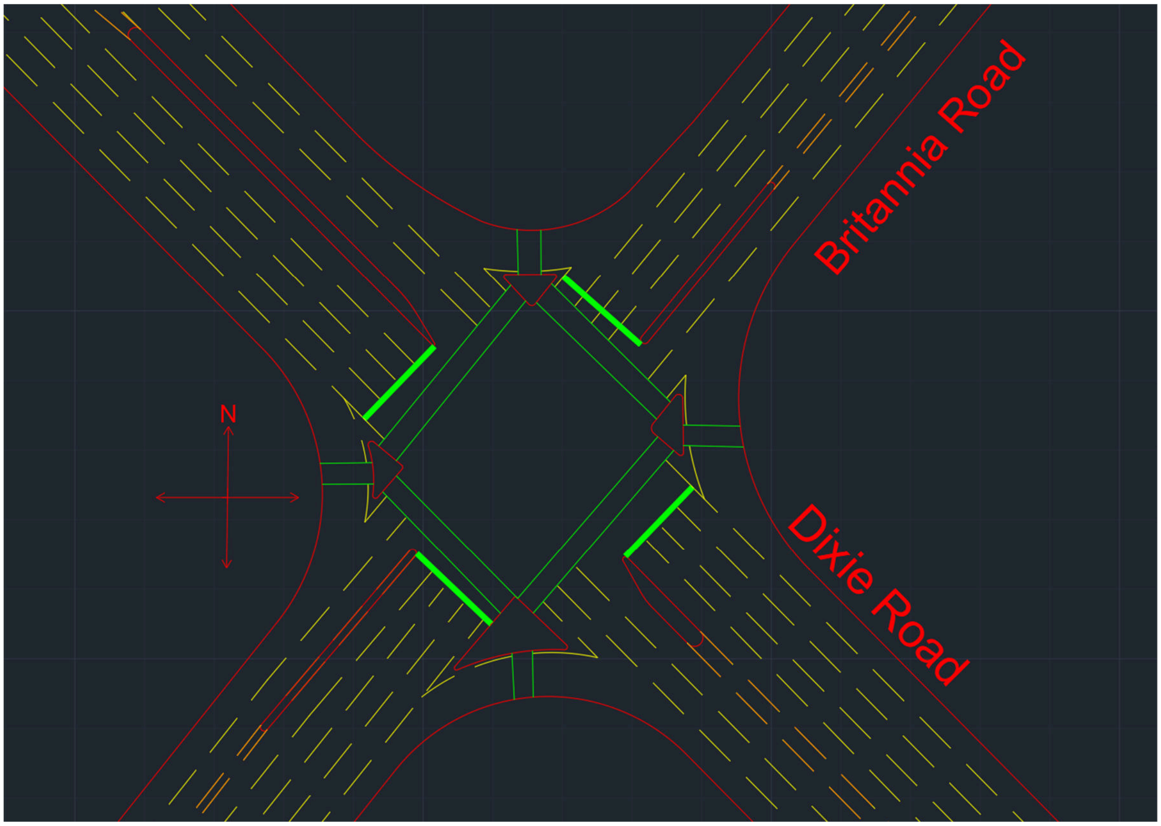

Figure 36: AutoCAD Drawing of Dixie Rd - Britannia Rd

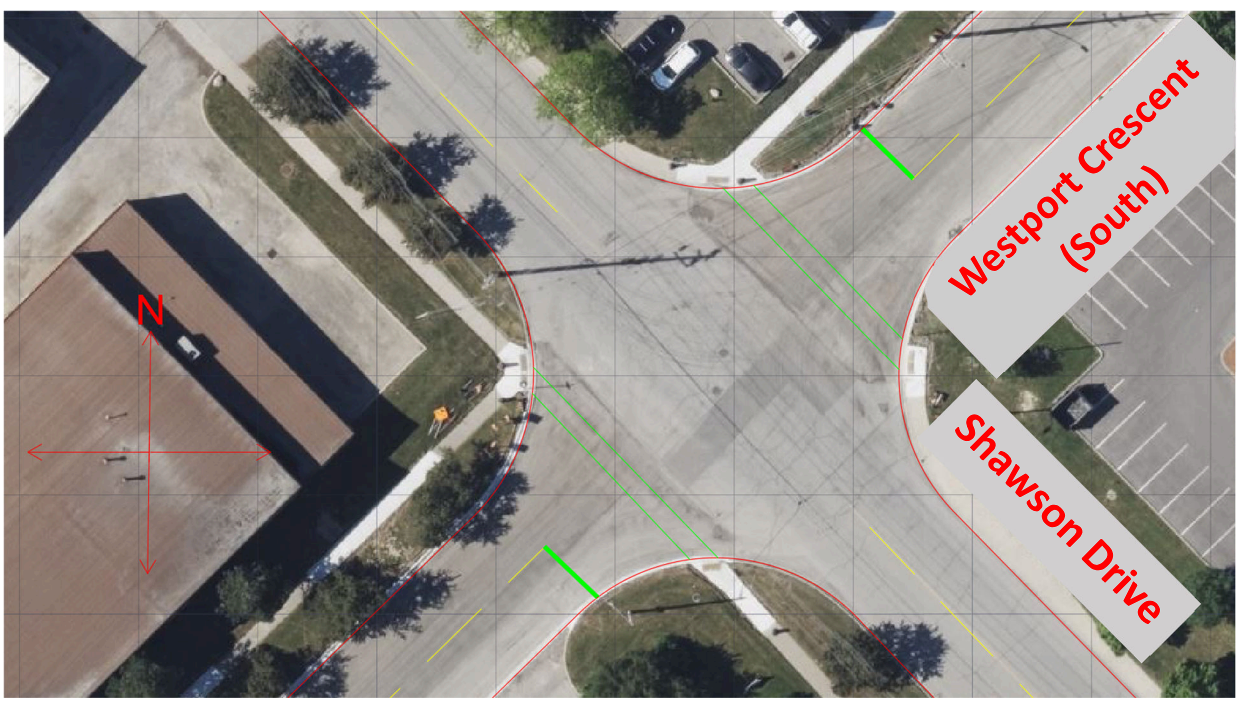

Figure 37: AutoCAD Drawing of Shawson Dr - Westport Cr S

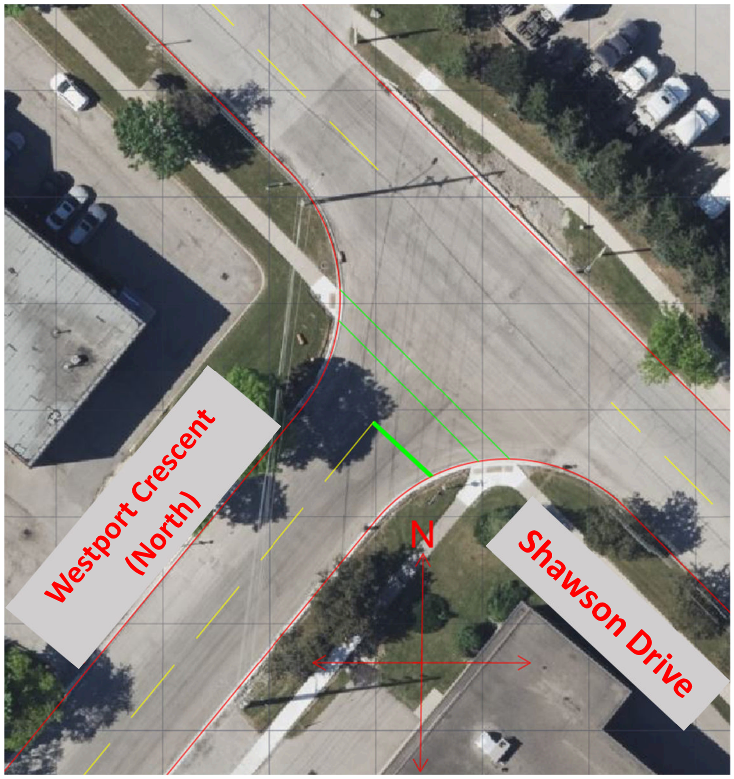

Figure 38: AutoCAD Drawing of Shawson Dr - Westport Cr N

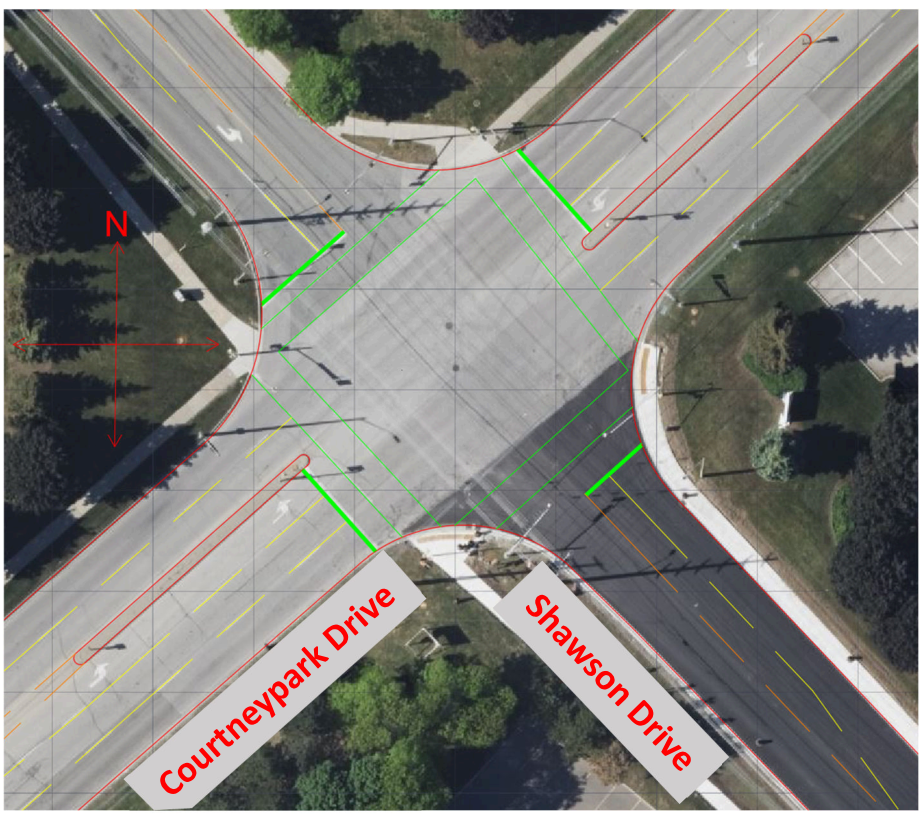

Figure 39: AutoCAD Drawing of Shawson Dr - Courtneypark Dr

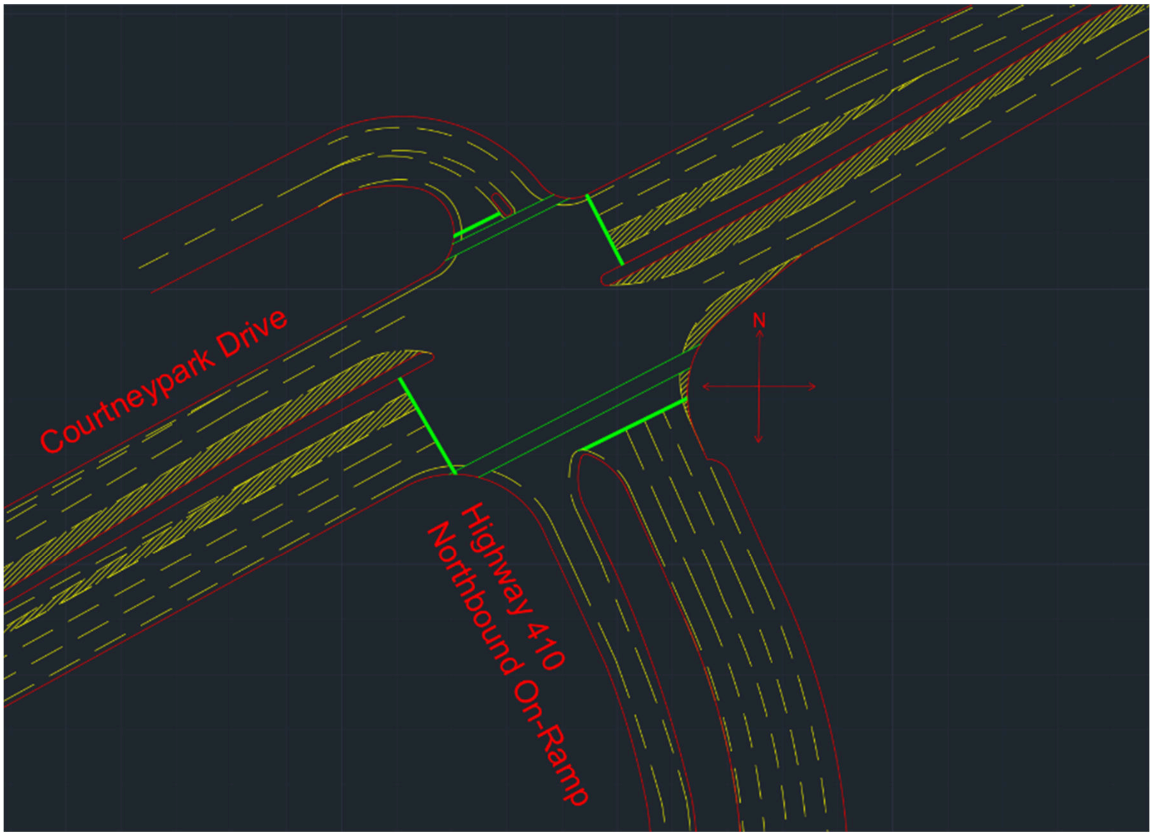

Figure 40: AutoCAD Drawing of Courtneypark Dr - Hwy 410 On-Ramp

A.2 – Swept Path Analysis for Existing Intersections

The results of the swept path analysis for each of the six intersections with their existing dimensions can be found in the following sections. This includes the swept path drawing of the turnpike double for all six intersections for Scenarios A through D.

A.2.1 – Dixie Road and Highway 401 Off-Ramp

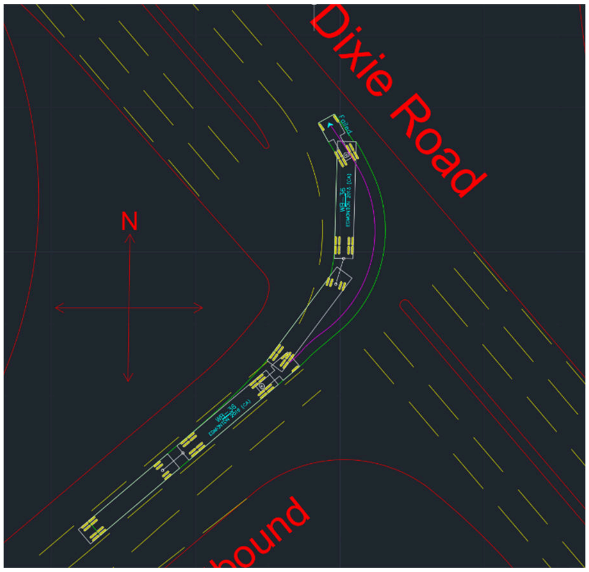

Figure 41: Dixie Rd and Hwy 401 Off-Ramp - Scenario A

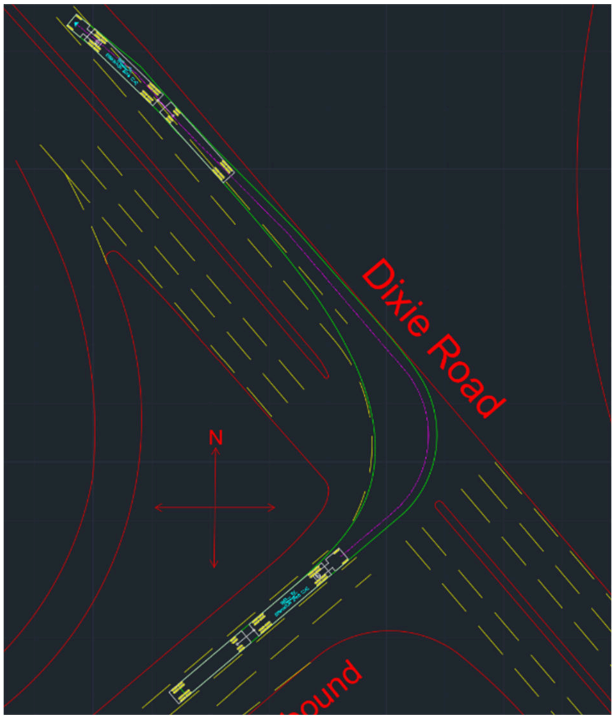

Figure 42: Dixie Rd and Hwy 401 Off-Ramp - Scenario B

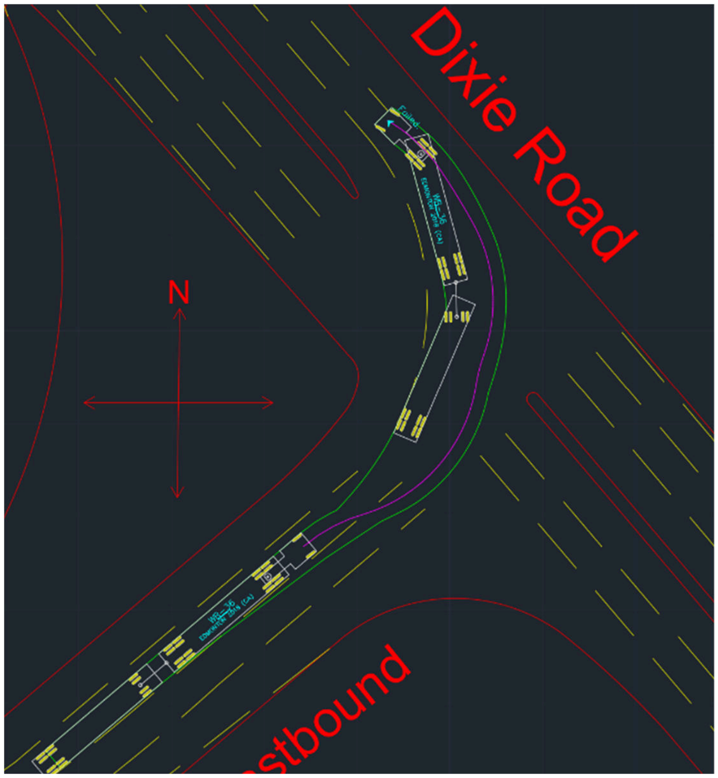

Figure 43: Dixie Rd and Hwy 401 Off-Ramp - Scenario C

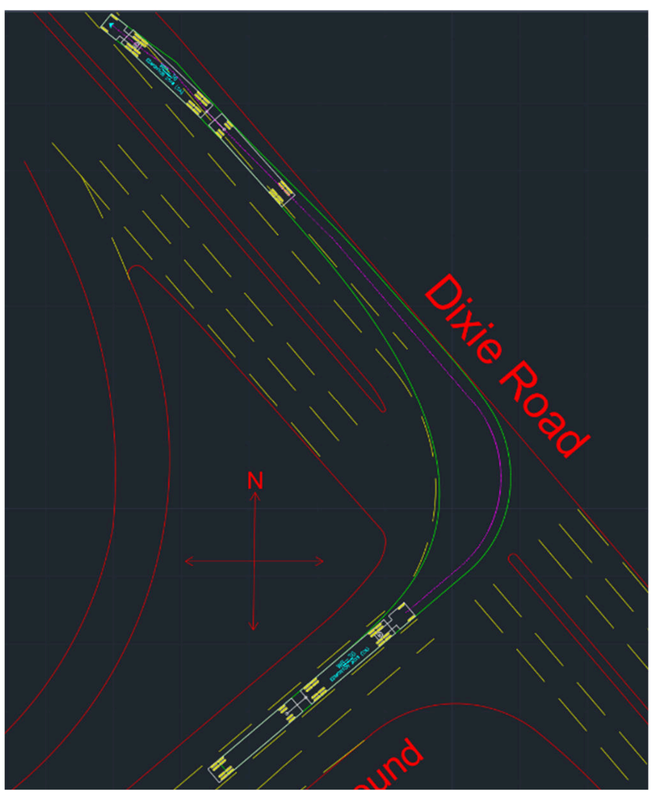

Figure 44: Dixie Rd and Hwy 401 Off-Ramp - Scenario D

A.2.2 – Dixie Road and Britannia Road

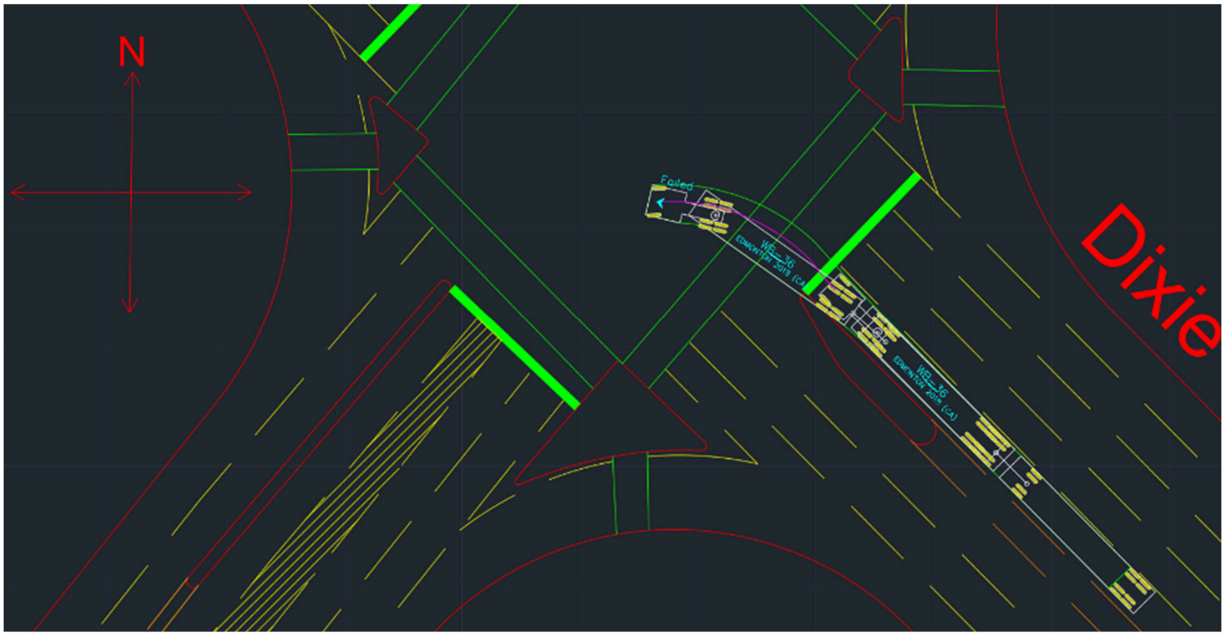

Figure 45: Dixie Rd and Britannia Rd - Scenario A

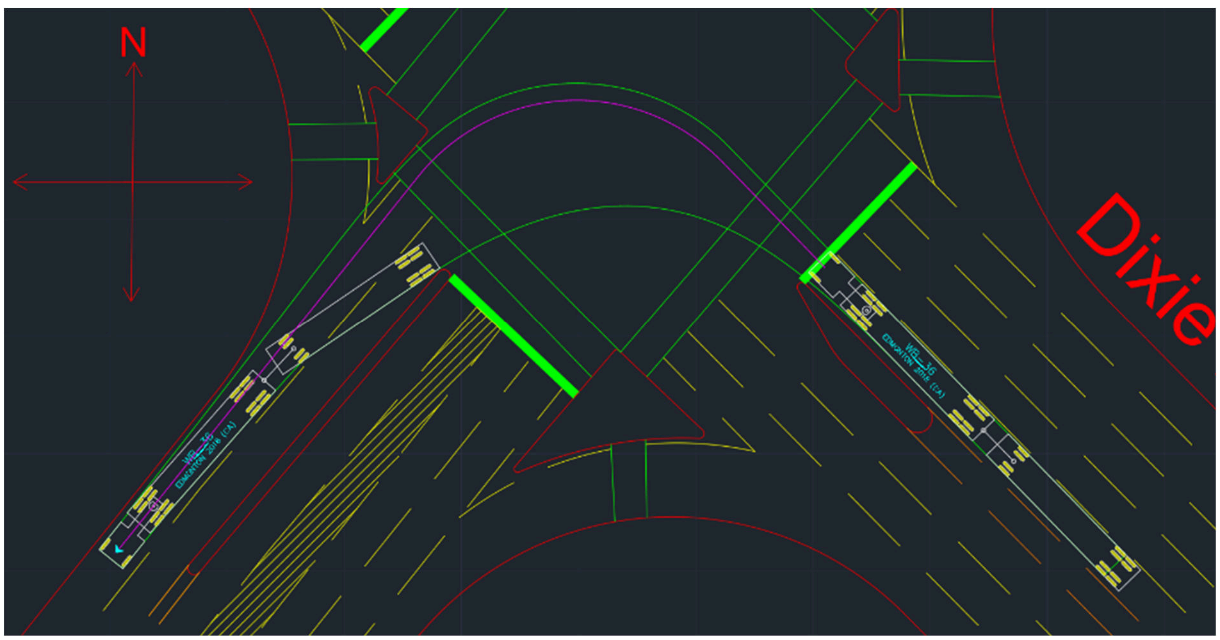

Figure 46: Dixie Rd and Britannia Rd - Scenario B

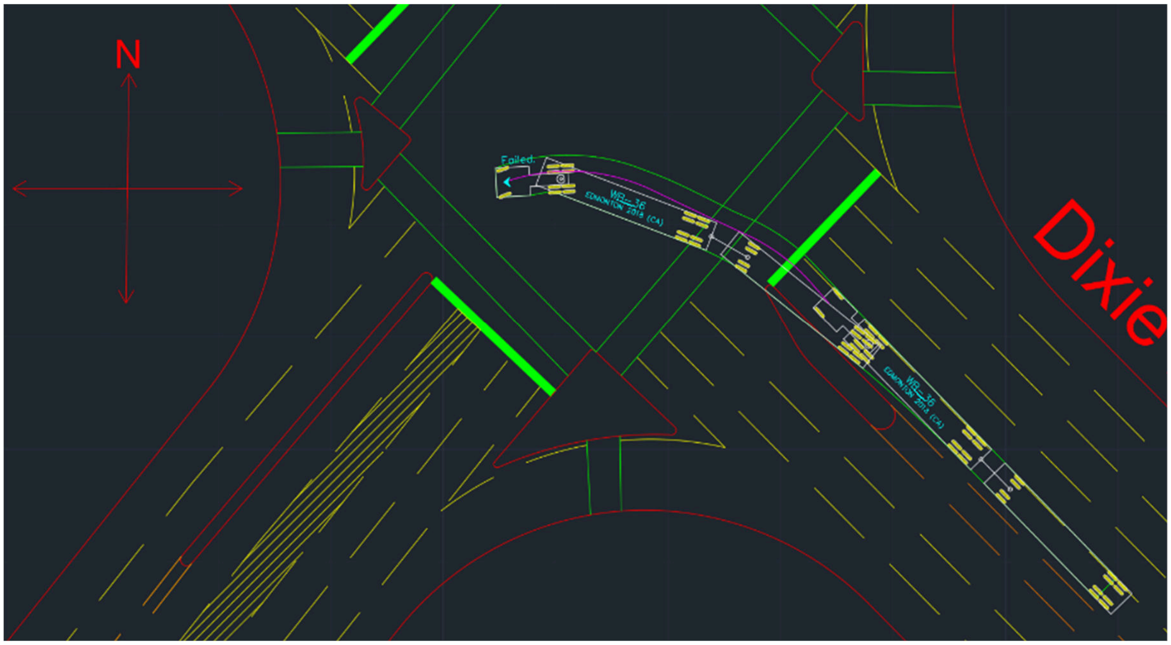

Figure 47: Dixie Rd and Britannia Rd - Scenario C

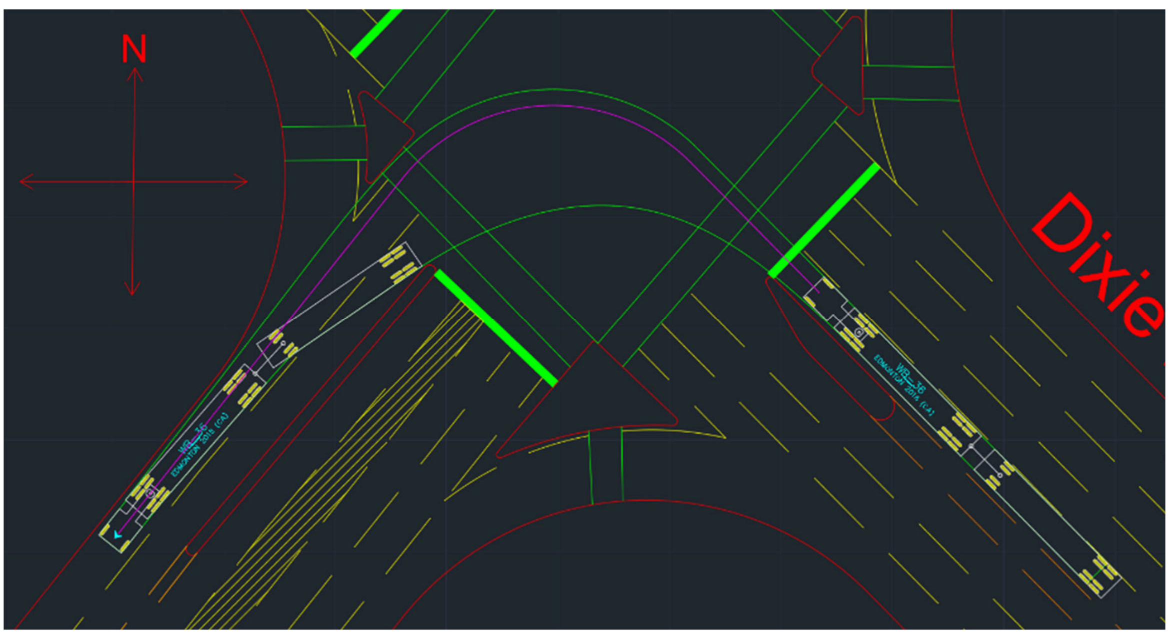

Figure 48: Dixie Rd and Britannia Rd - Scenario D

A.2.3 – Shawson Drive and Westport Crescent South

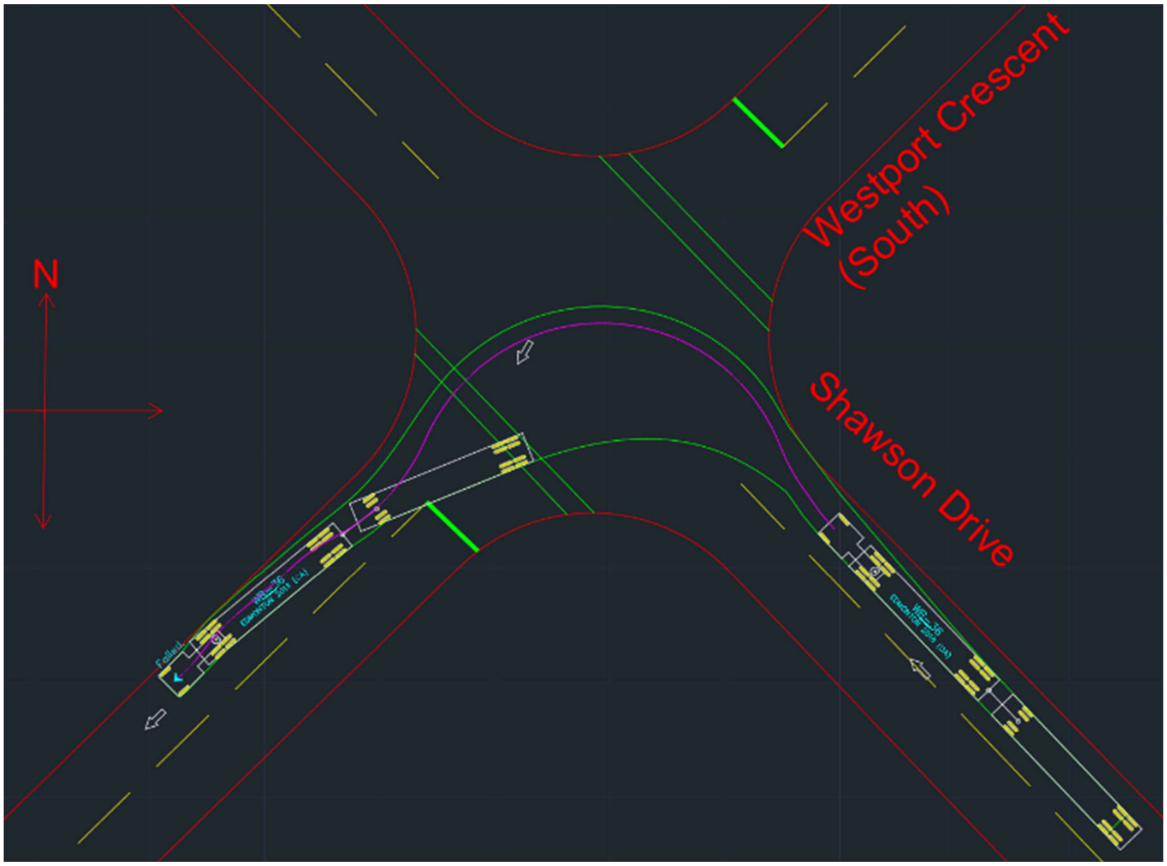

Figure 49: Shawson Dr and Westport Cr S - Scenario A

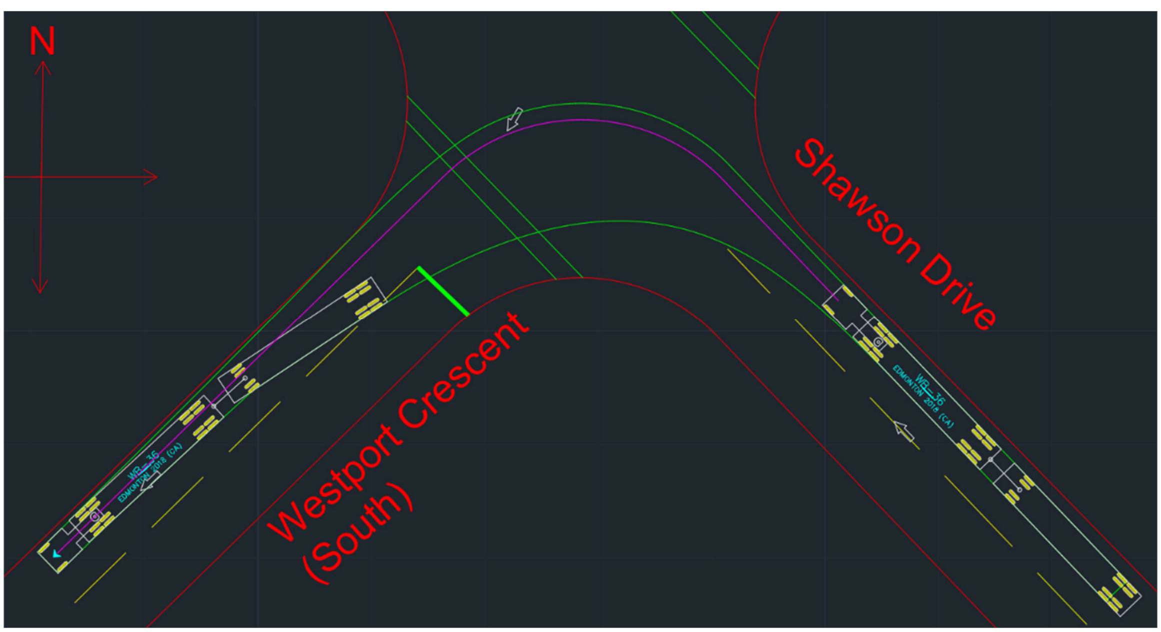

Figure 50: Shawson Dr and Westport Cr S - Scenario B

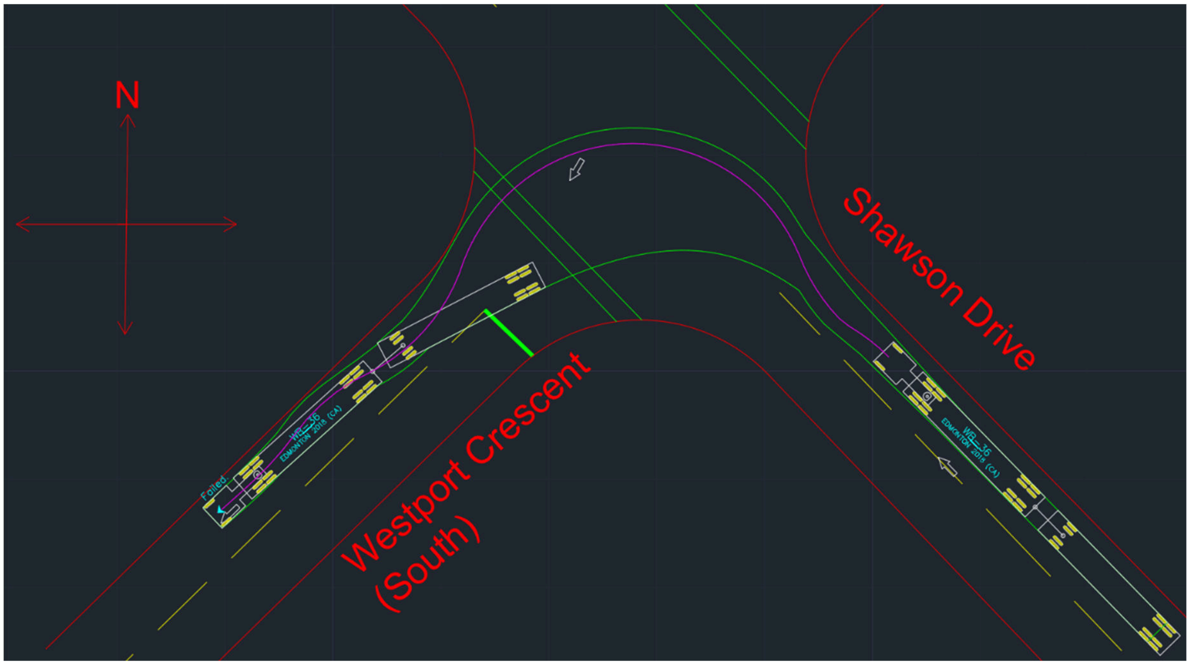

Figure 51: Shawson Dr and Westport Cr S - Scenario C

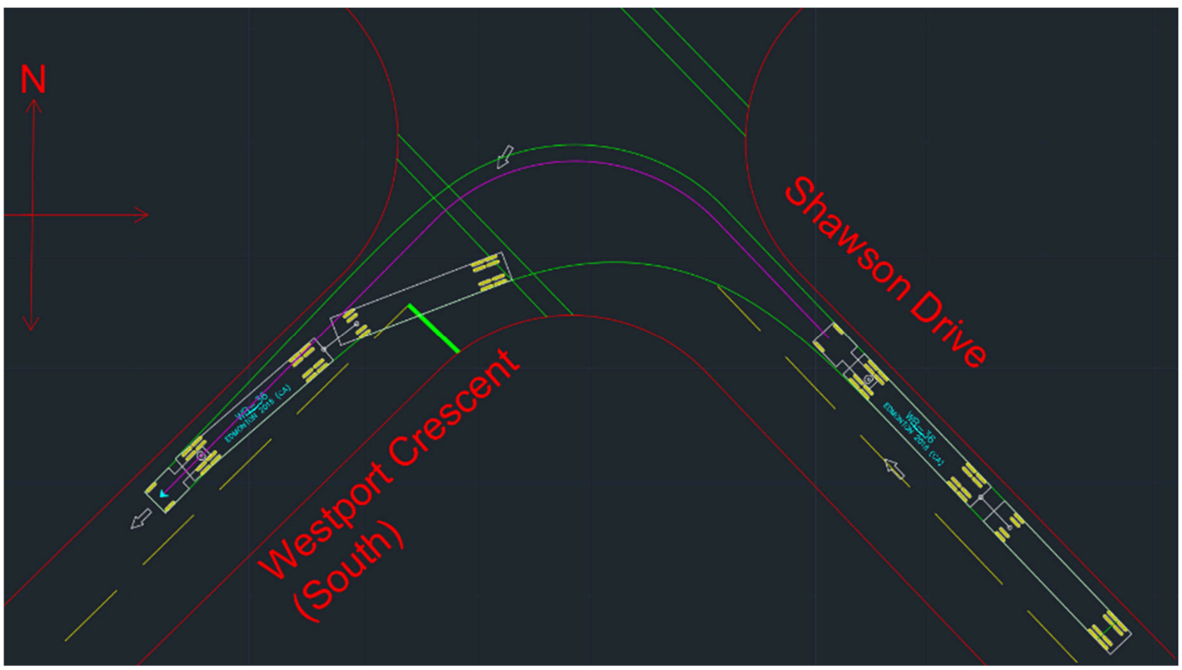

Figure 52: Shawson Dr and Westport Cr S - Scenario D

A.2.4 – Shawson Drive and Westport Crescent North

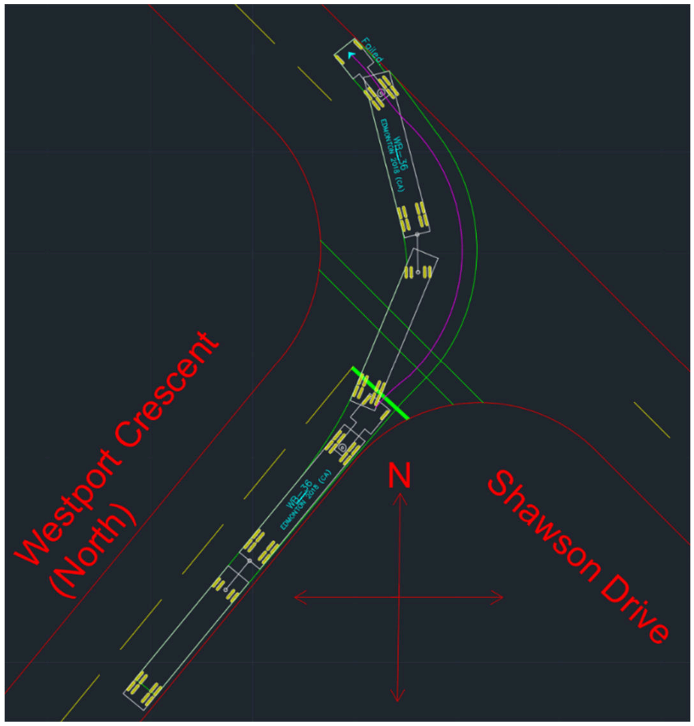

Figure 53: Shawson Dr and Westport Cr N - Scenario A

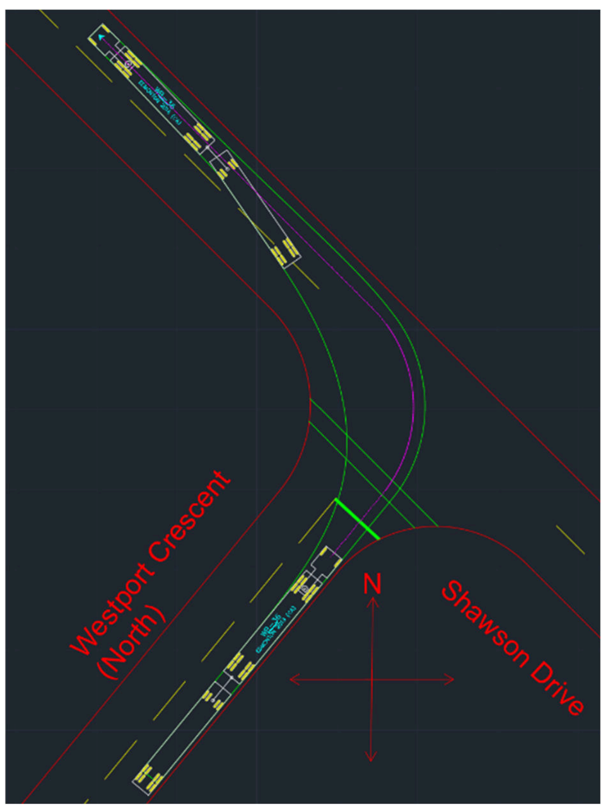

Figure 54: Shawson Dr and Westport Cr N - Scenario B

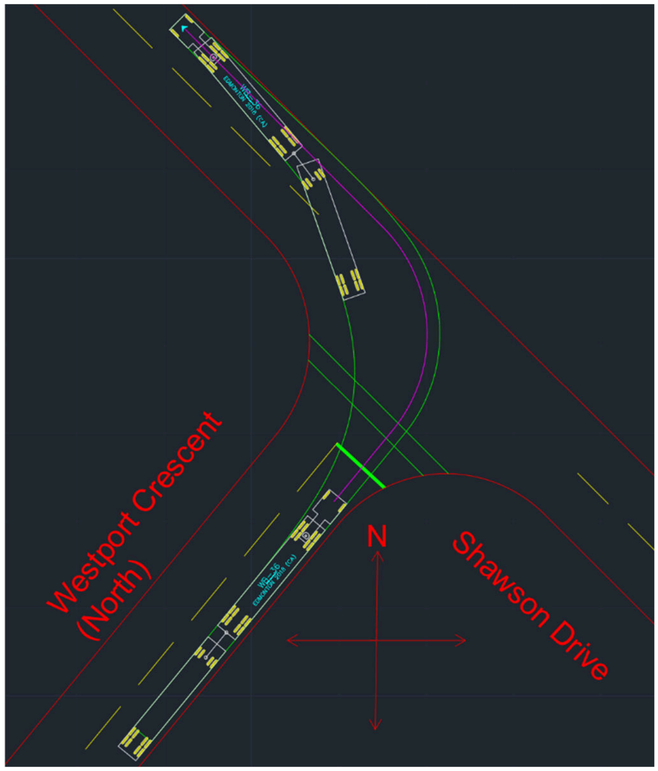

Figure 55: Shawson Dr and Westport Cr N - Scenario C

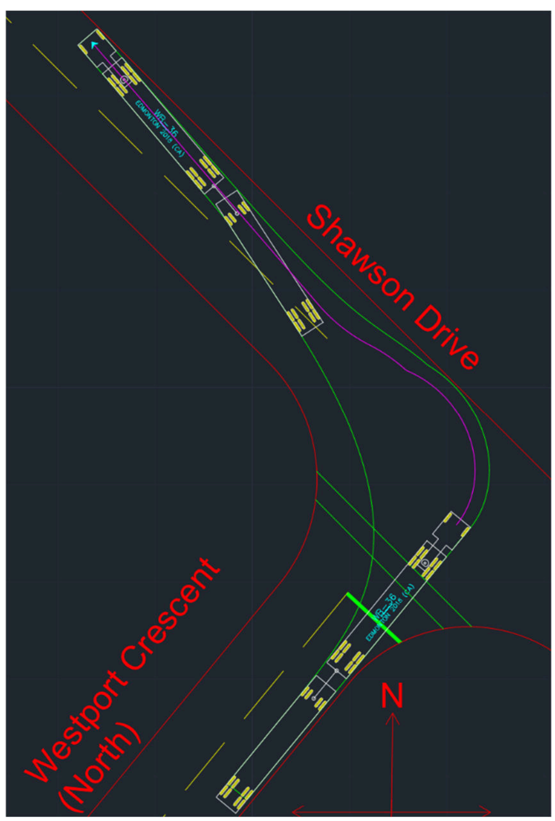

Figure 56: Shawson Dr and Westport Cr N - Scenario D

A.2.5 – Shawson Drive and Courtneypark Drive

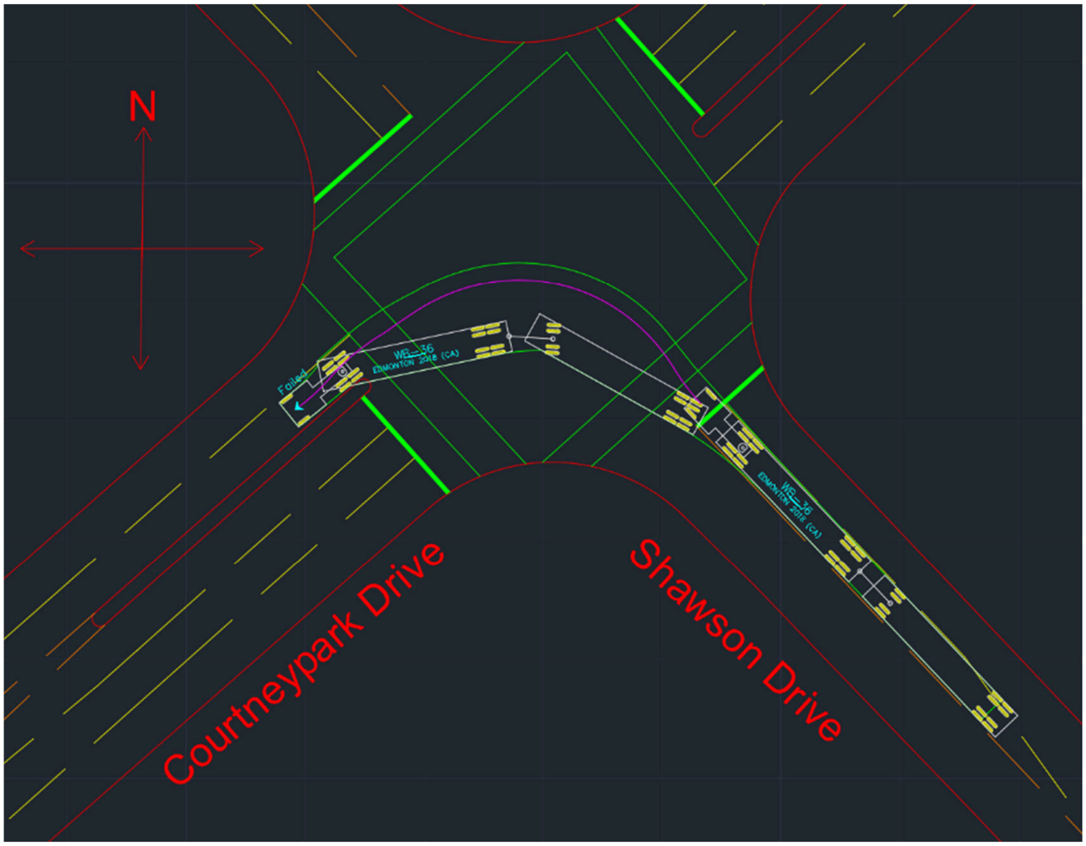

Figure 57: Shawson Dr and Courtneypark Dr - Scenario A

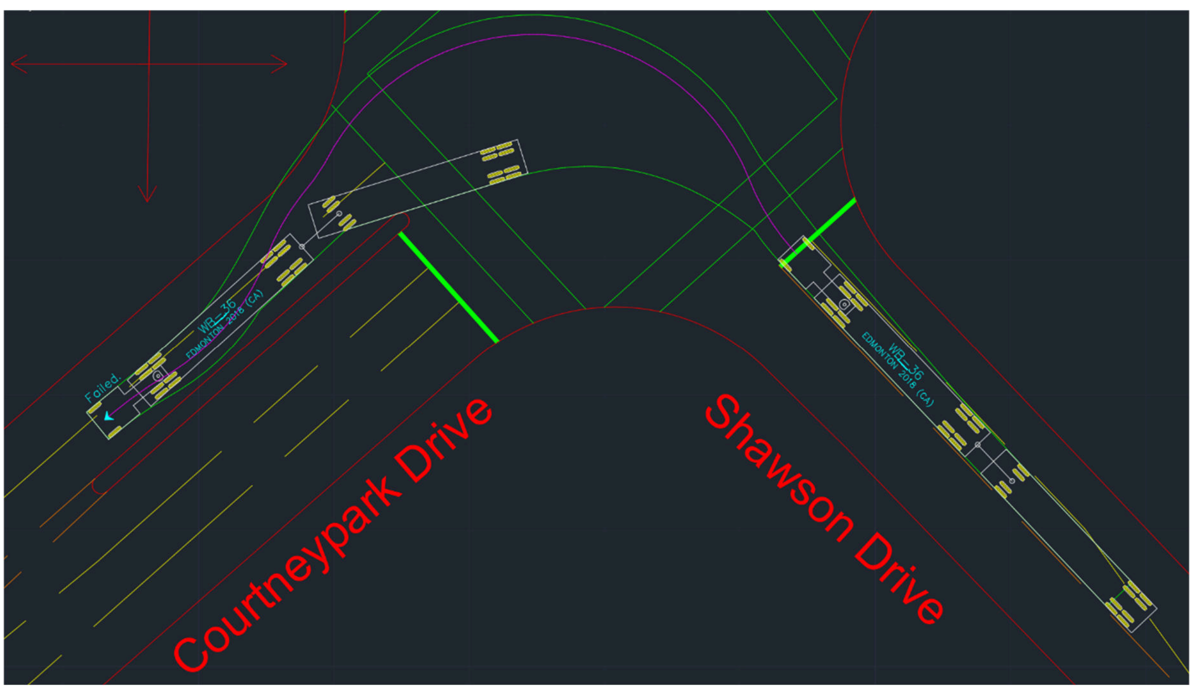

Figure 58: Shawson Dr and Courtneypark Dr - Scenario B

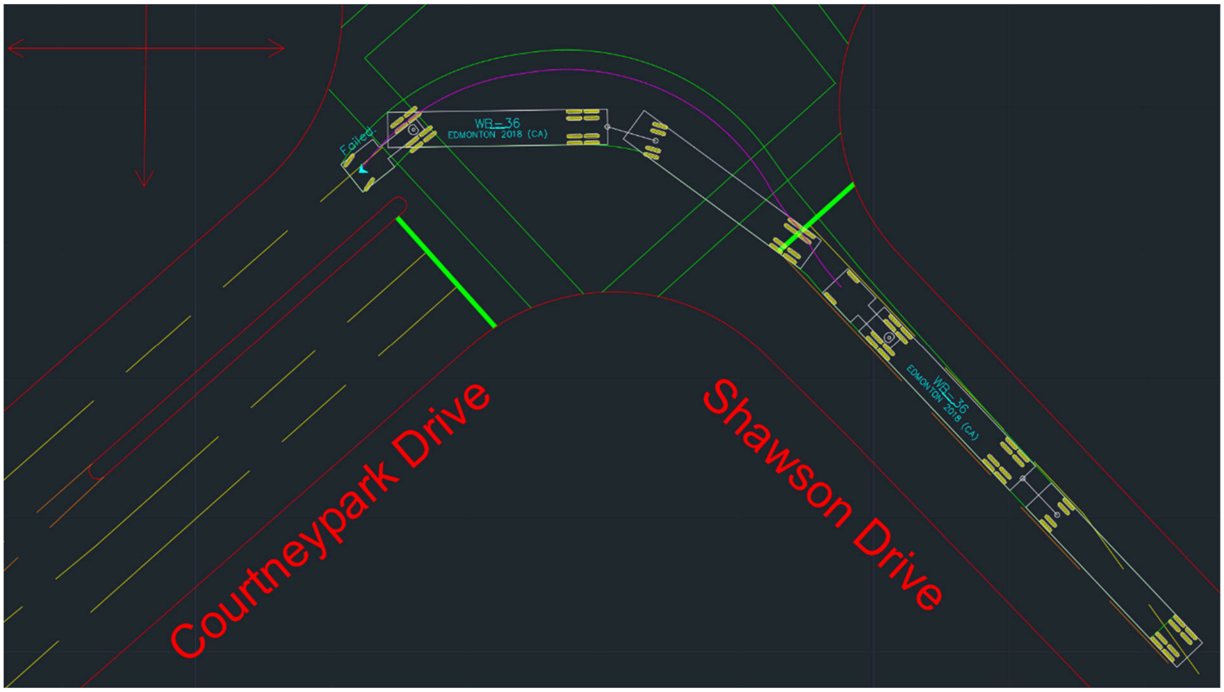

Figure 59: Shawson Dr and Courtneypark Dr - Scenario C

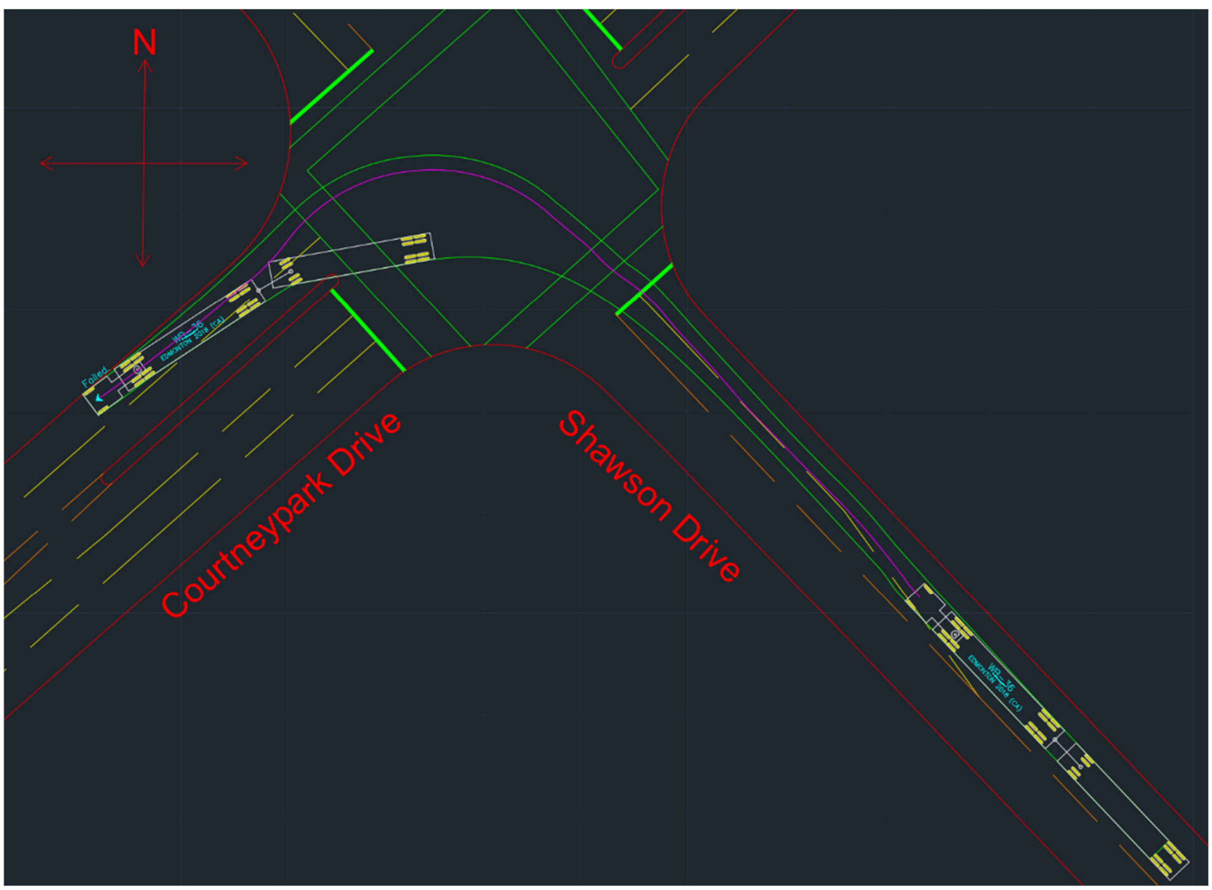

Figure 60: Shawson Dr and Courtneypark Dr - Scenario D

A.2.6 – Courtneypark Drive and Highway 410 On-Ramp

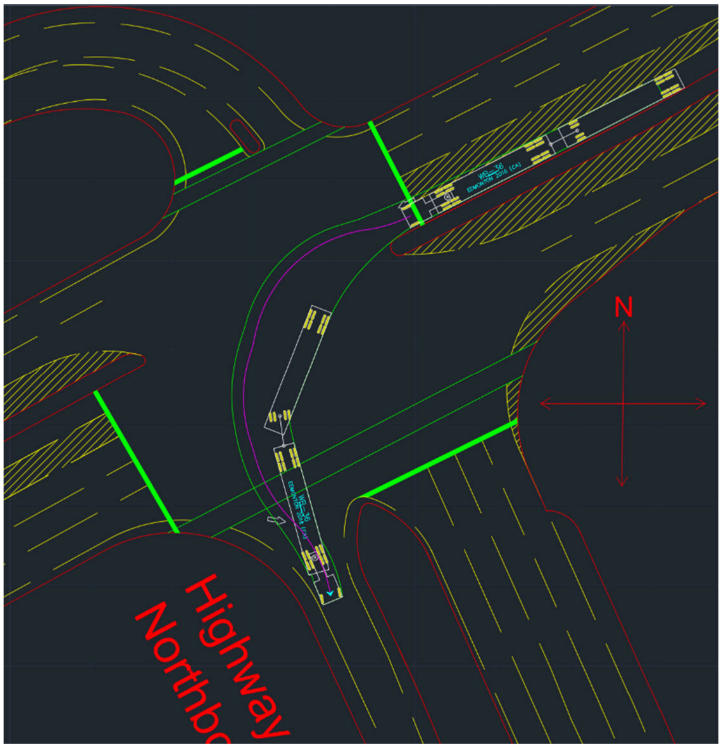

Figure 61: Courtneypark Dr and Hwy 410 On-Ramp - Scenario A

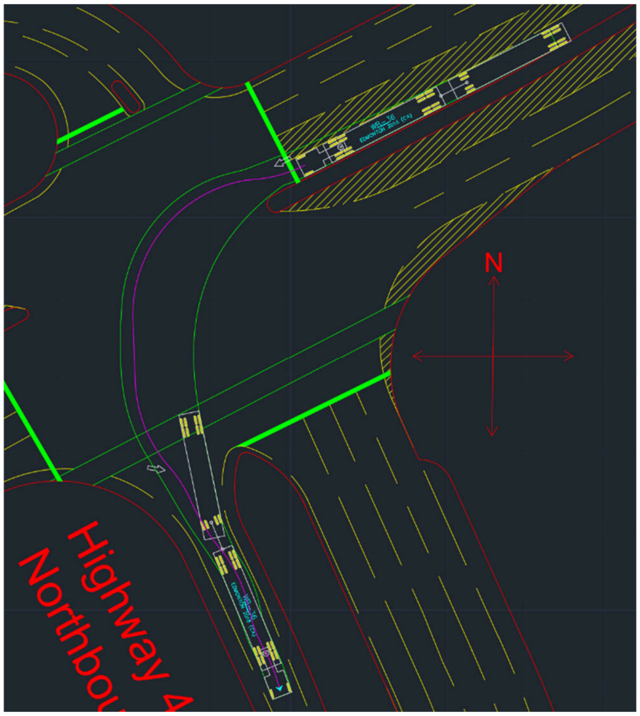

Figure 62: Courtneypark Dr and Hwy 410 On-Ramp - Scenario C

A.3 – NLOGIT Model

The following section provides the developed code used in the NLOGIT software, shown in blue, and their respective outputs, shown in red. The following table also serves as a dictionary to define the meaning behind the different variable numbers.

Table 23: NLOGIT Model Variable Dictionary

Variable	Meaning	Binary or Continuous?
V1	Scenario A	Binary
V2	Scenario B	Binary
V3	Scenario C	Binary
V4	Scenario D	Binary
V10	Setback Distance	Continuous
V19	Intersection 1	Binary
V20	Intersection 4	Binary
V21	Intersection 7	Binary
V22	Intersection 8	Binary
V23	Intersection 11	Binary
V24	Intersection 13	Binary
V31	Avg. Departing Width	Continuous
V32	Avg. Destination Width	Continuous

A.3.1 – Model 1: Logit Model Before Adding Intermediate Data Points

```
NLOGIT
;Lhs = Choice
;Choices = P,F
;Model:
U(P)  = P_Const + ScB*V2 + ScC*V3 + ScD*V4 + WAvg_Dep*V19+WAvg_Dest*V20 +
Setback*V10/
U(F)= 0
;effects:V2[P]/V3[P]/V4[P]/V19(P)/V20(P)/V10(P)$
```

```
---------------------------------------------------------------------------
Discrete choice (multinomial logit) model
Dependent variable                 Choice
Log likelihood function         -33.58774
Estimation based on N =      78, K =   7
Inf.Cr.AIC  =      81.2 AIC/N =    1.041
---------------------------------------
            Log likelihood R-sqrd R2Adj
Constants only     -52.8022  .3639 .3012
Note: R-sqrd = 1 - logL/Logl(constants)
---------------------------------------
```

```
Chi-squared[ 6]           =     38.42899
Prob [ chi squared > value ] =    .00000
Response data are given as ind. choices
Number of obs.=    78, skipped    0 obs
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
  CHOICE|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
 P_CONST|   -10.0473***     2.82577    -3.56  .0004    -15.5857   -4.5089
     SCB|    2.57958**      1.08522     2.38  .0175      .45258   4.70658
     SCC|    1.53257         .95522     1.60  .1086    -.33962    3.40476
     SCD|    5.17887***     1.39446     3.71  .0002    2.44578    7.91195
WAVG_DEP|     .40139**       .19378     2.07  .0383      .02159     .78120
WAVG_DES|    1.06795***      .27564     3.87  .0001      .52771   1.60819
 SETBACK|     .07508*        .04138     1.81  .0696    -.00601     .15618
--------+--------------------------------------------------------------------
***, **, * ==>  Significance at 1%, 5%, 10% level.
Model was estimated on Aug 22, 2024 at 00:02:20 PM
-----------------------------------------------------------------------------


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V2      |        P        F
--------+-----------------
       P|   .3559   -.3559


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V3      |        P        F
--------+-----------------
       P|   .2114   -.2114


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V4      |        P        F
--------+-----------------
       P|   .7145   -.7145


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V19     |        P        F
--------+-----------------
       P|   .7521  -1.1954


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V20     |        P        F
--------+-----------------
       P|  1.8313  -3.4859
```

```
Elasticity wrt change of X in row choice on Prob[column choice]
---------+-----------------
V10      |       P        F
---------+-----------------
        P|   .4862   -.6466
```

* It should be noted that in this model, V19 refers to the average departing lane width and V20 refers to the average destination width, unlike the other three models

A.3.2 – Model 2: Logit Model With Lane Width Variables, Setback Distance and Scenarios

```
NLOGIT
;Lhs = Choice
;Choices = P,F
;Model:
U(P) = P_Const + ScB*V2 + ScC*V3 + ScD*V4 + WAvg_Dep*V31+WAvg_Dest*V32 +
Setback*V10/
U(F)= 0
;effects:V2[P]/V3[P]/V4[P]/V31(P)/V32(P)/V10(P)$
```

```
--------------------------------------------------------------------------
Discrete choice (multinomial logit) model
Dependent variable               Choice
Log likelihood function      -111.78369
Estimation based on N =    264, K =   7
Inf.Cr.AIC  =    237.6 AIC/N =     .900
---------------------------------------
              Log likelihood R-sqrd R2Adj
Constants only   -172.4794  .3519 .3342
Note: R-sqrd = 1 - logL/Logl(constants)
---------------------------------------
Chi-squared[ 6]            =    121.39137
Prob [ chi squared > value ] =    .00000
Response data are given as ind. choices
Number of obs.=   264, skipped    0 obs
---------+----------------------------------------------------------------
         |                  Standard            Prob.      95% Confidence
  CHOICE|  Coefficient        Error         z    |z|>Z*          Interval
---------+----------------------------------------------------------------
 P_CONST|   -8.55066***     1.62840     -5.25  .0000   -11.74226  -5.35907
     SCB|    3.51665***      .61286      5.74  .0000     2.31547   4.71783
     SCC|    -.20670         .46561      -.44  .6571    -1.11927    .70587
     SCD|    4.73523***      .71648      6.61  .0000     3.33096   6.13951
WAVG_DEP|     .17478*        .10075      1.73  .0828     -.02268    .37224
WAVG_DES|     .86128***      .17127      5.03  .0000      .52561   1.19696
 SETBACK|     .06741**       .02970      2.27  .0232      .00920    .12563
---------+----------------------------------------------------------------
***, **, * ==>  Significance at 1%, 5%, 10% level.
Model was estimated on Aug 22, 2024 at 11:56:08 AM
--------------------------------------------------------------------------
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V2      |       P        F
--------+-----------------
       P|   .4832   -.4832


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V3      |       P        F
--------+-----------------
       P|  -.0284    .0284


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V4      |       P        F
--------+-----------------
       P|   .6507   -.6507


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V31     |       P        F
--------+-----------------
       P|   .5465   -.3056


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V32     |       P        F
--------+-----------------
       P|  2.6321  -1.6063


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V10     |       P        F
--------+-----------------
       P|   .6709   -.3392
```

<u>A.3.3 – Model 3: Logit Model With Only Intersection Dummy Variables</u>

```
NLOGIT
;Lhs = Choice
;Choices = P,F
;Model:
U(P) = P_Const + Int1 * V19 + Int4 * V20 + Int7 * V21 + Int8 * V22 + Int11
* V23 /
U(F)= 0
;effects:V19[P]/V20[P]/V21[P]/V22[P]/V23[P]/V24[P] $
```

```
--------------------------------------------------------------------------
Discrete choice (multinomial logit) model
```

```
Dependent variable                 Choice
Log likelihood function      -155.29688
Estimation based on N =    264, K =   6
Inf.Cr.AIC  =    322.6 AIC/N =    1.222
---------------------------------------
            Log likelihood R-sqrd R2Adj
Constants only   -172.4794  .0996 .0787
Note: R-sqrd = 1 - logL/Logl(constants)
---------------------------------------
Chi-squared[ 5]            =     34.36499
Prob [ chi squared > value ] =   .00000
Response data are given as ind. choices
Number of obs.=   264, skipped    0 obs
--------+--------------------------------------------------------------------
        |                Standard            Prob.      95% Confidence
  CHOICE|  Coefficient     Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
 P_CONST|    1.46634**      .64051     2.29  .0221       .21096   2.72172
    INT1|   -2.25479***     .74546    -3.02  .0025    -3.71587   -.79372
    INT4|   -2.22011***     .70864    -3.13  .0017    -3.60901   -.83120
    INT7|   -1.58412**      .69926    -2.27  .0235    -2.95464   -.21360
    INT8|   -1.66305**      .69961    -2.38  .0174    -3.03425   -.29184
   INT11|   -3.27645***     .73454    -4.46  .0000    -4.71612  -1.83677
--------+--------------------------------------------------------------------
***, **, * ==>  Significance at 1%, 5%, 10% level.
Model was estimated on Aug 22, 2024 at 11:55:41 AM
-----------------------------------------------------------------------------
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V19     |       P        F
--------+-----------------
       P|  -.4549    .4549
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V20     |       P        F
--------+-----------------
       P|  -.4479    .4479
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V21     |       P        F
--------+-----------------
       P|  -.3196    .3196
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V22     |       P        F
--------+-----------------
```

```
       P|  -.3355    .3355


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V23     |        P        F
--------+-----------------
       P|  -.6610    .6610


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V24     |        P        F
--------+-----------------
```

A.3.4 – Model 4: Logit Model Including Scenarios, Average Widths, Setback Distances, and Intersections

```
NLOGIT
;Lhs = Choice
;Choices = P,F
;Model:
U(P) = P_Const + ScB*V2 + ScC*V3 + ScD*V4 + WAvg_Dep*V31+WAvg_Dest*V32 +
Setback*V10+ Int1 * V19 + Int4 * V20 + Int7 * V21 + Int8 * V22 + Int11 *
V23 /
U(F)= 0
;effects:V2[P]/V3[P]/V4[P]/V31(P)/V32(P)/V10(P)/V19[P]/V20[P]/V21[P]/V22[P
]/V23[P]/V24[P] $

----------------------------------------------------------------------------
Discrete choice (multinomial logit) model
Dependent variable               Choice
Log likelihood function       -90.14765
Estimation based on N =    264, K =  12
Inf.Cr.AIC  =    204.3 AIC/N =     .774
----------------------------------------
            Log likelihood R-sqrd R2Adj
Constants only   -172.4794  .4773 .4525
Note: R-sqrd = 1 - logL/Logl(constants)
----------------------------------------
Chi-squared[11]          =    164.66346
Prob [ chi squared > value ] =   .00000
Response data are given as ind. choices
Number of obs.=   264, skipped    0 obs
--------+-------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
  CHOICE|  Coefficient       Error         z    |z|>Z*        Interval
--------+-------------------------------------------------------------------
 P_CONST|   -9.05281***     2.29319     -3.95  .0001   -13.54738  -4.55823
     SCB|    4.79574***      .82337      5.82  .0000     3.18197   6.40952
     SCC|     .31581         .52105       .61  .5444     -.70543   1.33705
     SCD|    6.68187***      .99594      6.71  .0000     4.72986   8.63388
WAVG_DEP|     .33101*        .18127      1.83  .0678     -.02428    .68630
WAVG_DES|    1.26307***      .25119      5.03  .0000      .77074   1.75539
```

```
 SETBACK|     .13374***      .03811     3.51  .0004      .05906    .20843
    INT1|   -4.22484***     1.22263    -3.46  .0005    -6.62115  -1.82853
    INT4|   -2.89308***     1.09535    -2.64  .0083    -5.03993   -.74623
    INT7|   -4.51892***     1.27833    -3.54  .0004    -7.02441  -2.01343
    INT8|   -4.61632***     1.33017    -3.47  .0005    -7.22340  -2.00924
   INT11|   -5.53715***     1.16860    -4.74  .0000    -7.82756  -3.24674
--------+------------------------------------------------------------------
***, **, * ==>  Significance at 1%, 5%, 10% level.
Model was estimated on Aug 22, 2024 at 11:51:32 AM
---------------------------------------------------------------------------


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V2      |        P        F
--------+-----------------
       P|   .5154   -.5154


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V3      |        P        F
--------+-----------------
       P|   .0339   -.0339


Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V4      |        P        F
--------+-----------------
       P|   .7181   -.7181


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V31     |        P        F
--------+-----------------
       P|  1.0351   -.5787


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V32     |        P        F
--------+-----------------
       P|  3.8600  -2.3556


Elasticity wrt change of X in row choice on Prob[column choice]
--------+-----------------
V10     |        P        F
--------+-----------------
       P|  1.3312   -.6729
```

```
Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V19     |       P        F
--------+-----------------
       P|  -.4541    .4541

Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V20     |       P        F
--------+-----------------
       P|  -.3109    .3109

Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V21     |       P        F
--------+-----------------
       P|  -.4857    .4857

Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V22     |       P        F
--------+-----------------
       P|  -.4961    .4961

Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V23     |       P        F
--------+-----------------
       P|  -.5951    .5951

Derivative wrt change of X in row choice on Prob[column choice]
--------+-----------------
V24     |       P        F
```

Appendix B – Microsimulation Modelling

B.1 – Volume Balancing and Turning Proportions

As outlined in *Chapter 3. Data*, the traffic data that is used in the Vissim simulation is provided by the City of Mississauga and the Region of Peel. This volume data is used as the vehicle inputs throughout the traffic network. However, before this data can be applied to the simulation, a volume balancing procedure must be completed. Volume balancing is the process of resolving discrepancies in collected traffic volume data between adjacent locations in a road network (Wisconsin DOT, 2019a). In this thesis, volume balancing is needed for two reasons. First, volume balancing is typically needed in instances where traffic data has been collected across multiple time periods and sources, leading to inconsistencies and errors in traffic volumes between adjacent intersections (Wisconsin DOT, 2019a). The volume data being used here is provided by two independent entities, where Dixie Road data is provided by the Region of Peel and other signalized intersections are provided by the City of Mississauga. Data is also provided across multiple months, years, and times of day. As a result, the provided data when observed together exhibit large discrepancies in traffic volumes between adjacent intersections in the network. Second, the data provided by the City of Mississauga and the Region of Peel does not include information pertaining to the four unsignalized intersections in the network, as well as the data missing from the Courtneypark Drive-Highway 410 Northbound Ramp intersection. As a result, volume balancing is applied to manually ensure that the generated volumes at these intersections are consistent with the known volumes at adjacent intersections.

In this thesis, the intersections for which data is provided by the City of Mississauga or Region of Peel are referred to as known intersections, while intersections where no data is provided by either jurisdiction are referred to as unknown intersections.

B.1.1 – Selecting a Uniform Time Period

To balance the traffic volumes, a uniform time of day and year must be selected and all volumes must be converted to this time period. The Region of Peel data is provided consistently in 2022 and full study data is provided from 7:00 am until 6:00 pm in 15-minute segments for each intersection. Meanwhile, the City of Mississauga has provided full study reports, AM peak period data, PM peak period data, and midday peak period data for each intersection, while the collection year ranges from 2015 to 2018. Therefore, the PM peak period, from 4:00 pm to 5:00 pm, is selected as the consistent time period since it is provided for all known intersections. Some intersections are provided with a PM peak period from 4:30 pm to 5:30 pm or from 4:15 pm to 5:15 pm, but due to the small difference in time and the potential for creating more error in adjusting these time periods, the data is not adjusted for this time difference. This initial data is shown in *Table 24*. All unknown intersections without data are excluded from this table.

Table 24: Raw Traffic Volumes for Known Intersections

Int. #	Intersection	Selected Time Period	Initial Year	Top Leg ↓			Right Leg ←			Bottom Leg ↑			Left Leg →		
				Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left
1	Highway 401 EB Off-Ramp & Dixie	4:00 - 5:00 pm	2022		1554						1348		405		183
2	Highway 401 WB Off-Ramp & Dixie	4:00 - 5:00 pm	2022		1727		566		440		1047				
3	Shawson & Dixie	4:00 - 5:00 pm	2022	59	1459	99	142	53	456	185	1191	117	247	47	68
4	Britannia & Dixie	4:00 - 5:00 pm	2022	144	1160	108	124	165	230	150	1108	99	197	86	70
6	Britannia & Shawson	4:30 - 5:30 pm	2016	77	130	78	123	382	72	49	160	207	83	219	33
9	Meyerside & Shawson	4:30 - 5:30 pm	2016	54	88	14	76	406	46	79	287	165	55	165	14
11	Courtneypark & Shawson	4:15 - 5:15 pm	2015	36	0	15	22	998	21	42	0	143	26	339	56
12	Courtneypark & Tomken	4:00 - 5:00 pm	2018	360	453	87	132	1256	24	31	1195	574	163	369	126

With respect to the year, all volume data must be adjusted to reflect 2022 traffic data. To do this, future travel demand forecasting of the local area is needed. In a 2015 traffic analysis report of Courtneypark Drive East, the annual growth rate (AGR) of traffic on several roads between 2015 and 2031 was determined, including a 1% annual growth rate on Courtneypark Drive East and a 0.5% annual growth rate on Dixie Road, Tomken Road, Derry Road, and Kennedy Road (Soo et al., 2015). Shawson Drive, which is similar in characteristics to Tomken Road, is assumed to also have a growth rate of 0.5% per year. Therefore, intersection data from 2015 to 2018 utilized these growth rates in the following equation to convert all values to 2022 values:

Equation 7: $$Volume_{2022} = Volume_{Initial} * (1 + \frac{AGR}{100})^{(2022 - Initial\ Year)}$$

Using the corresponding AGR and initial year values for each intersection, all data is converted to 2022 values, summarized in *Table 25*. Volumes adjusted from their base year to 2022 are highlighted with italicized font.

Table 25: Traffic Volumes Adjusted to 2022 Data

Int. #	Intersection	Year	Top Leg ↓			Right Leg ←			Bottom Leg ↑			Left Leg →		
			Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left
1	Highway 401 EB Off-Ramp & Dixie	2022		1554						1348		405		183
2	Highway 401 WB Off-Ramp & Dixie	2022		1727		566		440		1047				
3	Shawson & Dixie	2022	59	1459	99	142	53	456	185	1191	117	247	47	68
4	Britannia & Dixie	2022	144	1160	108	124	165	230	150	1108	99	197	86	70
6	*Britannia & Shawson*	*2022*	*79*	*134*	*80*	*127*	*394*	*74*	*50*	*165*	*213*	*86*	*226*	*34*
9	*Meyerside & Shawson*	*2022*	*56*	*91*	*14*	*78*	*418*	*47*	*81*	*296*	*170*	*57*	*170*	*14*
11	*Courtneypark & Shawson*	*2022*	*39*	*0*	*16*	*24*	*1070*	*23*	*45*	*0*	*153*	*28*	*363*	*60*
12	*Courtneypark & Tomken*	*2022*	*375*	*471*	*91*	*137*	*1307*	*25*	*32*	*1244*	*597*	*170*	*384*	*131*

*Italicized entries indicate data is adjusted to the year 2022 using the AGR

B.1.2 – Turning Proportions for 2022 Time Period

For the intersection data provided by the City of Mississauga or the Region of Peel, another key piece of information carried over into Vissim is the proportions of vehicles turning at each intersection for each vehicle input. Since these values are provided as proportions of the total traffic volume, these turning proportions are still used for each intersection, regardless of whether they are adjusted to represent 2022 data.

B.1.3 – Volume and Turning Proportions for Unknown Intersections

Now that all the known intersections have been converted to 2022 data, the next step is to determine the volumes and turning proportions used for intersections with missing data, referred to as the unknown intersections. This includes the Vipond Dr-Britannia Rd intersection, Shawson Dr-Westport Cr southern intersection, Shawson Dr-Westport Cr northern intersection, Shawson Dr-Tristar Dr intersection, and the Courtneypark Dr-Highway 410 Ramp intersection, all of which have no volume data or turning proportion data. To fill in this data, numerous methods are used by taking into account data from adjacent known intersections in the area. Starting with the turning proportions, these are determined for the unknown intersections by calculating the average proportion of turns in each direction across all known intersections. For each known intersection, a major road and a minor road are identified, with the major road referring to the road with the higher volume of traffic, and the minor road referring to the road with the lower traffic volume. From these two road categories, an average turning proportion is calculated. *Table 26* shows the turning proportions across all intersections, as well as the calculated averages for the major and minor road proportions. In this table, the major road cells are in bold font and the minor roads are written in normal font. Since this process is used to determine turning proportions of local roads, the turning proportions from any highway off-ramp is excluded from this calculation.

Table 26: Turning Proportion Calculations and Averages on Known Intersections

Int. #	Intersection	Top Leg ↓			Right Leg ←			Bottom Leg ↑			Left Leg →		
		Right (%)	Thru (%)	Left (%)	Right (%)	Thru (%)	Left (%)	Right (%)	Thru (%)	Left (%)	Right (%)	Thru (%)	Left (%)
3	Shawson & Dixie	**3.65**	**90.23**	**6.12**	21.81	8.14	70.05	**12.39**	**79.77**	**7.84**	68.23	12.98	18.78
4	Britannia & Dixie	**10.20**	**82.15**	**7.65**	23.89	31.79	44.32	**11.05**	**81.65**	**7.30**	55.81	24.36	19.83
6	Britannia & Shawson	26.96	45.73	27.30	**21.34**	**66.22**	**12.44**	11.68	38.55	49.77	**24.86**	**65.32**	**9.83**
9	Meyerside & Shawson	**34.78**	**56.52**	**8.70**	14.36	76.98	8.66	**14.81**	**54.11**	**31.08**	23.65	70.54	5.81
11	Courtneypark & Shawson	70.91	0	29.09	**2.15**	**95.79**	**2.06**	22.73	0	77.27	**6.21**	**80.49**	**13.30**
12	Courtneypark & Tomken	40.02	50.27	9.71	**9.33**	**88.97**	**1.70**	1.71	66.42	31.87	**24.82**	**56.06**	**19.12**

Averages	Right (%)	Thru (%)	Left (%)
Major Road	**14.63**	**74.77**	**10.59**
Minor Road	31.81	35.48	32.71

*Bold indicates that road is denoted as the major road in the intersection

Therefore, these averages will be the assumed turning proportions for any intersection with missing data. For example, for traffic approaching an unknown intersection on that intersection's major road, it will be assumed that 14.63% will turn right, 74.77% will travel through, and 10.59% will turn left. Of course, these values are open to adjustment based on whether the intersection is 4-legged or 3-legged. In the case of a 3-legged intersection, not all three maneuvers will be possible. As a result, when applying this to an unknown intersection, the unavailable turning maneuver's proportion is divided amongst the remaining available intersections.

In determining the initial volumes for the unknown intersections, data from adjacent intersections is used. First, for each known intersection, a major-road-major-direction (MaRMD) and a minor-road-major-direction (MiRMD) are defined. The MaRMD is the approach direction on the major road with a greater volume of traffic. For example, on a major road with 50 cars in the northbound direction and 100 cars in the southbound direction, the MaRMD is the southbound direction. On the other hand, the MiRMD is the approach direction on the minor road with a greater volume of traffic. With these two categories assigned for each known intersection, the percentage of traffic in the MaRMD and MiRMD is calculated as a portion of the sum of the MaRMD and MiRMD. The results of this step are shown in the table below. Therefore, for an intersection with no data provided, it will be assumed that 63.6% of the total intersection volume will travel on the major road and 36.4% on the minor road.

Table 27: Traffic Volume Proportions for the MaRMD and MiRMD

Int. #	Intersection	MaRMD	MiRMD	MaRMD Volumes			MiRMD Volumes			MaRMD Volume %	MiRMD Volume %
				Right	Thru	Left	Right	Thru	Left		
3	Shawson & Dixie	SB	WB	59	1459	99	142	53	456	71.3	28.7
4	Britannia & Dixie	SB	WB	144	1160	108	124	165	230	73.1	26.9
6	Britannia & Shawson	WB	NB	127	394	74	50	165	213	58.2	41.8
9	Meyerside & Shawson	WB	NB	78	418	47	81	296	170	50.2	49.8
11	Courtneypark & Shawson	WB	NB	24	1070	23	45	0	153	84.9	15.1
12	Courtneypark & Tomken	NB	WB	32	1244	597	137	1307	25	56.0	43.9
Average of All Intersections										63.6	36.4

This calculated volume proportion is now used to deduce an assumed value for each unknown intersection. This process differs slightly based on whether the intersection is adjacent to known intersections on both sides, or adjacent to one known intersection and one unknown intersection. For an unknown intersection with known intersections on both sides, the traffic volume input on the minor road is calculated by proportioning the known direction's MaRMD volume with the average percentages calculated for all intersection's MaMRD and MiMRD. Using the intersection of Vipond Dr and Britannia Dr as an example, the MaRMD along Britannia Dr is westbound, with 408 vehicles travelling in this direction from the adjacent intersection. To calculate the incoming volume on Vipond Dr, the ratio of volumes on the MaMRD and MiMRD are determined in the following equation:

Equation 8: $$V_{minor\ road} = \frac{V_{MaMRD}}{0.6361} * (1 - 0.6361)$$

In assigning the 408 vehicles for the V_{MaMRD} value, the incoming volume on Vipond Dr is 234. The proportion of these vehicles turning in each direction of the intersection are then calculated using the average turning proportions identified in *Table 26* corresponding to the major and minor roads. The following schematic summarizes this process for the Britannia Rd-Vipond Dr intersection. This process is repeated for the Shawson Dr-Tristar Dr intersection as well.

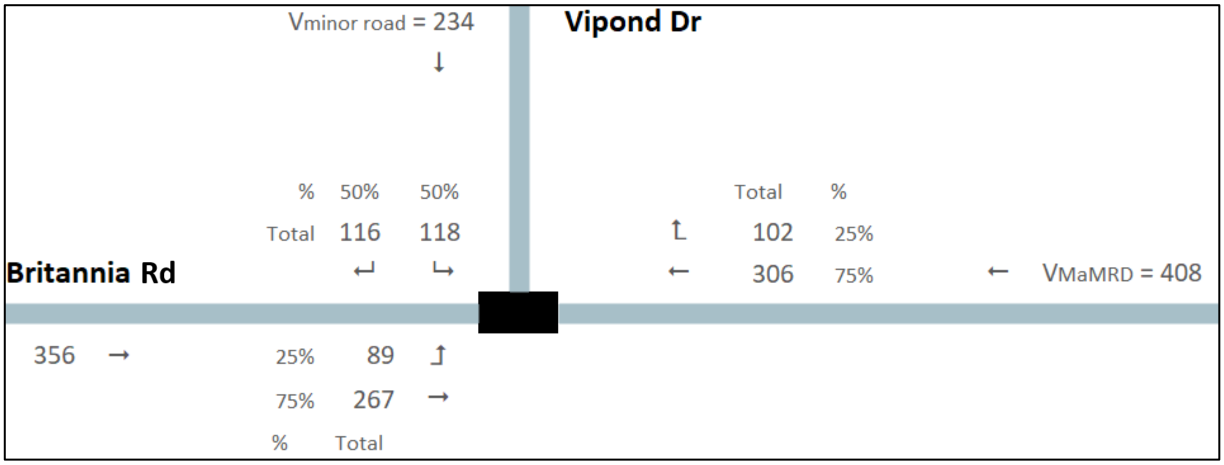

Figure 63: Britannia Rd and Vipond Dr Volume Calculation

For the two Shawson Dr-Westport Cr northern and southern intersections, both unknown intersections are adjacent to one known intersection and one unknown intersection. In this case, additional steps are needed. At the leg of the intersection adjacent to the known intersection, both the input entering the intersection (V_{MaMRD}) and the expected output to the adjacent intersection (E) are known. Using the V_{MaMRD}, the input volume on the minor road V_{minor} is calculated using *Equation 8*, as are the turning proportions using the calculated percentages. However, an additional step involves determining the incoming traffic volumes on the other side of the major road, which are still unknown. To calculate this value, the E value and the minor road turning proportions are used.

For example, consider the intersection between Shawson Dr and Westport Cr North, exhibited in *Figure 64*. The input from the Meyerside Dr and Shawson Dr intersection to the north (V_{MaMRD} = 195) and the expected output to the northern intersection (E = 547) are both known values. The traffic input on Westport Cr (V_{minor}) is calculated using *Equation 8* and specific volumes for each maneuver are calculated using the known minor road turning proportions. Using the E value and the left turn from Westport Cr, the through volume heading northbound from the southern leg is calculated. Once again referring to the turning proportions, the left turn onto Westport Cr from Shawson Dr is then determined. This process is repeated for the southern Westport Cr-Shawson Dr intersection as well.

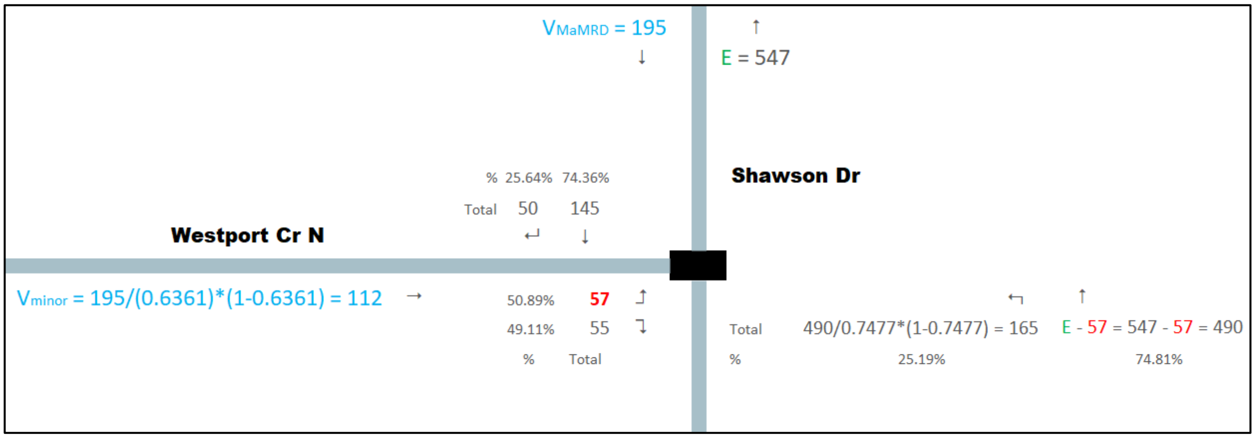

Figure 64: Westport Cr N and Shawson Dr Volume Calculation

The final unknown intersection still missing traffic volumes is the Courtneypark Dr-Highway 410 Ramp intersection. Since this intersection involves a major highway interchange and a major arterial road, a different approach is taken to fill in the missing values for this intersection since the local road values used in the previous intersections would not likely be applicable to this major intersection. Instead, the values here are filled in using data taken from the only other similar intersections in the area; the Dixie Rd-Highway 401 Off-Ramps. To fill in the off-ramp input values, the volume is copied from the Highway 401 Westbound Off-Ramp. The eastbound traffic volume along Courtneypark Dr is filled in using the average of the northbound and southbound directions from both Highway 401-Dixie Rd intersections. The westbound volume is determined using the traffic flow from the adjacent intersection, Courtneypark Dr and Tomken Rd. Lastly, the traffic volume output from the carpool in the north is assumed to be 40% of the available parking capacity of the carpool. In filling in the turning volume proportions, the previously calculated turning proportions for major and minor roads are used with an additional assumption being that that only 2% of traffic in each direction enters the carpool, so turning proportions are adjusted accordingly.

It is important to note that one of the limitations of these methods of determining the traffic volumes is the number of assumptions being made. Due to the fact that no data was available for many of the intersections along the selected route, much of this data is filled in with assumed values and methods that result in plausible traffic volume values but may not perfectly reflect the values in the network. Fortunately, the purpose of this thesis is not necessarily concerned with the performance of the network in terms of traffic flow. Since this thesis is studying the safety concerns associated with last-mile LCV usage, a sensitivity analysis of LCVs in the network is instead performed. Therefore, as long as a reasonable base scenario is devised in Vissim for these LCVs to operate within, the sensitivity analysis should still yield appropriate results and should not require perfectly accurate traffic volume data.

B.1.4 – Wisconsin DOT Balancing Tool and Remaining Volume Imbalance Resolutions

Once initial values have been assigned to all known and unknown intersections, the next step is to perform the volume balancing. The *Traffic Engineering, Operations & Safety Manual*, developed by the Wisconsin DOT (2019a), provides an efficient process to effectively balance traffic volumes in a network and reduce inconsistencies between adjacent intersections. This process involves assembling an initial set of traffic volume counts, including their turning proportions, inputting this data into their volume balancing tool, and then reviewing and adjusting remaining imbalances to eliminate volume inconsistencies (Wisconsin DOT, 2019a). Thus, this process and tool is used in this thesis to balance the volumes in the traffic network. The data required for this tool, including initial volume counts and turning proportions, is already collected and estimated through the procedures outlined in *Sections B.1.1* through *B.1.3*, where the data was converted to 2022 year data and estimates were calculated for unknown intersections. The next step is to set up the network within the balancing tool. Within the balancing tool, templates of multiple intersection and interchange types are provided for the user to build a traffic network (Wisconsin DOT, 2019b). Using templates of various types of 3-legged and 4-legged intersections, a schematic is developed for the current route's thirteen intersections. After drawing the network in the tool's Excel file, the balancing tool initializes the network through an Excel macro, where each intersection is assigned an intersection number, lists are compiled of possible turning maneuvers for each intersection, and origin-destination (OD) matrices are developed for each intersection (Wisconsin DOT, 2019b). Once initialization is complete, the raw data for the volumes for each turn at each intersection are entered in a list, from which the tool will automatically transfer the initial volumes to the schematic (Wisconsin DOT, 2019b). Taking the traffic volumes generated in the previous report sections, a list of initial volumes is shown in *Table 28*.

Table 28: Initial Traffic Volume Counts

Int. #	Intersection	Top Leg ↓			Right Leg ←			Bottom Leg ↑			Left Leg →		
		Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left
1	Highway 401 EB Off-Ramp & Dixie		1554						1348		405		183
2	Highway 401 WB Off-Ramp & Dixie		1727		566		440		1047				
3	Shawson & Dixie	59	1459	99	142	53	456	185	1191	117	247	47	68
4	Britannia & Dixie	144	1160	108	124	165	230	150	1108	99	197	86	70
5	Britannia & Vipond	116		118	102	306						267	89
6	Britannia & Shawson	79	134	80	127	394	74	50	165	213	86	226	34
7	Westport Crescent South & Shawson	74	173	76	61	66	59	48	244	34	61	66	59
8	Westport Crescent North & Shawson	50	145						490	165	55		57
9	Meyerside & Shawson	56	91	14	78	418	47	81	296	170	57	170	14
10	Tristar & Shawson	13	38						291	97	110		112
11	Courtneypark & Shawson	39	0	16	24	1070	23	45	0	153	28	363	60
12	Courtneypark & Tomken	375	471	91	137	1307	25	32	1244	597	170	384	131
13	Highway 410 NB & Courtneypark	50		50	46	1704	529	478	21	507	330	1061	28

By transferring these values into the schematic, the balancing tool automatically fills in each OD matrix, calculates the volume input from each leg of each intersection, and calculates the volume imbalance between adjacent intersections (Wisconsin DOT, 2019b). From here, a built-in Excel macro is available to the user to balance the traffic volumes in the network. This macro works by first evaluating the existing imbalances between each intersection and calculates the average between the traffic volumes on each side of the imbalance (Wisconsin DOT, 2019b). For example, if the volume output from one intersection to the next is expected to be 100, and the volume input of the adjacent intersection is expected to be 150, the imbalance between the two is 50 vehicles. In this case, the average between these two adjacent intersections along this junction is 125. These averages at each intersection are then set as targets for the tool to achieve in order to eliminate the imbalance between the two intersections (Wisconsin DOT, 2019b). The macro then utilizes Excel's goal-seeking optimization capabilities by setting a maximum network imbalance as the primary target of the macro, with these averages between intersections set as conditions to be met by iteratively adjusting input volumes and turning proportions within the network (Wisconsin DOT, 2019a; Wisconsin DOT, 2019b). This goal-seeking optimization multiplies the rows and columns of each OD matrix until the row and column sums meet the targeted average values. With each iteration, the averages between imbalanced junctions will be re-evaluated and set as new targets to be met each time (Wisconsin DOT, 2019b). The user is able to set a maximum total network imbalance and a maximum number of iterations, so the macro will continue to run until either the maximum number of iterations or network imbalance is achieved.

In using this tool, a maximum number of iterations of 15000 is used, while a maximum network imbalance is set to the default value of 0.01. From these settings, the total network imbalance is reduced from 3730 to 7. It should be noted that the tool warns that a higher number of iterations will not necessarily achieve a lower network imbalance (Wisconsin DOT, 2019b), so after running the tool with multiple maximum iterations values, 15000 iterations yielded the best results.

The final step of utilizing the balancing tool involves reviewing the resulting turning volumes compared to their initial values and resolving the remaining imbalances manually. The remaining imbalances, all of which are imbalances of one vehicle sparsely occurring throughout the schematic, are each manually resolved by slightly increasing or decreasing a corresponding turning movement by one to eliminate the imbalance. This is completed seven times to reduce the total imbalance to zero. A list is created in the excel file showcasing for each turning movement at each intersection the initial traffic volume, the balanced traffic volume, difference between the two, and the root normalized square error (RNSE). The RNSE value is an error calculation that assesses the difference between the initial and balanced traffic volume values, using the equation below,

Equation 4: $$RNSE = \sqrt{\frac{(y_1 - y_0)^2}{y_0}}$$

where y_0 represents the initial traffic volume for a selected turning motion before volume balancing is completed and y_1 is the final balanced traffic volume for the same turning motion. In general, a RNSE value less than 5 is acceptable, while a RNSE value greater than 5 should be investigated further (Wisconsin DOT, 2019b). Overall, only 18 out of the 121 possible turning movements exceed this RNSE threshold, which is acceptable for this thesis project.

Table 29 exhibits the output from the Wisconsin DOT's balancing tool. Here, all the initial and balanced turning volumes are shown, along with the differences between the two and the calculated RNSE value. It is important to note that in the balanced volume column, any values highlighted in green are manually increased by 1 vehicle to eliminate a remaining imbalance of 1. Meanwhile, any value highlighted in red is reduced by 1 vehicle to eliminate a remaining imbalance. Furthermore, in the RNSE column, cells highlighted in yellow are approaching the RNSE = 5 threshold and values in red cells have exceeded this threshold.

Table 29: Balanced Volume Inputs Used in Vissim

Int. #	Intersection	Turn	Volume			RNSE
			Raw (y_0)	Balanced (y_1)	Difference ($y_1 - y_0$)	
1	Highway 401 EB Off-Ramp & Dixie	NBT	1348	1127	-221	6.0
		SBT	1554	2086	532	13.5
		EBL	183	108	-75	5.5
		EBR	405	370	-35	1.7
2	Highway 401 WB Off-Ramp & Dixie	NBT	1047	1235	188	5.8
		WBL	440	304	-136	6.5
		WBR	566	416	-150	6.3
		SBT	1727	1782	55	1.3
3	Shawson & Dixie	NBL	117	147	30	2.8
		NBT	1191	1274	83	2.4
		NBR	185	230	45	3.3
		WBL	456	336	-120	5.6
		WBT	53	59	6	0.8
		WBR	142	134	-8	0.7
		SBL	99	127	28	2.8
		SBT	1459	1266	-193	5.1
		SBR	59	77	18	2.3
		EBL	68	63	-5	0.6
		EBT	47	51	4	0.6
		EBR	247	180	-67	4.3
4	Britannia & Dixie	NBL	99	115	16	1.6
		NBT	1108	1201	93	2.8
		NBR	150	155	5	0.4
		WBL	230	195	-35	2.3
		WBT	165	175	10	0.8
		WBR	124	123	-1	0.1
		SBL	108	112	4	0.4
		SBT	1160	1075	-85	2.5
		SBR	144	166	22	1.8
		EBL	70	83	13	1.6
		EBT	86	98	12	1.3
		EBR	197	200	3	0.2

Int. #	Intersection	Turn	Volume			RNSE
			Raw (y_0)	Balanced (y_1)	Difference ($y_1 - y_0$)	
5	Britannia & Vipond	WBT	306	357	51	2.9
		WBR	102	99	-3	0.3
		SBL	118	116	-2	0.2
		SBR	116	134	18	1.7
		EBL	89	87	-2	0.2
		EBT	267	265	-2	0.1
6	Britannia & Shawson	NBL	213	217	4	0.3
		NBT	165	212	47	3.7
		NBR	50	56	6	0.8
		WBL	74	67	-7	0.8
		WBT	394	302	-92	4.6
		WBR	127	122	-5	0.4
		SBL	80	79	-1	0.1
		SBT	134	142	8	0.7
		SBR	79	71	-8	0.9
		EBL	34	37	3	0.5
		EBT	226	217	-9	0.6
		EBR	86	88	2	0.2
7	Westport Crescent South & Shawson	NBL	34	26	-8	1.4
		NBT	244	309	65	4.2
		NBR	48	36	-12	1.7
		WBL	59	69	10	1.3
		WBT	66	68	2	0.2
		WBR	61	105	44	5.6
		SBL	76	57	-19	2.2
		SBT	173	151	-22	1.7
		SBR	74	57	-17	2.0
		EBL	59	102	43	5.6
		EBT	66	68	2	0.2
		EBR	61	72	11	1.4
8	Westport Crescent North & Shawson	NBL	165	116	-49	3.8
		NBT	490	400	-90	4.1
		SBT	145	178	33	2.7
		SBR	50	33	-17	2.4
		EBL	57	57	0	0.0
		EBR	55	87	32	4.3
9	Meyerside & Shawson	NBL	170	138	-32	2.5
		NBT	296	251	-45	2.6
		NBR	81	68	-13	1.4

Int. #	Intersection	Turn	Volume			RNSE
			Raw (y_0)	Balanced (y_1)	Difference ($y_1 - y_0$)	
		WBL	47	54	7	1.0
		WBT	418	429	11	0.5
		WBR	78	83	5	0.6
		SBL	14	13	-1	0.3
		SBT	91	93	2	0.2
		SBR	56	51	-5	0.7
		EBL	14	14	0	0.0
		EBT	170	172	2	0.2
		EBR	57	64	7	0.9
10	Tristar & Shawson	NBL	97	115	18	1.8
		NBT	291	233	-58	3.4
		SBT	38	39	1	0.2
		SBR	13	15	2	0.6
		EBL	112	91	-21	2.0
		EBR	110	118	8	0.8
11	Courtneypark & Shawson	NBL	153	241	88	7.1
		NBT	0	0	0	-
		NBR	45	83	38	5.7
		WBL	23	24	1	0.2
		WBT	1070	1161	91	2.8
		WBR	24	30	6	1.2
		SBL	16	18	2	0.5
		SBT	0	0	0	-
		SBR	39	36	-3	0.5
		EBL	60	76	16	2.1
		EBT	363	470	107	5.6
		EBR	28	30	2	0.4
12	Courtneypark & Tomken	NBL	597	598	1	0.0
		NBT	1244	1298	54	1.5
		NBR	32	29	-3	0.5
		WBL	25	44	19	3.8
		WBT	1307	1257	-50	1.4
		WBR	137	137	0	0.0
		SBL	91	52	-39	4.1
		SBT	471	521	50	2.3
		SBR	375	225	-150	7.7
		EBL	131	186	55	4.8
		EBT	384	495	111	5.7
		EBR	170	427	257	19.7

Int. #	Intersection	Turn	Volume			RNSE
			Raw (y_0)	Balanced (y_1)	Difference ($y_1 - y_0$)	
13	Courtneypark & Highway 410 NB	NBL	507	537	30	1.3
		NBT	21	28	7	1.5
		NBR	478	386	-92	4.2
		WBL	529	601	72	3.1
		WBT	1704	1431	-273	6.6
		WBR	46	48	2	0.3
		SBL	50	41	-9	1.3
		SBR	50	53	3	0.4
		EBL	28	29	1	0.2
		EBT	1061	681	-380	11.7
		EBR	330	375	45	2.5

Now that all volumes in the network are balanced, the last issue to resolve before entering the values in Vissim pertains to the volume travelling through Westport Crescent between the two Shawson Drive intersections. Since the volume balancing tool does not recognize the continuity between the northern and southern intersections via a crescent road, the volumes entering one side of the crescent are not balanced with the volumes exiting the other side of the crescent. Therefore, to resolve this remaining imbalance, driveways within the crescent are used as additional traffic sources and sinks to resolve the remaining imbalances. In the northbound direction, an imbalance of -7 is present, so a driveway is used as a volume sink, with 7 vehicles turning into the driveway. In the southbound direction, an imbalance of 95 is present, so another driveway will generate a traffic volume of 95 vehicles turning onto Westport Crescent.

Again, one of the limitations present throughout this volume balancing process is the additional assumptions about the volumes and RNSE values, as these assumptions may not be perfectly representative of the true traffic conditions. However, as discussed at the end of *Section B.1.3*, the network performance is not the focus of this thesis. Since these traffic volumes are being generated for the purposes of developing a base scenario, the volumes used should be acceptable given they are reasonable. All other scenarios studying LCVs will then be assessed in comparison to this established baseline to determine safety concerns.

With a fully balanced set of traffic volumes, these volumes can now be applied to the Vissim model as vehicle inputs. These vehicle inputs are placed throughout the network on links that enter the network. For example, at a 4-legged intersection located between two adjacent intersections, vehicle inputs are only placed on the links entering that intersection that do not connect with the adjacent intersections.

B.2 – Calculated Vehicle Compositions

The following table showcases the full results for the calculation of vehicle compositions for each vehicle input for unknown intersections, as well as the vehicle source created in a driveway along Westport Crescent.

Table 30: Vehicle Composition Calculation for Unknown Intersections and Traffic Sources

Intersection or Traffic Source	Adjacent Known Vehicle Compositions	Truck Percentage on Known Road	Average Truck Percentage	Final Vehicle Proportions		
				Car	LGV	HGV
Westport Cr South and Shawson in the WB Direction	Shawson SB at Britannia	12.28%	12.64%	87.36%	6.32%	6.32%
	Shawson NB at Meyerside	12.99%				
Vehicle Source in Westport Cr Driveway	Shawson SB at Britannia	12.28%	12.64%	87.36%	6.32%	6.32%
	Shawson NB at Meyerside	12.99%				
Vipond and Britannia in the SB Direction	Britannia EB at Dixie	25.78%	20.43%	79.57%	10.21%	10.21%
	Britannia WB at Shawson	15.08%				
Tristar and Shawson in the EB Direction	Shawson SB at Meyerside	17.31%	9.46%	90.54%	4.73%	4.73%
	Shawson NB at Courtneypark	1.62%				

B.3 – Freight Vehicle Engine Power Summary

The following table summarizes the vehicle models and engine models for six prominent truck brands in Ontario, including their engine horsepower.

Table 31: Modern Truck Engine Power

Brand	Model	Engine	Power (hp)
International Trucks (2024)	LT Series	International S13	515
		International A26	515
		Cummins X15	565
Mack Trucks (2024)	Anthem	MP7	325 - 425
		MP8	415 - 505
		MP8HE	415 - 445
	Pinnacle	MP8	415 - 505
Peterbilt Trucks (2024)	579	Paccar MX-13	Max 605
		Paccar MX-11	Max 605
		Paccar PX-9	Max 605
	589	Paccar MX-13	405 - 605
		Paccar X15	405 - 605
Freightliner Trucks (2024)	Cascadia	Detroit DD13	370 - 525
		Detroit DD15	400 - 505
		Detroit DD16	500 - 600
		Cummins X12	Up to 500
		Cummins X15	Up to 500
	eCascadia	Detroit ePowertrain	195 - 395
	Cascadia Natural Gas	Cummins Westport ISX12N	400
Volvo Trucks (2024)	VNL 300	Volvo D13TC	405 - 455
		Volvo D13	405 - 500
		Cummins X15	400 - 565
	VNL 400	Volvo D13TC	405 - 455
		Volvo D13	405 - 500
		Cummins X15	400 - 565
	VNL 740	Volvo D13TC	405 - 455
		Volvo D13	500
		Cummins X15	400 - 565
	VNL 760	Volvo D13TC	405 - 455
		Volvo D13	500
		Cummins X15	400 - 565
	VNL 860	Volvo D13TC	405 - 455
		Volvo D13	500
		Cummins X15	400 - 565

Brand	Model	Engine	Power (hp)
	VNR 300	Volvo D11	325 - 425
		Volvo D13	405 - 500
	VNR 400	Volvo D11	325 - 425
		Volvo D13	405 - 500
	VNR 640	Volvo D11	325 - 425
		Volvo D13	405 - 500
	VNR 660	Volvo D11	325 - 425
		Volvo D13	405 - 500
	VNR Electric	VNR Electric Drivetrain	455
Kenworth Trucks (2024)	T680	Paccar MX-13	405 - 510
	W990	Paccar MX-13	405 - 510
	T680 Fuel Cell EV	Toyota 310kW Dual Motor	415
	T680E	Meritor 14XE	536

B.4 – GEH Statistics for Microsimulation Model Calibration

The following table showcases the results of the GEH statistic calculations for each turning movement in the network. A green-shaded value indicates that the GEH value is less than 5 and that the model is a good fit for this turning movement. A yellow-shaded value indicates that the GEH value lies between 5 and 10 and that the model may or may not be a good fit for these turning movements. A red-shaded value indicates that the GEH value is over 10 and is not a good fit for this turning movement. A summary of the total results is included in the final four rows of the table.

Table 32: Calculated GEH Values

Int. #	Intersection	Turning Movement	Model Volume [E]	Field Count (2022 Data) [V]	GEH
1	Dixie Rd and Highway 401 EB Off Ramp	NBT	1118	1127	0.3
		SBT	2104	2086	0.4
		EBL	133	108	2.3
		EBR	345	370	1.3
2	Dixie Rd and Hwy 401 WB Off-Ramp	NBT	1242	1235	0.2
		SBT	1814	1782	0.8
		WBL	298	304	0.3
		WBR	439	416	1.1
3	Dixie Rd and Shawson Dr	NBT	1262	1274	0.3
		NBR	246	230	1.0
		NBL	157	147	0.8
		SBL	130	127	0.3
		SBT	1278	1266	0.3
		SBR	71	77	0.7
		EBL	63	63	0
		EBR	190	180	0.7
		EBT	54	51	0.4
		WBR	129	134	0.4
		WBL	353	336	0.9
		WBT	53	59	0.8
4	Dixie Rd and Britannia Rd	WBL	211	195	1.1
		WBR	133	123	0.9
		WBT	179	175	0.3
		NBR	161	155	0.5
		NBT	1116	1201	2.5
		NBL	168	115	4.5
		SBT	1062	1075	0.4

Int. #	Intersection	Turning Movement	Model Volume [E]	Field Count (2022 Data) [V]	GEH
		SBL	125	112	1.2
		SBR	181	166	1.1
		EBR	211	200	0.8
		EBT	91	98	0.7
		EBL	91	83	0.9
5	Britannia Rd and Vipond Dr	WBR	116	99	1.7
		WBT	414	357	2.9
		SBL	124	116	0.7
		SBR	133	134	0.1
		EBT	266	265	0.1
		EBL	86	87	0.1
6	Britannia Rd and Shawson Dr	EBT	219	217	0.1
		EBL	32	37	0.9
		EBR	78	88	1.1
		WBT	311	302	0.5
		WBR	123	122	0.1
		WBL	23	67	6.6
		NBR	57	56	0.1
		NBL	224	217	0.5
		NBT	228	212	1.1
		SBL	75	79	0.5
		SBR	73	71	0.2
		SBT	134	142	0.7
7	Shawson Dr and Westport Cr S	NBR	43	36	1.1
		NBL	22	26	0.8
		NBT	318	309	0.5
		WBL	68	69	0.1
		WBT	63	68	0.6
		WBR	104	105	0.1
		EBR	73	72	0.1
		EBT	68	68	0
		EBL	99	102	0.3
		SBT	141	151	0.8
		SBL	49	57	1.1
		SBR	57	57	0
8	Shawson Dr and Westport Cr N	EBR	81	87	0.7
		EBL	56	57	0.1
		NBL	120	116	0.4
		NBT	400	400	0
		SBR	30	33	0.5

Int. #	Intersection	Turning Movement	Model Volume [E]	Field Count (2022 Data) [V]	GEH
		SBT	165	178	1.0
9	Shawson Dr and Meyerside Dr	NBR	65	68	0.4
		NBL	136	138	0.2
		NBT	254	251	0.2
		WBL	49	54	0.7
		WBT	415	429	0.7
		WBR	78	83	0.6
		EBR	59	64	0.6
		EBT	169	172	0.2
		EBL	11	14	0.8
		SBT	87	93	0.6
		SBL	18	13	1.3
		SBR	49	51	0.3
10	Shawson Dr and Tristar Dr	NBT	239	233	0.4
		NBL	106	115	0.9
		SBT	31	39	1.4
		SBR	14	15	0.3
		EBL	85	91	0.6
		EBR	122	118	0.4
11	Shawson Dr and Courtneypark Dr	SBL	15	18	0.7
		SBT	0	0	0
		SBR	39	36	0.5
		NBR	79	83	0.4
		NBT	0	0	0
		NBL	244	241	0.2
		EBT	454	470	0.7
		EBL	70	76	0.7
		EBR	24	30	1.2
		WBR	25	30	1.0
		WBL	22	24	0.4
		WBT	1049	1161	3.4
12	Courtneypark Dr and Tomken Rd	WBL	47	44	0.4
		WBR	128	137	0.8
		WBT	1107	1257	4.4
		EBT	467	495	1.3
		EBR	379	427	2.4
		EBL	167	186	1.4
		SBL	57	52	0.7
		SBT	519	521	0.1
		NBR	25	29	0.8

Int. #	Intersection	Turning Movement	Model Volume [E]	Field Count (2022 Data) [V]	GEH
		NBT	928	1298	11.1
		SBR	218	225	0.5
		NBL	419	598	7.9
13	Courtneypark Dr and Highway 410 On/Off-Ramp	SBL	43	41	0.3
		SBR	52	53	0.1
		WBL	501	601	4.3
		WBR	48	48	0
		WBT	1205	1431	6.2
		NBR	291	386	5.2
		NBT	18	28	2.1
		NBL	388	537	6.9
		EBL	26	29	0.6
		EBT	694	681	0.5
		EBR	363	375	0.6

GEH Threshold	Count	Percentage
GEH < 5	115	95.1 %
5 < GEH < 10	5	4.1 %
GEH > 10	1	0.8 %

B.5 – Vehicle Inputs and Compositions for Each LCV Penetration Level

The following table exhibits the final calculated vehicle compositions and vehicle totals calculated for each LCV penetration level. The vehicle totals, highlighted with bold text, are used as the vehicle inputs in the simulation, but only after being rounded up to the next integer value. It should also be noted that the Shawson Drive southbound vehicle input at Courtneypark Drive and the carpool vehicle input, highlighted with italicized text in the table, both contain no heavy goods vehicles (HGV) in their base scenario. Thus, these proportions do not change throughout each LCV penetration level since no LCVs are added at these locations.

Table 33: Vehicle Inputs and Compositions for LCV Penetration Levels

Vehicle Input	Vehicle Class	0% Proportions (Data From Mississauga and Peel)		Updated Proportions				
				0%	25%	50%	75%	100%
Hwy 401 EB onto Dixie Road	Car	451.0	0.77	366.6	366.6	366.6	366.6	366.6
	LGV	53.0	0.09	43.1	43.1	43.1	43.1	43.1
	HGV	84.0	0.14	68.3	51.2	34.1	17.1	0.0
	LCV	0.0	0.00	0.0	8.5	17.1	25.6	34.1
	Vehicle Total	588.0		**478.0**	**469.5**	**460.9**	**452.4**	**443.9**
Dixie Road NB near 401 EB Off-Ramp	Car	1174.0	0.87	980.8	980.8	980.8	980.8	980.8
	LGV	93.0	0.07	77.7	77.7	77.7	77.7	77.7
	HGV	75.0	0.06	62.7	47.0	31.3	15.7	0.0
	Bus	7.0	0.01	5.9	5.9	5.9	5.9	5.9
	LCV	0.0	0.00	0.0	7.8	15.7	23.5	31.3
	Vehicle Total	1349.0		**1127.0**	**1119.2**	**1111.3**	**1103.5**	**1095.7**
Hwy 401 WB Off-Ramp onto Dixie Road	Car	696.0	0.69	498.1	498.1	498.1	498.1	498.1
	LGV	139.0	0.14	99.5	99.5	99.5	99.5	99.5
	HGV	170.0	0.17	121.7	91.3	60.8	30.4	0.0
	Bus	1.0	0.00	0.7	0.7	0.7	0.7	0.7
	LCV	0.0	0.00	0.0	15.2	30.4	45.6	60.8
	Vehicle Total	1006.0		**720.0**	**704.8**	**689.6**	**674.4**	**659.2**
Shawson WB at Dixie Road	Car	569.0	0.87	462.4	462.4	462.4	462.4	462.4
	LGV	41.0	0.06	33.3	33.3	33.3	33.3	33.3
	HGV	41.0	0.06	33.3	25.0	16.7	8.3	0.0
	LCV	0.0	0.0	0.0	4.2	8.3	12.5	16.7
	Vehicle Total	651.0		**529.0**	**524.8**	**520.7**	**516.5**	**512.3**
Shawson EB at Dixie Road	Car	313.0	0.86	254.2	254.2	254.2	254.2	254.2
	LGV	29.0	0.08	23.6	23.6	23.6	23.6	23.6
	HGV	20.0	0.06	16.2	12.2	8.1	4.1	0.0
	LCV	0.0	0.00	0.0	2.0	4.1	6.1	8.1

Vehicle Input	Vehicle Class	0% Proportions (Data From Mississauga and Peel)		Updated Proportions				
				0%	25%	50%	75%	100%
	Vehicle Total	362.0		**294.0**	**292.0**	**289.9**	**287.9**	**285.9**
Britannia WB at Dixie Road	Car	409.0	0.79	388.5	388.5	388.5	388.5	388.5
	LGV	58.0	0.11	55.1	55.1	55.1	55.1	55.1
	HGV	52.0	0.10	49.4	37.1	24.7	12.4	0.0
	LCV	0.0	0.00	0.0	6.2	12.4	18.5	24.7
	Vehicle Total	519.0		**493.4**	**486.8**	**480.7**	**474.5**	**468.3**
Dixie SB at Britannia Road	Car	1208.0	0.85	1154.3	1154.3	1154.3	1154.3	1154.3
	LGV	111.0	0.08	106.1	106.1	106.1	106.1	106.1
	HGV	92.0	0.06	87.9	65.9	44.0	22.0	0.0
	Bus	5.0	0.00	4.8	4.8	4.8	4.8	4.8
	LCV	0.0	0.00	0.0	11.0	22.0	33.0	44.0
	Vehicle Total	1416.0		**1353.0**	**1342.0**	**1331.0**	**1320.0**	**1309.0**
Britannia EB at Shawson Drive	Car	267.0	0.80	272.6	272.6	272.6	272.6	272.6
	LGV	34.0	0.10	34.7	34.7	34.7	34.7	34.7
	HGV	34.0	0.10	34.7	26.0	17.4	8.7	0.0
	LCV	0.0	0.00	0.0	4.3	8.7	13.0	17.4
	Vehicle Total	335.0		**342.0**	**337.7**	**333.3**	**329.0**	**324.6**
Shawson NB at Britannia Road	Car	368.0	0.88	429.0	429.0	429.0	429.0	429.0
	LGV	24.0	0.06	28.0	28.0	28.0	28.0	28.0
	HGV	24.0	0.06	28.0	21.0	14.0	7.0	0.0
	LCV	0.0	0.00	0.0	3.5	7.0	10.5	14.0
	Vehicle Total	416.0		**485.0**	**481.5**	**478.0**	**474.5**	**471.0**
Meyerside WB at Shawson Drive	Car	484.0	0.92	518.8	518.8	518.8	518.8	518.8
	LGV	22.0	0.04	23.6	23.6	23.6	23.6	23.6
	HGV	22.0	0.04	23.6	17.7	11.8	5.9	0.0
	LCV	0.0	0.00	0.0	3.0	5.9	8.8	11.8
	Vehicle Total	528.0		**566.0**	**563.1**	**560.1**	**557.2**	**554.2**
Meyerside EB at Shawson Drive	Car	202.0	0.86	215.8	215.8	215.8	215.8	215.8
	LGV	16.0	0.07	17.1	17.1	17.1	17.1	17.1
	HGV	16.0	0.07	17.1	12.8	8.6	4.3	0.0
	LCV	0.0	0.00	0.0	2.1	4.3	6.4	8.6
	Vehicle Total	234.0		**250.0**	**247.9**	**245.7**	**243.6**	**241.5**
Shawson SB at Courtneypark Drive	*Car*	*1.00*	***Does Not Change***					
	Vehicle Total	*54.0*						
Courtneypark WB at Shawson Drive	Car	1017.0	0.97	1179.1	1179.1	1179.1	1179.1	1179.1
	LGV	12.0	0.01	13.9	13.9	13.9	13.9	13.9
	HGV	12.0	0.01	13.9	10.4	7.0	3.5	0.0
	Bus	7.0	0.01	8.1	8.1	8.1	8.1	8.1

Vehicle Input	Vehicle Class	0% Proportions (Data From Mississauga and Peel)		Updated Proportions				
				0%	25%	50%	75%	100%
	LCV	0.0	0.00	0.0	1.7	3.5	5.2	7.0
	Vehicle Total	1048.0		**1215.0**	**1213.3**	**1211.5**	**1209.8**	**1208.0**
Tomken NB at Courtneypark Drive	Car	1652.0	0.9	1759.9	1759.9	1759.9	1759.9	1759.9
	LGV	74.0	0.04	78.8	78.8	78.8	78.8	78.8
	HGV	74.0	0.04	78.8	59.1	39.4	19.7	0.0
	Bus	7.0	0.00	7.5	7.5	7.5	7.5	7.5
	LCV	0.0	0.00	0.0	9.9	19.7	29.6	39.4
	Vehicle Total	1807.0		**1925.0**	**1915.2**	**1905.3**	**1895.4**	**1885.6**
Tomken SB at Courtneypark Drive	Car	781.0	0.86	687.1	687.1	687.1	687.1	687.1
	LGV	59.5	0.07	52.4	52.4	52.4	52.4	52.4
	HGV	59.5	0.07	52.4	39.3	26.2	13.1	0.0
	Bus	7.0	0.01	6.2	6.2	6.2	6.2	6.2
	LCV	0.0	0.00	0.0	6.5	13.1	19.6	26.3
	Vehicle Total	907.0		**798.0**	**791.5**	**784.9**	**778.4**	**771.8**
Carpool	*Car*	*0.96*	***Does Not Change***					
	LGV	*0.02*						
	Bus	*0.02*						
	Vehicle Total	*94.0*						
Hwy 410 NB onto Courtneypark Drive	Car	451.0	0.77	729.4	729.4	729.4	729.4	729.4
	LGV	53.0	0.09	85.7	85.7	85.7	85.7	85.7
	HGV	84.0	0.14	135.9	101.9	67.9	34.0	0.0
	LCV	0.0	0.0	0.0	17.0	34.0	51.0	67.9
	Vehicle Total	588.0		**951.0**	**934.0**	**917.0**	**900.1**	**883.1**
Courtneypark EB at Hwy 410	Car	1174.0	0.87	944.3	944.3	944.3	944.3	944.3
	LGV	93.0	0.07	74.8	74.8	74.8	74.8	74.8
	HGV	75.0	0.06	60.3	45.2	30.2	15.1	0.0
	Bus	7.0	0.01	5.6	5.6	5.6	5.6	5.6
	LCV	0.0	0.00	0.0	7.5	15.1	22.6	30.2
	Vehicle Total	1349.0		**1085.0**	**1077.5**	**1069.9**	**1062.4**	**1054.8**
Vipond SB	Car	198.3	0.80	198.9	198.9	198.9	198.9	198.9
	LGV	25.53	0.10	25.5	25.5	25.5	25.5	25.5
	HGV	25.5	0.10	25.5	19.1	12.8	6.4	0.0
	LCV	0.0	0.00	0.0	3.2	6.4	9.6	12.8
	Vehicle Total	250.0		**250.0**	**246.8**	**243.6**	**240.4**	**237.2**
Westport Crescent Source (Traffic Source)	Car	81.3	0.87	81.3	81.3	81.3	81.3	81.3
	LGV	5.9	0.06	5.9	5.9	5.9	5.9	5.9
	HGV	5.9	0.06	5.9	4.4	2.9	1.5	0.0
	LCV	0.0	0.00	0.0	0.7	1.5	2.2	3.0
	Vehicle Total	93.0		**93.0**	**92.3**	**91.5**	**90.8**	**90.1**

Vehicle Input	Vehicle Class	0% Proportions (Data From Mississauga and Peel)		Updated Proportions				
				0%	25%	50%	75%	100%
Tristar EB	Car	189.2	0.91	189.2	189.2	189.2	189.2	189.2
	LGV	9.8	0.05	9.8	9.8	9.8	9.8	9.8
	HGV	10.0	0.05	10.0	7.5	5.0	2.5	0.0
	LCV	0	0.00	0.0	1.3	2.5	3.8	5.0
	Vehicle Total	209.0		**209.0**	**207.8**	**206.5**	**205.2**	**204.0**
Westport Crescent Source (Westport Crescent South WB)	Car	211.5	0.87	211.5	211.5	211.5	211.5	211.5
	LGV	15.3	0.06	15.3	15.3	15.3	15.3	15.3
	HGV	15.3	0.06	15.3	11.4	7.6	3.8	0.0
	LCV	0	0.00	0.0	1.9	3.8	5.7	7.6
	Vehicle Total	242.0		**242.0**	**240.1**	**238.2**	**236.3**	**234.4**

B.6 – Freight Left Turns at Each Intersection

The following table shows the number of freight left turns completed in each intersection at each LCV penetration level.

Table 34: Freight Left Turns at Each Intersection by LCV Level

LCV Penetration Level		0%	25%	50%	75%	100%
Freight Left Turns	Intersection 1	137	116	100	90	73
	Intersection 2	107	92	74	77	49
	Intersection 3	215	189	167	209	106
	Intersection 4	228	204	170	212	114
	Intersection 5	61	57	48	84	28
	Intersection 6	90	79	71	160	55
	Intersection 7	45	47	28	126	25
	Intersection 8	44	38	26	66	20
	Intersection 9	35	36	30	124	22
	Intersection 10	32	34	24	88	12
	Intersection 11	61	49	50	184	31
	Intersection 12	158	123	110	207	79
	Intersection 13	299	263	218	271	146

B.7 – Intersection-Level Vissim Results

The following sections outline the TTC, PET, and conflict type results for each of the six identified high-conflict intersections.

B.7.1 – Intersection 2 Results: Dixie Road and Highway 401 Westbound Off-Ramp

Table 35: Intersection 2 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	0 (0.0%)	1 (100.0%)	0 (0.0%)	0 (0.0%)
25%	41 (83.7%)	7 (14.3%)	0 (0.0%)	1 (2.0%)
50%	33 (80.5%)	6 (14.6%)	2 (4.9%)	0 (0.0%)
75%	76 (82.6%)	13 (14.1%)	0 (0.0%)	3 (3.3%)
100%	94 (84.7%)	12 (10.8%)	2 (1.8%)	3 (2.7%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 36: Intersection 2 Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	1 (100.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
25%	41 (83.7%)	8 (16.3%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
50%	33 (80.5%)	7 (17.1%)	1 (2.4%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
75%	76 (82.6%)	14 (15.2%)	2 (2.2%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
100%	94 (84.7%)	15 (13.5%)	1 (0.9%)	1 (0.9%)	0 (0.0%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 37: Intersection 2 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	0 (0.0%)	1 (100.0%)	0 (0.0%)
25%	2 (4.1%)	33 (67.4%)	14 (28.6%)
50%	0 (0.0%)	27 (65.9%)	14 (34.2%)
75%	2 (2.2%)	59 (64.1%)	31 (33.7%)
100%	1 (0.9%)	75 (67.6%)	35 (31.5%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

B.7.2 – Intersection 3 Results: Dixie Road and Shawson Drive

Table 38: Intersection 3 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	14 (51.9%)	4 (14.8%)	1 (3.7%)	8 (29.6%)
25%	96 (83.5%)	10 (8.7%)	1 (0.9%)	8 (7.0%)
50%	105 (74.5%)	30 (21.3%)	2 (1.4%)	4 (2.8%)
75%	165 (78.6%)	30 (14.3%)	4 (1.9%)	11 (5.2%)
100%	162 (89.5%)	15 (8.3%)	1 (0.6%)	3 (1.7%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 39: Intersection 3 Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	16 (59.3%)	4 (14.8%)	2 (7.4%)	4 (14.8%)	1 (3.7%)	0 (0.0%)
25%	97 (84.4%)	8 (7.0%)	5 (4.4%)	4 (3.5%)	1 (0.9%)	0 (0.0%)
50%	108 (76.6%)	16 (11.4%)	12 (8.5%)	1 (0.7%)	3 (2.1%)	1 (0.7%)
75%	169 (80.5%)	21 (10.0%)	10 (4.8%)	6 (2.9%)	2 (1.0%)	2 (1.0%)
100%	162 (89.5%)	8 (4.4%)	7 (3.9%)	2 (1.1%)	2 (1.1%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 40: Intersection 3 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	9 (33.3%)	16 (59.3%)	2 (7.4%)
25%	40 (34.8%)	72 (62.6%)	3 (2.6%)
50%	48 (34.0%)	80 (56.7%)	13 (9.2%)
75%	81 (38.6%)	110 (52.4%)	19 (9.1%)
100%	71 (39.2%)	85 (47.0%)	25 (13.8%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

B.7.3 – Intersection 4 Results: Dixie Road and Britannia Road

Table 41: Intersection 4 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	7 (36.8%)	1 (5.3%)	1 (5.3%)	10 (52.6%)
25%	61 (77.2%)	8 (10.1%)	3 (3.8%)	7 (8.9%)
50%	106 (79.1%)	20 (14.9%)	3 (2.2%)	5 (3.7%)
75%	176 (83.8%)	26 (12.4%)	2 (1.0%)	6 (2.9%)
100%	213 (84.2%)	21 (8.3%)	2 (0.8%)	17 (6.7%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 42: Intersection 4 Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	7 (36.8%)	2 (10.5%)	5 (26.3%)	4 (21.1%)	1 (5.3%)	0 (0.0%)
25%	61 (77.2%)	7 (8.9%)	6 (7.6%)	1 (1.3%)	3 (3.8%)	1 (1.3%)
50%	108 (80.6%)	9 (6.7%)	13 (9.7%)	3 (2.2%)	1 (0.8%)	0 (0.0%)
75%	177 (84.3%)	14 (6.7%)	11 (5.2%)	1 (0.5%)	4 (1.9%)	3 (1.4%)
100%	213 (84.2%)	16 (6.3%)	9 (3.6%)	2 (0.8%)	11 (4.4%)	2 (0.8%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 43: Intersection 4 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	15 (79.0%)	4 (21.1%)	0 (0.0%)
25%	51 (64.6%)	23 (29.1%)	5 (6.3%)
50%	82 (61.2%)	37 (27.6%)	15 (11.2%)
75%	130 (61.9%)	62 (29.5%)	18 (8.6%)
100%	172 (68.0%)	66 (26.1%)	15 (5.9%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

B.7.4 – Intersection 11 Results: Courtneypark Drive and Shawson Drive

Table 44: Intersection 11 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	55 (96.5%)	0 (0.0%)	0 (0.0%)	2 (3.5%)
25%	31 (96.9%)	0 (0.0%)	1 (3.1%)	0 (0.0%)
50%	87 (95.6%)	1 (1.1%)	2 (2.2%)	1 (1.1%)
75%	85 (96.6%)	1 (1.1%)	0 (0.0%)	2 (2.3%)
100%	68 (93.2%)	0 (0.0%)	2 (2.7%)	3 (4.1%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 45: Intersection 11 Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	55 (96.5%)	0 (0.0%)	1 (1.8%)	1 (1.8%)	0 (0.0%)	0 (0.0%)
25%	31 (96.9%)	1 (3.1%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
50%	87 (95.6%)	3 (3.3%)	0 (0.0%)	1 (1.1%)	0 (0.0%)	0 (0.0%)
75%	85 (96.6%)	2 (2.3%)	0 (0.0%)	1 (1.1%)	0 (0.0%)	0 (0.0%)
100%	68 (93.2%)	1 (1.4%)	1 (1.4%)	3 (4.1%)	0 (0.0%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 46: Intersection 11 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	54 (94.7%)	3 (5.3%)	0 (0.0%)
25%	32 (100.0%)	0 (0.0%)	0 (0.0%)
50%	87 (95.6%)	3 (3.3%)	1 (1.1%)
75%	84 (95.5%)	4 (4.6%)	0 (0.0%)
100%	65 (89.0%)	8 (11.0%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

B.7.5 – Intersection 12 Results: Courtneypark Drive and Tomken Road

Table 47: Intersection 12 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	50 (51.6%)	18 (18.6%)	1 (1.0%)	28 (28.9%)
25%	92 (75.4%)	12 (9.8%)	3 (2.5%)	15 (12.3%)
50%	106 (80.3%)	19 (14.4%)	1 (0.8%)	6 (4.6%)
75%	129 (87.8%)	11 (7.5%)	4 (2.7%)	3 (2.0%)
100%	168 (93.3%)	7 (3.9%)	2 (1.1%)	3 (1.7%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 48: Intersection 12 Post-Encroachment Time Results

LCV Penetration Level	Post-Encroachment Time					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	53 (54.6%)	14 (14.4%)	9 (9.3%)	19 (19.6%)	2 (2.1%)	0 (0.0%)
25%	94 (77.1%)	14 (11.5%)	2 (1.6%)	12 (9.8%)	0 (0.0%)	0 (0.0%)
50%	109 (82.6%)	18 (13.6%)	2 (1.5%)	2 (1.5%)	1 (0.8%)	0 (0.0%)
75%	131 (89.1%)	8 (5.4%)	5 (3.4%)	2 (1.4%)	1 (0.7%)	0 (0.0%)
100%	169 (93.9%)	5 (2.8%)	3 (1.7%)	1 (0.6%)	2 (1.1%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 49: Intersection 12 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	35 (36.1%)	47 (48.5%)	15 (15.5%)
25%	41 (33.6%)	70 (57.4%)	11 (9.0%)
50%	38 (28.8%)	87 (65.9%)	7 (5.3%)
75%	60 (40.8%)	75 (51.0%)	12 (8.2%)
100%	63 (35.0%)	108 (60.0%)	9 (5.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

B.7.6 – Intersection 13 Results: Coutneypark Drive and Highway 410 Northbound Ramp

Table 50: Intersection 13 Time-to-Collision Results

LCV Penetration Level	Freight Conflicts			
	TTC = 0	0 < TTC ≤ 0.5	0.5 < TTC ≤ 1	1 < TTC ≤ 1.5
0%	2 (50.0%)	2 (50.0%)	0 (0.0%)	0 (0.0%)
25%	86 (82.7%)	14 (13.5%)	1 (1%)	3 (2.9%)
50%	125 (77.2%)	27 (16.7%)	1 (0.6%)	9 (5.6%)
75%	181 (77.0%)	38 (16.2%)	1 (0.4%)	15 (6.4%)
100%	200 (79.4%)	35 (13.9%)	2 (0.8%)	15 (6.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 51: Intersection 13 Post-Encroachment Time Results

LCV Penetration Level	Freight Conflicts					
	PET = 0	0 < PET ≤ 1	1 < PET ≤ 2	2 < PET ≤ 3	3 < PET ≤ 4	4 < PET ≤ 5
0%	2 (50.0%)	2 (50.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
25%	86 (82.7%)	17 (16.4%)	0 (0.0%)	1 (1.0%)	0 (0.0%)	0 (0.0%)
50%	125 (77.2%)	30 (18.5%)	5 (3.1%)	1 (0.6%)	1 (0.6%)	0 (0.0%)
75%	181 (77.0%)	41 (17.5%)	5 (2.1%)	7 (3.0%)	0 (0.0%)	1 (0.4%)
100%	200 (79.4%)	42 (16.7%)	4 (1.6%)	4 (1.6%)	2 (0.8%)	0 (0.0%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets

Table 52: Intersection 13 Conflict Type Results

LCV Penetration Level	Freight Conflicts		
	Crossing Conflicts	Rear End Conflicts	Lane Change Conflicts
0%	0 (0.0%)	4 (100.0%)	0 (0.0%)
25%	1 (1.0%)	79 (76.0%)	24 (23.1%)
50%	1 (0.6%)	122 (75.3%)	39 (24.1%)
75%	6 (2.6%)	172 (73.2%)	57 (24.3%)
100%	1 (0.4%)	191 (75.8%)	60 (23.8%)

*Each entry exhibits the number of freight conflicts recorded in the selected interval followed by the proportion of all conflicts within that LCV penetration level in brackets