

Pavement, Traffic, and Environmental Impacts of Autonomous Truck Platoons: A Quebec Case Study

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ABSTRACT

On Pavement, Traffic and Environmental Impacts of Autonomous Truck Platoons: A Quebec Case Study

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The commercial deployment of Autonomous Trucks (ATs) and the implementation of platooning technology present a transformative opportunity for the freight industry in Quebec, offering potential benefits in efficiency, safety, and environmental sustainability. However, the aggressive loading patterns of channelized truck platoons pose significant risks to pavement infrastructure, particularly regarding permanent deformation (rutting). This research investigates the optimal AT platoon configurations, specifically speed and headway, that minimize pavement distress without compromising traffic flow efficiency or environmental gains.

The methodology employs a multi-scale approach, combining laboratory rheological testing with microscopic traffic simulation. Uniaxial compressive haversine loading tests were conducted on asphalt concrete samples (ESG-10 mix with PG 64E-28 binder) to simulate traffic loading frequencies of 8 Hz and 10 Hz (corresponding to speeds of 80 km/h and 100 km/h) and rest periods of 0.5 s, 1.2 s, and 1.5 s. The experimental results identified that a higher frequency (100 km/h) coupled with a shorter rest period (0.5 s) resulted in the least accumulated permanent strain, contrary to traditional assumptions regarding material recovery time. These parameters were subsequently used to calibrate a PTV VISSIM microsimulation model of a 7.5 km segment of Autoroute 15 in Quebec. The simulation evaluated mixed-traffic scenarios with AT penetration rates ranging from 0% to 100% under varying traffic volumes. Results indicate that a three-truck platoon configuration with 0.5 s headway at 100 km/h significantly improves traffic operations, reducing network delay by up to 63% and CO₂ emissions by approximately 16% at full penetration. The study concludes that aggressive platoon parameters with high speeds and short headways are optimal for both rutting resistance and traffic efficiency in the Quebec context, provided that safety systems can reliably support such operations.

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LIST OF ABBREVIATIONS

Abbreviation	Definition	Abbreviation	Definition
AAA	American Automobile Association	LEF	Load Equivalency Factor
AADT	Average Annual Daily Traffic	LKA	Lane Keeping Assist
AASHTO	American Association of State Highway and Transportation Officials	LMRS	Lane Change Model with Relaxation and Synchronization
AEB	Automatic Emergency Brake	MEPDG	Mechanistic Empirical Pavement Design Guide
ACC	Adaptive Cruise Control	MOE	Measures of Effectiveness
ADAS	Advanced Driver Assistance System	MSCR	Multiple Stress Creep Recovery
ADS	Automated Driving Systems	MTD	Maximum Trailing Deceleration
ANN	Artificial Neural Network	MTMD	<i>Ministère des Transports et de la Mobilité Durable</i>
ATRI	American Transportation Research Institute	MTS	Material Testing System
AV	Autonomous Vehicle	MVTC	Motor Vehicle Test Center
AT	Autonomous Truck	NCDB	National Collision Database
CACC	Cooperative Adaptive Cruise Control	NCHRP	National Cooperative Highway Research Program
CAV	Connected and Autonomous Vehicle	NHTSA	National Highway Traffic Safety Administration
CT	Conventional Truck	PCE	Passenger Car Equivalent
CV	Conventional Vehicle	PCI	Pavement Condition Index
D	Directional Split	PHEV	Plug-in-Hybrid-Electric-Vehicles
DDHV	Directional Design Hourly Volume	PR	Penetration Rate
DDV	Design Hourly Volume	PRT	Perception Reaction Time
DMV	Department of Motor Vehicles	PSO	Particle Swarm Optimization
DOT	Department of Transportation	RPLD	Repeated Permanent Load Damage Test
DSR	Direct Shear Rheometer	SAE	Society of Automotive Engineers
ESAL	Equivalent Single Axle Load	SGC	Superpave Gyratory Compactor
EV	Electric Vehicle	SHRP	Strategic Highway Research Program
EAV	Electric Autonomous Vehicle	SSD	Stopping Sight Distance
FHWA	Federal Highway Administration	V2I	Vehicle-to-Infrastructure
GCI	Government Condition Index	V2N	Vehicle-to-Network
GDP	Gross Domestic Product	V2P	Vehicle-to-Pedestrian
GHG	Greenhouse Gas	V2V	Vehicle-to-Vehicle
HCM	Highway Capacity Manual	V2X	Vehicle-to-Everything
HDV	Human Driven Vehicle	VCED	Viscoelastic Continuum Damage Model
HGV	Heavy Goods Vehicle	VKT	Vehicle Kilometers Travelled
HMA	Hot Mix Asphalt	VFA	Voids Filled with Asphalt
HV	Heavy Vehicle	VMA	Voids in Mineral Aggregate
HWTT	Hamburg Wheel Tracking Test	VMT	Vehicle Miles Travelled
IDT	Indirect Tensile Test	VRU	Vulnerable Road Users
IGO	<i>Infrastructure Géomatique Ouverte</i>	WHO	World Health Organization
LC	<i>Laboratoire de Chaussée</i>	WTT	Wheel Tracking Test
LCA	Life Cycle Assessment	ZEHV	Zero Emission Heavy Vehicle
LCCA	Life Cycle Cost Analysis	ZETP	Zero Emission Trucking Program

1. INTRODUCTION

Self-driving cars have been a staple of science fiction for over a century; however, it is only in the past few decades that the science and technology have evolved enough to make these vehicles a reality. Over the past years, research and investment into the development of Autonomous Vehicles (AVs) as well as Connected Automated Vehicles (CAVs) has significantly increased. Leveraging advanced sensor suites, onboard computational algorithms, and actuator control systems, AVs execute dynamic driving tasks ranging from partial assistance to full automation. Concurrent with automation, the automotive sector is undergoing a paradigm shift toward connectivity, enabling CAVs to communicate with their environment via V2X protocols. V2X protocols are shown in Figure 1, and includes Vehicle-to-Vehicle (V2V), Vehicle-to-Pedestrian (V2P), Vehicle-to-Network (V2N), and Vehicle-to-Infrastructure (V2I), providing increased safety, mobility and efficiency. V2V in particular, provides the transmittance of information between vehicles, resulting in smoother and more efficient traffic operations as well as safety. V2I involves the communication of vehicles and their surrounding environment, including traffic signals, lane markings, and pavement.

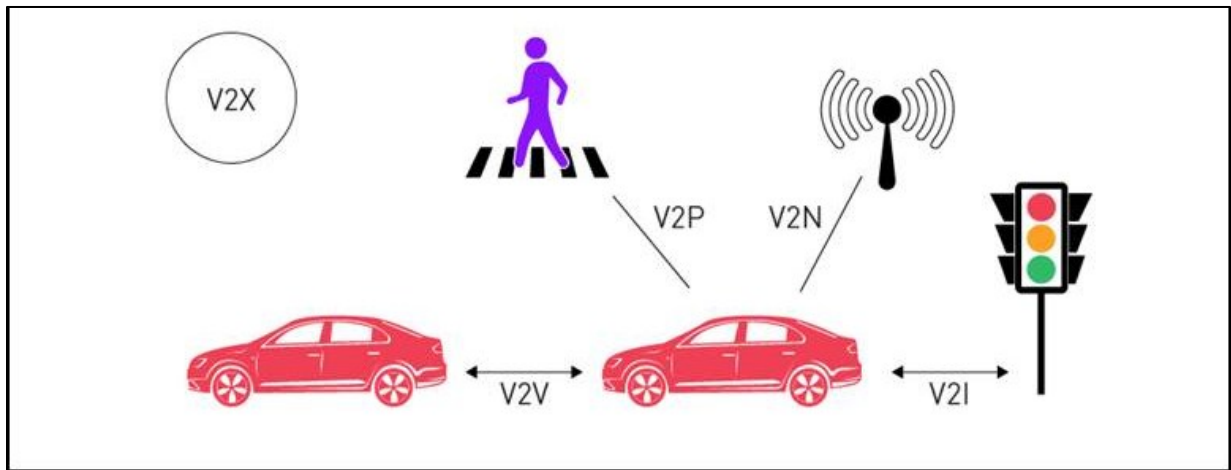


Figure 1: V2X Technology [1]

The Society of Automotive Engineers (SAE) international standard for levels of autonomy is widely used when describing vehicles with autonomous technology. The standard defines six levels of autonomy, with Level 0 having little to no autonomous features, and Level 5 being fully autonomous [2]. Currently, only vehicles with Level 2 technology are available, with Level 3 systems in the process of being tested. The permittance of AVs varies by province and by state. In Quebec, Level 3 and up vehicles are not permitted on public roads, unless part of a pilot project [3]. British Columbia has similar rules regarding vehicles Level 3 and above, where they are prohibited on public roads to protect Vulnerable Road Users (VRU), and will only be permitted once the technology is considered safe for public use [4]. On the other hand, the Ontario AVs Pilot Program allows for these vehicles to be tested on public roads, provided a driver is present and certain conditions are followed [5]. Until the technology for AVs is advanced enough to allow for large scale testing of high-level automation, the use of microsimulation and laboratory material testing is necessary to predict future infrastructure impacts in the absence of field data.

While Canada is taking a slower and more cautious approach to the deployment of autonomous technology, greater testing and development is already happening in the United States, China, Japan and Europe, with the USA and China as leaders of the industry. Southern American states, such as California, Nevada, Texas and Arizona, to name a few, have been testing AVs technology for a few years, with over 145 million miles driven on public American roads [6]. Robotaxis have been deployed in several states, along with testing for passenger and commercial vehicles. As of 2023, 41 states and the District of Columbia have laws or executive orders regarding AVs, however, there is no nation-wide federal regulation or framework regarding the testing, manufacturing or deployment of these vehicles [7]. Due to the large quantity of AVs testing in California, the California Department of Motor Vehicles (DMV) has passed regulations requiring crash and disengagement reports in the case of any accident involving an autonomous vehicle in autonomous mode or using autonomous features within 30 seconds of an incident [8] and a similar policy has been enacted by the National Highway Traffic Safety Administration (NHTSA) for AVs nationwide [9]. These reports will allow manufactures to better understand AVs interactions in mixed traffic scenarios and to continue to improve the vehicles from a safety point of view. Other than the USA, another major player in the AVs industry is China, who in 2022, had a penetration rate of almost 50% of vehicles equipped with Advanced Drivers Assistance System (ADAS) and testing and development for Level 3 and above vehicles, such as passenger vehicles, robotaxis, and freight [10].

AVs are predicted to bring many benefits to the transportation sector, such as safer roads, economic and environmental benefits, and improved congestion and traffic operations, as well as increased mobility and access. Globally, road traffic accidents are the cause of 1.19 million people losing their lives each year, and tens of millions more suffering injuries [11]. In Canada in 2022, almost 2,000 fatalities occurred because of road traffic accidents, with 85% of accidents due to human driving behaviour, such as speeding, distraction, or impairment. There is also an associated economic cost of fatal and injury collisions, costing \$40B in 2021 [12]. In the United States, over 40.9 thousand deaths occurred due to motor vehicle crashes in 2023, with 94% of these due to human error. One of the greatest potential benefits of AVs and CVs is the possibility to reduce the number and severity of accidents [13]. Through sensors, cameras, and connectivity the vehicles will be able to detect potential hazards or incidents ahead of time, and will be able to either warn the driver, or take action to prevent collisions. AVs will aid in moving the transportation sector towards Vision Zero, that is, zero deaths due to traffic accidents. In addition to increased road safety, highly and fully AVs are expected to improve mobility and accessibility as well. AVs would be able to drive those who are not able to drive themselves, such as older and younger people, and those with disabilities preventing them from driving, as well as those without access to a vehicle or in remote or rural areas [14, 15]. A report by the Autonomous Vehicle Industry noted that the National Federation of the blind consider AVs to be a 'gamechanger' for mobility of those impaired or blind, including increased opportunity for employment [16]. Greater mobility would result in greater access for those mentioned before for job opportunities, services, medical care, as well as retail and entertainment and therefore, greater productivity [15].

The increased access to transportation for non-drivers or those without access to a car, will result in increased Vehicle Kilometers Travelled (VKT), from increased travel by non-drivers, as well as increased empty-vehicle miles, which may occur when an AVs may be driving to pick someone up, or after dropping them off [17]. Additionally, increased access may result in greater sprawl, therefore increasing travel as well [15, 17, 18]. However, the negative effects of increased VKT may be mitigated due to autonomous technologies allowing for smarter routing, faster response time and smaller gaps, which may reduce congestion, increase capacity or stabilize traffic [19]. Also, the number of individual vehicles on the road may be decreased by nine to thirteen vehicles given just one AVs in ride-share mode, resulting in reduced congestion [19], and through

connectivity and autonomy, there would be less congestion due to carousing for parking [15]. The possible increase in VKT and congestion may result in greater emissions [19], however, smoother and more efficient travel would be able to mitigate this effect [18, 19]. Moreover, depending on how the energy is generated, Electric Autonomous Vehicle (EAVs) could result in decreased fuel emissions, specifically CO₂ and NO_x emissions [15]. Also, total vehicle emissions would be lowered given ride-share AVs scenarios, due to fewer individual vehicles and higher vehicle occupancy [15, 19, 20]. The potential safety, mobility, traffic and environmental benefits of AVs are also expected to incur economic savings. The US Intelligent Transportation Joint Office estimates \$434 billion USD in savings from 90% AVs penetration rate due to congestion improvements, as well as parking and accident reductions [19], while a Canadian Senate report cites collision and congestion reduction, fuel savings and increased productivity as the reasons for a potential savings of \$65 billion per year [21].

While there are many predicted benefits to AVs, the implementation of these vehicles will not be without challenges. The NHTSA has described cybersecurity and hacking as 'critical issues' regarding the deployment of autonomous driving technologies [13]. AVs will be able to collect and store personal data, including personal information, locations, and calls or recordings made during trips, and given V2V, V2N and V2I connectivity, this data would be highly susceptible to cybersecurity risks, requiring advanced protection from these threats [20]. In addition to the increased possibility of access to personal data, there is also the increased potential of hackers taking control of the vehicle, causing traffic disturbances or collisions, with the possibility of viruses impacting entire fleets of AVs [19]. Other considerations for the deployment of AVs includes the issues surrounding the ethics and liability of these vehicles. In the event of a potential accident, the vehicle may have to decide on which action to take, possibly weighing options in real-life trolley problem scenarios [17, 20], and when an accident does occur, how is responsibility and liability claimed [19]? These challenges, questions and concerns will need to be addressed prior to the mass roll-out of AVs on public roads to ensure safety and security of road users. Consistent and standardized regulation will be crucial to achieve these goals [19], however, regulation is also seen as one of the greatest 'bottlenecks' to AVs deployment [22]. The challenges and concerns are not only disadvantages with AVs, but are perceived disadvantages from the public as well, which can be a barrier to the implementation of any technology. A survey on the public opinion of Canadians regarding AVs showed that 42% of those surveyed considered legal liability a disadvantage of AVs, and 48% showed concern about cybersecurity [23]. Moreover, the American Automobile Association (AAA) found that 61% of those surveyed were 'afraid' of AVs [24]. Public pushback is seen as a major barrier to AVs [18, 25], and will need to be addressed in order for the technology to become mainstream and accepted.

Currently, Level 3 vehicles are being developed and tested, however, predictions for when AVs will become available vary. Level 4 vehicles exist, currently however, these vehicles are constrained to certain predefined routes and circumstances, such as shuttles around universities or airport terminals [20]. The timeline for highly and fully AVs has slowed down and is expected to take longer than originally thought for development and deployment [22, 26]. A report by the Victoria Transport Policy Report predicts that fully autonomous vehicles will be available by 2030 and become commercially available during that decade. Moreover, during the same period, self-driving buses and trucks are expected as well [17]. McKinsey Consulting also estimates that fully autonomous vehicles will become a reality in the next few years, before 2031 [22]. However, despite the technology possibly being available as soon as 2030, Transport Canada estimates that it may be 'years or decades' before these vehicles are used in everyday life by Canadians [27]. Other estimates include 10% of all light commercial vehicles being autonomous by 2030 [28], and 10% of new car sales will be Level 3, along with 20% of heavy-duty trucks by the same year [29]. By 2040, vehicles that are level 3 or higher may compose up to 60% of light vehicle

sales [29]. Figure 2 illustrates IDTechEx's predicted percentage of AVs penetration based on the SAE level capabilities of the vehicle [26]. The predictions for AVs deployment may be optimistically in the near future, however, recent incidents involving AVs may set back this roll out, as well as weaken public opinion. As of January 2025, Tesla has had over 1800 crash reports submitted to the NHTSA [30], with a recent investigation into the company following crashed and traffic violations [31], along with Waymo, who was also under investigation and had over 1200 vehicles recalled [32]. AVs from Tesla, Waymo and Cruise have been involved in accidents with reported fatalities [33, 34]. The accidents, investigations, and recalls faced by AV companies may result in the optimistic deployment predictions to be deployed.

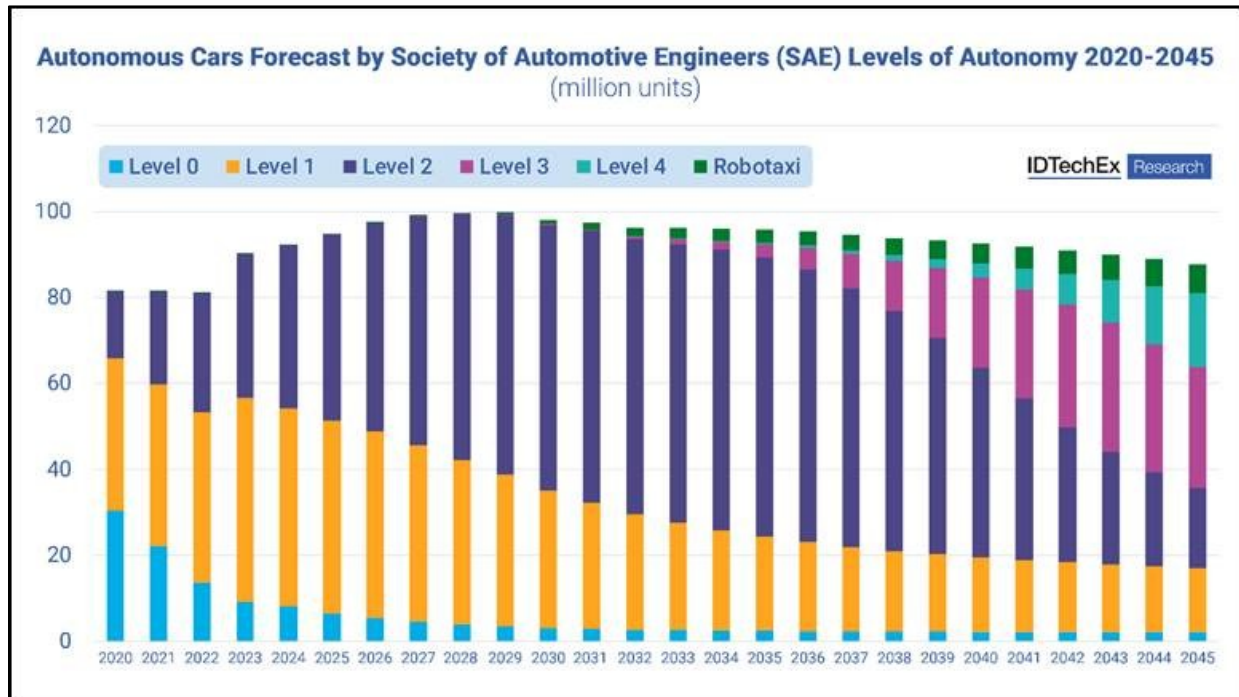


Figure 2: IDTechEx Autonomous Vehicle Market Forecast [26]

Robotaxis have been an application of autonomous driving technology that has gotten a lot of interest, with a predicted market value of \$174B USD in 2045 [26]. Despite 53% of Americans surveyed by the American Automobile Association (AAA) saying that they would not trust robotaxis [24], a JD Power survey found that confidence in AVs is increased in people that have traveled in robotaxis before [35]. Aside from robotaxis, long-haul freight transportation may be one the first commercial sectors to become automated [7, 10, 36]. In addition to robotaxis, automated heavy-duty freight is expected to have considerable future market size and public interest [10]. A United Kingdom survey of experts related to the field of autonomous driving technology showed that 77% of experts agree or strongly agree that companies that move goods and materials (also known as freight transport) will benefit the most from the introduction of self-driving cars [37]. Figure 3 shows the Boston Consulting Group predictions for AVs adoption rates. By 2030, the consulting group estimates that inter-city delivery and long-haul transportation of medium-duty trucks will be priority cases for AVs roll out, with 16% and 18% adoption rates respectively, in the United States [28]. A white paper report by The World Economic Forum and Boston Consulting Group predict that Autonomous Trucks (ATs) will comprise 30% of new truck sales by 2035 [38].

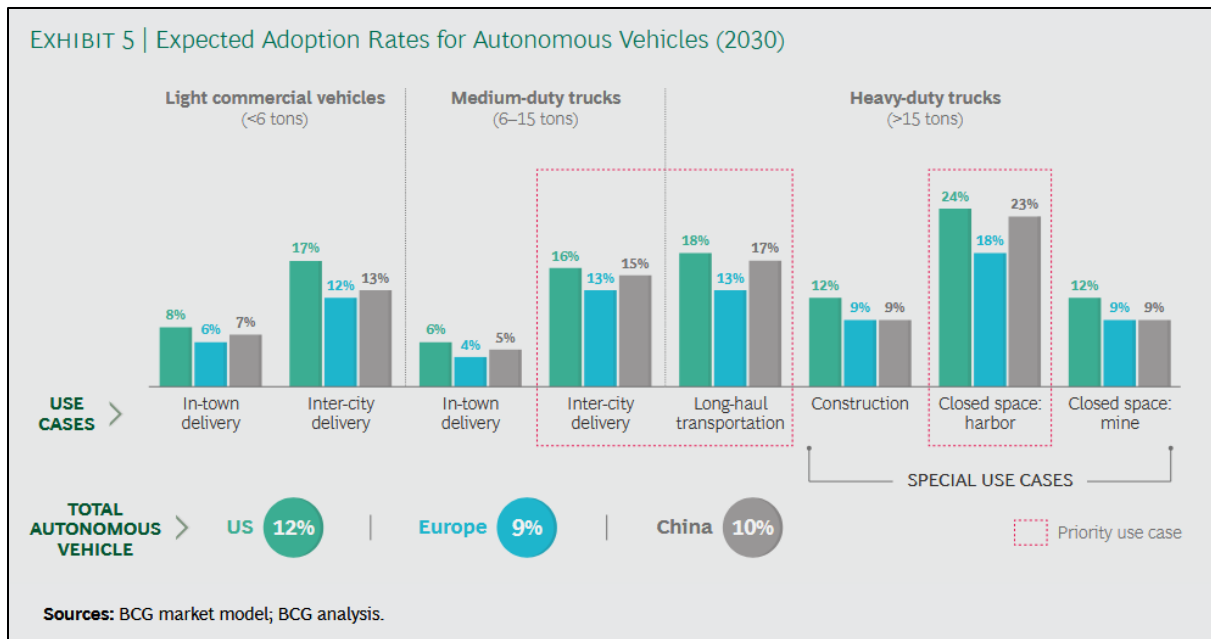


Figure 3: 2030 AVs Adoption Rates Prediction [28]

Autonomous trucks (ATs) may be implemented for hub-to-hub long-haul transportation, since the easy and repetitive driving conditions between hubs, such as distribution centers or ports, may be suitable for early autonomous technology [38]. Human drivers would carry out first-and-last-mile trips, from distribution centers to stores or costumers, where driving situations may be more complex [36, 39]. This process is illustrated in the infographic in Figure 4. Full autonomy, as well as connectivity would allow trucks to make the door-to-door trip since the vehicles would be able to perform the complex driving functions required for urban arterials and local streets. this is, however, only expected to become a gradual reality by 2040s, while trucks that possess enough autonomy for hub-to-hub trips may be available as soon as 2027 [40]. Through V2V connectivity, ATs will also be able to travel in platoons, where two or more vehicles travel together in a convoy at short headways from one another [7, 39, 41]. Early platoon structures may involve the lead vehicle to be human driven, with support from ADAS features, followed by higher or fully AVs at short, maintained distances and speeds, to reduce drag due to air resistance on following vehicles [7]. The European Environmental Agency predicts that this type of platooning will likely be available before fully ATs on highways [39]. A possible implication of ATs platooning as well as hub-to-hub transportation may be dedicated platooning lanes [39]. This may expedite the deployment of ATs, since there will be even fewer interactions with non-autonomous vehicles, as well as smooth traffic flow for Human Driven Vehicles (HDVs), however, dedicated platooning lanes may cause a capacity drop in unreserved lanes as a result [17].

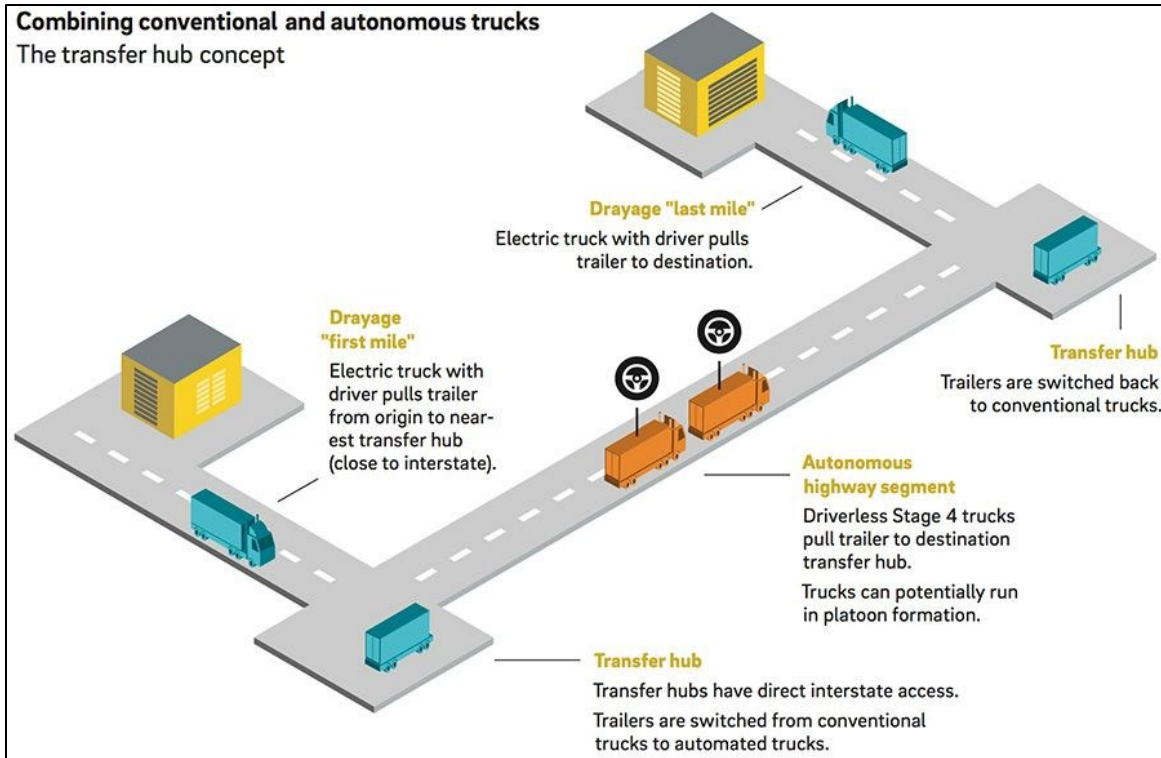


Figure 4: ATs Hub-to-Hub Process [42]

Despite research showing that up 84% of Canadians polled believed that automation would take over jobs done by humans [43], this will likely not be the case for truck drivers with the advancement of ATs technology, as least not in the near future [7, 17, 36]. Forbes estimates that 1.2 million new truck drivers will be needed in the next ten years to keep up with increasing freight demands [44]. In addition to growing freight demands, there is already a shortage of truck drivers globally; in the United States in 2018 alone, there was a truck driver shortage of 60 thousand drivers, expected to reach 160 thousand by 2028 [7]. ATs would help to address these labour shortages, and improve the quality of life of drivers [7, 36], as opposed to eliminating jobs. This scenario of autonomous driving (hub-to-hub) would allow truck drivers to forego the long trips required and have better driving schedules and more time at home [44]. Moving the freight industry away from drivers for long-haul trips will also be beneficial for the trucking industry. Human drivers require, and are legally mandated to, rest at least a certain amount every day, and are only able to drive on the road during daytime hours, but ATs would not be restricted by typical hours-of-service that are followed by human driven trucks [36, 45]. ATs that could drive at all hours, including during the night, would greatly increase the efficiency of freight transportation, as well as reduce the number of trucks present during peak hours, which will minimize the congestion effects for trucks and passenger vehicles [7, 38, 45].

ATs are also expected to reduce freight deadhead time, which are the trips made by trucks carrying nothing. In the EU deadhead miles make up 15% of miles traveled by trucks [7]. Deadhead miles can be reduced with ATs, since the vehicles will not be required to drive back to their point of origin and may go straight to pick up a different load, and they will also be able to wait for a new shipment to be available for pick up as opposed to making drivers wait [45]. This is not only more efficient for shipping and freight companies, but also results in greater environmental efficiency as well, since the travel is optimized to reduce empty driven miles, reducing unnecessary fuel consumption and emissions [46]. The increased travel of ATs outside

of hours-of-service and lack of travel during congested times may result in costumers and consumers receiving their deliveries or shipments faster, as well as possible reduced freight or delivery costs [7]. From a shipping or freight company perspective, more efficient freight transportation will reduce costs as well. Labour costs associated with long-haul travel may be reduced through automation, and fuel savings may be incurred as well with more efficient travel [7, 15].

In addition to increased efficiency on truck drivers, there are many other potential benefits of autonomous trucking, such as reduced fuel consumption and Greenhouse Gas (GHGs) emissions and traffic improvement. Possibly the greatest benefit to ATs will be improved safety conditions. ADAS and Automated Driving Systems (ADS) allow for trucks to detect any hazards or potential incidents before they occur, as well as to warn the driver, or in the case of higher automated vehicles, to take action to prevent or mitigate a potential crash [7, 44]. In addition to advanced sensor and decision-making technology, for highly or fully autonomous trucks there would be no risk, or a lessened risk, of collisions or accidents due to fatigue, distraction, impairment or emotion, as there would be with a human driver [36]. As previously mentioned, ATs would be able to travel at any time of day for their long-haul journeys, and therefore, it would be possible for trucks to avoid the times of day when most trucking collisions, reducing the potential for collisions. Data from Transport Canada, shows that 70% and 78% of fatal and injury collisions respectively, involving trucks happen between the hours of 6 am and 6 pm [47]. If trucks were limited during this time, and operated during off-peak or nighttime hours when fewer vehicles are on the roads, accidents involving trucks and Heavy Vehicles (HVs) could be greatly reduced. Despite the potential improvements in safety, there are some concerns with platooning, specifically, that a single failure of one vehicle in a platoon, or an incident involving the lead vehicle, could affect the other platoon vehicles and others not in the platoon [15].

HVs, such as trucks and buses, contribute to 35% of road transport CO₂ emissions, despite comprising only 8% of the vehicles on the roads [48]. ATs offer ways to mitigate the environmental issues brought about by HVs. Smoother driving maneuvers through consistent speeds, gentle acceleration, deceleration and braking, also known as eco-driving, would result in reduced inefficient energy consumption as well as reduced emissions [18, 19, 45]. ATs will also not have the same incentive as human truck drivers to drive at increased speeds, who are paid on a per-mile basis, and as such get more bang for their buck if they arrive at their destination quicker [46]. The consistent, lowered speeds as well as smoother driving maneuvers will allow for better fuel consumption and reduced emissions. Furthermore, ATs would have limited idle time, required by human drivers for rest, breaks and food, and therefore, there would be less fuel consumed and fewer emissions due to idle time [36]. Moreover, in platoon formation, reduced drag on the following vehicles results in less fuel consumed and less GHGs emissions produced [19, 41]. Transport Canada testing on truck platooning has found that fuel savings between 4.5% and 18% are possible depending on the platoon formation [41]. This is especially significant in areas with cold climates, such as Quebec, since a reduced temperature causes increased drag, with an 18% increase at -30 °C compared to 15°C [49]. An example of highway truck platooning is shown in Figure 5. Finally, ATs that operate outside of peak hours will not only be less affected by the stop-and-go conditions of congestion, which causes increased fuel consumption, but will also contribute to the congestion less themselves, reducing traffic congestion for other vehicles, and therefore resulting in reduced overall fuel consumption [45]. In a report by ATs manufacturer Aurora, ATs were found to have potential energy efficiency savings of 13% - 32% compared to human driven trucks as a result of the above factors and moreover, the immediate environmental benefits could happen in the near future with the launch of ATs [46].



Figure 5: Truck Platooning [50]

There are many potential benefits of ATs, however, the deployment of such vehicles does not come without challenges. Specifically for ATs, the United States Federal Highway Administration (FHWA) describes both technical and operational challenges, such as the implementation of cooperative adaptive cruise control, following distance thresholds, maintaining platoons, and acceptance of stakeholders and truck drivers [51]. Moreover, other challenges include operational complications in adverse weather, regulations and public acceptance [25]. Costs of technological manufacturing and testing and fleet management may be a hurdle in the beginning when the technology is still being rolled out, and consideration is needed for safety, liability and cybersecurity concerns, such as the transportation of hazardous material, susceptibility to hacking, and who is responsible in the case of an incident [7]. Research, investment, thorough testing and regulations will be incredibly important in ensuring that this technology is safe and able to provide the benefits mentioned above while overcoming the possible associated challenges.

This research aims to investigate the effects of ATs platoons, specifically the platoon configuration that is optimal for pavement distress, and how that platoon configuration will impact highway traffic through delay and average speeds, as well as environmental impact in terms of CO_2 , NO_x , PM_{10} and $\text{PM}_{2.5}$ emissions for the greater Montreal area in Quebec. The breakdown of the paper is as follows: First, background information on AVs, the trucking industry and asphalt pavement will be presented in Section 2. Relevant literature related to conventional trucking, truck platooning, recent developments on AVs and ATs effect on traffic, and pavement, and finally parameters related to pavement distress is presented in Section 3. The methodology for the pavement and AVs testing will be discussed in Section 4, followed by the presentation and discussion of results for both parts in Section 5. Finally, the summary, overall remarks, recommendations for future research, and limitations will be presented in the conclusion in Section 6.

2. BACKGROUND

2.1 Autonomous Vehicles

As defined by the *Gouvernement du Québec*, AVs are those road vehicles with systems such that they are able to operate at levels of autonomy of 3, 4 and 5 [3], as described by SAE [2]. The SAE International standard defines 6 levels of autonomy, from Level 0 to Level 5 with increasing autonomy up to full autonomy [2]. Up to and including Level 2 is described as ADAS technology, while Level 3 and up is ADS [2, 9]. A description of each SAE Level of Autonomy is given below, and a summary shown in Figure 6.

- Level 0: Considered as no automation or momentary assistance vehicles, human drivers are responsible and in control of all driving operations, however features may provide warnings to the driver of other vehicles' sudden movements. Driving assist features may include blind spot warning, lane departure warning, forward collision warning, as well as Automatic Emergency Braking (AEB) in the case of an imminent collision.
- Level 1: Human drivers are responsible for all driving operations, driving assist features aid with the execution of either lateral or longitudinal driving functions. For instance, a Level 1 vehicle may have Lane Keeping Assist (LKA) for lateral position control or Adaptive Cruise Control (ACC) to maintain space with other vehicles longitudinally, but not both features at the same time.
- Level 2: Considered as partial automation; the human driver is still in control of all driving tasks, however, driving assist features that aid steering and braking/acceleration, such as LKA and ACC may be used simultaneously to provide support to the driver.
- Level 3: Considered conditional automated driving, the vehicle can perform driving functions in certain conditions without a human, however, a human driver must be able to take over the driving operations when required.
- Level 4: Considered highly automated driving, the vehicle is able to perform all driving tasks without need for any human intervention given certain circumstances. Level 4 vehicles may be limited to operating only in specific areas, under specific weather conditions, or during set times.
- Level 5: Fully AVs, able to operate and function completely independently without human intervention in all scenarios, such as in any weather condition, and in any geographic area or environment, as well as areas that may be off-grid.

SAE J3016™ LEVELS OF DRIVING AUTOMATION™

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What does the human in the driver's seat have to do?

SAE LEVEL 0™	SAE LEVEL 1™	SAE LEVEL 2™	SAE LEVEL 3™	SAE LEVEL 4™	SAE LEVEL 5™
You <u>are</u> driving whenever these driver support features are engaged – even if your feet are off the pedals and you are not steering			You <u>are not</u> driving when these automated driving features are engaged – even if you are seated in “the driver's seat”		
You must constantly supervise these support features; you must steer, brake or accelerate as needed to maintain safety			When the feature requests, you must drive	These automated driving features will not require you to take over driving	

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What do these features do?

These are driver support features	These are automated driving features			
These features are limited to providing warnings and momentary assistance	These features provide steering OR brake/acceleration support to the driver	These features provide steering AND brake/acceleration support to the driver	These features can drive the vehicle under limited conditions and will not operate unless all required conditions are met	This feature can drive the vehicle under all conditions

Example Features

• automatic emergency braking • blind spot warning • lane departure warning	• lane centering OR • adaptive cruise control	• lane centering AND • adaptive cruise control at the same time	• traffic jam chauffeur	• local driverless taxi • pedals/steering wheel may or may not be installed	• same as level 4, but feature can drive everywhere in all conditions
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Figure 6: SAE Levels of Driving Automation [52]

AVs are able to carry out the driving functions described above through multiple sensors and computer systems. Sensors collect and process information [53] and AI and machine learning algorithms use that information for decision making and taking actions [15]. Multiple sensors are required for information gathering, including internal and external state sensors [54]. Internal state sensors, also known as proprioceptive sensors, provide information on the vehicles' internal data, such as force, angular rate, wheel load and battery voltage, as well as location sensors and inertial sensors. Sensors that gather information on the surrounding environment are external state, or exteroceptive, sensors, such as cameras, radar and LiDAR [53]. Cameras are able to capture high resolution images and videos for visual data of the vehicles' surroundings, such as traffic signs and lane markings, however, do not provide distance information of objects. Cameras aid in driving assist features such as lane departure warning, and allow for surround view. Radar involves radio waves emitted from the vehicle to obtain distance and speed information of objects, based on the rebounding waves. Medium-range radar supports features such as traffic, blind spot and rear-view monitoring, while long-range radar supports ACC. Similarly, Light Detection and Ranging (LiDAR), detects and tracks object distance using rebounding lasers, with a larger range than cameras. LiDAR allows for features such as AEB, object detection, and collision avoidance [53]. Finally, ultrasonic sensors receive information for large solid objects in close proximity, allowing for intelligent parking assist features. Figure 7 [54] illustrates the different technologies and their detection ranging, as well as the features they contribute to. Once information has been gathered by sensors, AI and machine learning algorithms process the data, predict future movements of other road users, and make decisions necessary for driving operations [55].

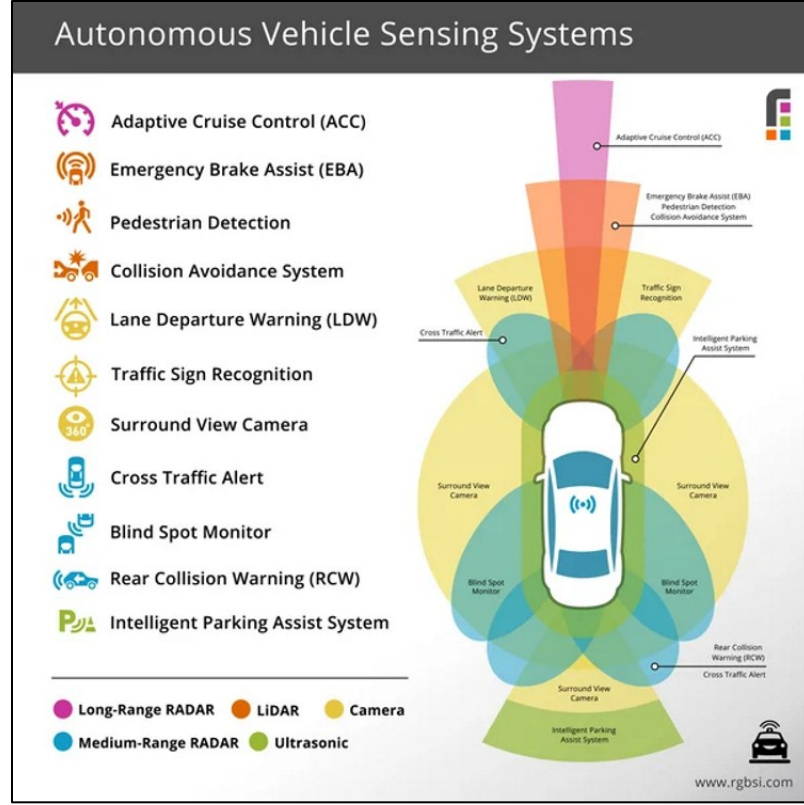


Figure 7: Autonomous Vehicle Sensing Systems [53]

The technology and features mentioned above will allow AVs to travel with reduced headways between vehicles, as well as at higher speeds. Shorter headways will be possible as a result of improved Perception Reaction Time (PRT), both from better detection of possible collisions and faster application of countermeasures. Through the sensor technology, as well as connectivity, AVs are aware of other vehicles, road users, objects, or animals' positions, speeds and possible accelerations, and are able to detect possible threats down the road or potential collisions [44]. Once a potential collision or risk is identified, the vehicle can react to those threats, either by warning the driver to take action in ADAS vehicles, or to take action through speed reduction, emergency braking, or object avoidance. The improved awareness and detection of the vehicles' surroundings, and faster time to react to potential collisions both result in shorter PRTs compared to HDVs. The American Association of State Highway and Transportation Officials (AASHTO) equation for Stopping Sight Distance (SSD), Equation 1 [56], shows that at the same speed and acceleration (denoted by V and a respectively), a shorter PRT (denoted by T_r), will result in a shorter required SSD, therefore reducing the spacing that should be maintained between vehicles. AASHTO specifies the use of 2.5 seconds as the reaction time for human drivers [56], while for AVs, a reaction of 0.5 seconds has been used in previous research [57]. The increased ability to detect and react to possible collisions will also allow AVs to travel at higher speeds compared to HDVs. Also, due to V2V, and V2I connectivity, vehicles will be able to travel more smoothly, with more standard speeds and gaps and fewer deviations, as well as more efficient acceleration and braking [19]. Shorter headways, due to reduced PRTs, will result in increased road capacities, and fewer disturbances in driving maneuvers or more standardized driving behaviour will result in greater traffic stability, with reduced likelihoods of capacity drops [58].

$$SSD = 0.278VT_r + 0.039\frac{V^2}{a} \quad \text{Equation 1}$$

In addition to maintaining certain speeds and headways, another feature of higher ADAS level vehicles and AVs is lane centering where the lateral position of the vehicle is maintained in the center of the lane through the help of cameras identifying the lane markings [59]. This feature, along with LKA, will allow for increased safety by keeping the vehicle in its line, and may result in a smoother ride for both passengers and cargo [60]. The ability of an AVs to maintain consistent positioning along a wheel path would allow for lanes and therefore roads to be designed narrower [17, 61], however, this type of channelized driving may have detrimental effects on the pavement infrastructure, since greater loading repetitions would occur over the same area of pavement, and therefore, damage to the pavement is increased. To reduce this impact, AVs can be programmed to maintain a lateral offset from one another, or to have distributed wheel wander, similar to human driven vehicles. Communication, connectivity and ADAS features, along with speed, headway and lateral positioning also allow for AVs platooning. Figure 8 illustrates the features of AVs platooning, demonstrating the headway, lateral offset from one vehicle to another, and the interplatoon distance from one platoon to the next. Road design will have to take into account the features of automated driving; in order to mitigate any negative effects that AVs could have on infrastructure.

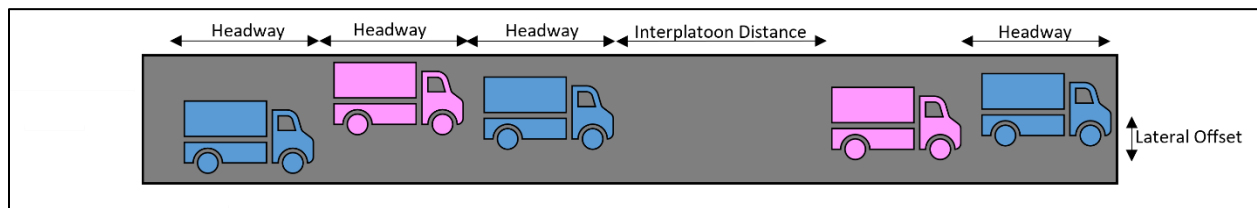


Figure 8: Vehicle Platooning

2.2 Trucking

As this research will focus on ATs, a brief introduction to the effect of Conventional Trucks (CTs) and the importance of the trucking industry is presented. The trucking industry in North America, specifically in Canada and the United States, is the main mode of freight transportation on the continent. In the United States in 2023, 72.6% of freight was moved via trucks, producing \$987 billion USD yearly, and moving over 11 billion tons of goods [62]. In Canada in 2022, trucking contributed to almost \$40 billion CAD dollars in revenue from over 65 million [63] shipments annually, and plays a greater part in the national Gross Domestic Product (GDP) than air, rail and marine freight transportation combined [64] and the trucking industry is estimated to transport over 90% of freight in the country [65]. Over 324 000 people are employed in Canada as truck drivers [63]. Out of the 26.2 million vehicles that were registered in Canada, 4.7% of those vehicles were classified as HVs defined as trucks that weigh more than 4535 kg [66]. Vehicle registration information for Quebec was obtained from the Quebec statistics data bank [67]. HVs in Quebec are defined as vehicles whose gross weight is greater than 4500 kg, and buses or minibuses [68], and as such, the total number of trucks, road tractors, and buses from the data set were used to determine the HVs. Table 1 shows the HVs, and total vehicles registered for the years of 2000 and 2022, as well as the proportion of HVs and the percent growth between 2000 and 2022. The values are extrapolated from the vehicle registration information in Table 27 in Appendix A. Based on a report carried out for the Cement Association of Canada, an annual 2% compound growth rate is estimated for traffic volume [69]. Additionally, the trucking industry in Canada is perceived as the most important mode of freight transportation by over 51% of Canadians, in a survey carried out by the Canadian Trucking Alliance [70]. All the statistics, as well as the perception of Canadians, illustrate the significance of the trucking industry.

Table 1: Summary of Quebec Total Vehicle and HVs Registration in 2000 and 2022

	2000	2022	22-Year Growth (%)	Average Yearly Increase (%)
#Trucks, Road Tractors	112 180	180 261	60.69	2.76
#Buses, School Buses	16 515	20 651	25.04	1.14
Total Vehicles	4 660 947	7 058 605	51.44	2.34
% HVs	2.76	2.85		

The trucking industry is a large part of the economy, and necessary for the transportation of goods across the continent, and as such, the impact of trucks on society is quite significant. The effect of CTs has been well documented, in terms of the effect on safety, traffic, environment and on asphalt. As previously mentioned, these vehicles play a very important part in the transportation of goods, and are crucial to the economy, however, the number of collisions and fatalities that are associated is a serious issue. Using data from the National Collision Database (NCDB), Transport Canada reports that from 2012 to 2021, commercial vehicles, specifically trucks, tractors trailers, and buses, were responsible for 8% of all collisions, however, this amount is increased to 20% for fatal collisions [47]. Up to 18% of fatal and injury collisions involving commercial vehicles occurred during weekdays, with up to 70% and 78% of fatal and injury collisions respectively, occurring between the hours of 6am to 6pm [47]. Driver fatigue is stated as the reason for collision for only 1.5% of both fatal and injury collisions, however, the report states that these values are likely underestimated since it is difficult to confirm that this is the case [47]. In Quebec specifically, 20% of fatal collisions involved a HVs in 2023 [71].

In terms of the impact of CTs on the environment, Transport Canada estimates that medium and heavy-duty trucks are responsible for 6% of the country's total GHG emissions, and 27% of transportation related emission [65]. The larger, and heavier a vehicle is, the greater amount of fuel is consumed, and more GHGs are emitted [72]. In Quebec specifically, the transportation sector is responsible for the majority of GHG emissions, at 39% [73]. Almost \$76 million has been allocated by the Zero Emission Trucking Program (ZETP) over five years to promote the electrification of medium and heavy-duty trucks [65], however there is a long way to go. Based on registration of vehicles by fuel type in Canada, battery electric, hybrid, or plug-in hybrid trucks represent only 0.0065 % of trucks or buses in 2023 [74]¹. A report from the University of British Columbia shows that emissions from HVs have been increasing over the past two decades, despite other transportation modes having reduced emissions, due to improvements in vehicle technology, as well as because of the introduction of electric and hybrid alternatives [75]. Figure 9 illustrates the changes in GHG emissions in the Canadian transport sector from 1990 to 2020 from the UBC Sustainability report. Similar findings are shown in Figure 10, illustrating the emissions due to the road transport sector in Quebec from 1990 to 2018. A report by the Pembina Institute, Jalon, and Equiterre, concluded that the emissions due to HVs traffic has increased by 190% in this period, and that HVs registration has increased by 16% in this period as well [76]. The impact of HVs on the environment is quite significant, and while the intent to electrify these vehicles promises the reduction of emissions, there is still a way to go to achieve this goal.

¹ Statistics Canada Vehicle Registration Data for 2023 can be found in the Appendix in Table 28 , along with specific HVs data in Table 29.

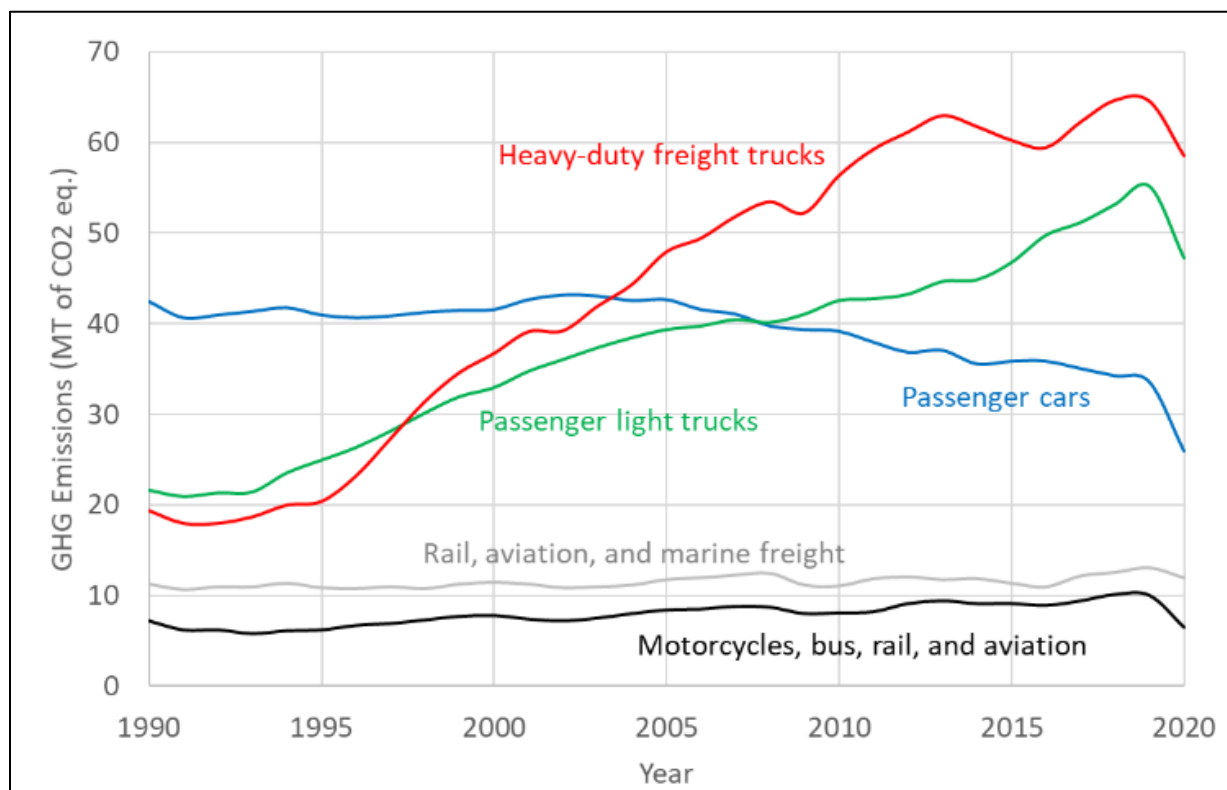


Figure 9: Canadian Transport Sector GHG Emissions 1990-2020 [75]

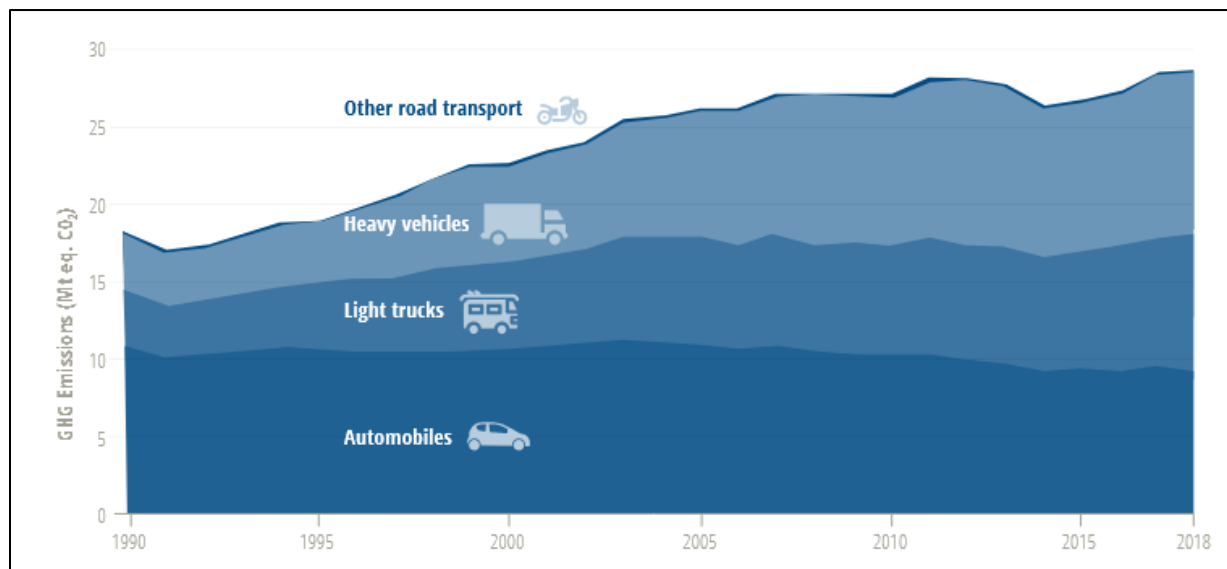


Figure 10: Quebec Road Transport Sector GHG Emissions 1990-2018 [76]

Due to their large size and weight, trucks and HVs maneuverability is decreased compared to smaller vehicles, more space is occupied, more time is needed for acceleration and deceleration, and a greater stopping distance is required for these vehicles. All of these factors result in the greater contribution of trucks and HVs to congestion [77]. A report by the American Transportation Research Institute (ATRI) showed that the cost of congestion for the trucking industry to be \$94.6B USD in 2021. Moreover, due to traffic and congestion, stop-and-go maneuvers were increased, as well as idle time, resulting in 6.7B gallons of fuel wasted during this period [78], representing a large economic and environmental loss. The amount of emissions by HVs in Quebec was found to increase more than four-fold when the speed of the vehicle is decreased from 105 km/h to 5 km/h [79]², further demonstrating the environmental cost of HVs in congestion. In addition to the traffic and congestion problems exacerbated by HVs, these vehicles also produce the greatest noise level and vibrations compared to other vehicles. A study by St-Jacques and Sicilia found that trucks travelling in a straight line produces a greater noise level than a car horn and is the greatest noise level due to a vehicle as well as that HVs vibrations, especially on poor roads, as increased [77].

Another important consideration when discussing CTs is their effect on infrastructure, specifically on pavement systems. Increasing vehicle weights cause exponentially more damage to pavement. The FHWA concluded that increasing truck weight would cause pavement to deteriorate faster and increase pavement costs [80]. As a result of this, many governing agencies place load limits and regulations on HVs to mitigate the damage done. In Quebec, load limits are set to 'ensure the safety of road users and protect road infrastructures' [81]. Moreover, load limits are further restricted during freeze-thaw periods in the spring, since the pavement structure can be up to 40% more susceptible to damage during this period [82]. Quebec has a road network of over 325 000 km, about 31 000 and 107 000 of which are regulated by the *Ministère des Transports et de la Mobilité Durable (MTMD)* and by municipalities respectively [83]. Only 56% of Quebec roads are described as being in a good condition, based on the Government Condition Indicator (GCI), meaning that little maintenance work is required over the next five years. The GCI is used to assess and describe the quality of various infrastructure components based on level of deterioration and defects, as well as the amount of investment required to maintain the roads in proper condition. The remaining portion of Quebec roads that are not in good condition exhibit high levels of deterioration and defects and require maintenance or rehabilitation [84]. Heavy truck traffic is one of the main reasons for the poor quality of pavement throughout the province, as well as climatic conditions, such as heavy precipitation, frost, and freeze-thaw cycles, all of which make road design and maintenance in Quebec, particularly in the Greater Montreal area challenging. Figure 11 illustrates the trucking route map of Quebec, with a focus on the Greater Montreal Area, where trucking routes are concentrated. It can be seen how the economic development of Quebec impacts the distribution of the trucking volumes, as most of it occurs in the southern part of the province, with a large portion of the trucking routes through or near the large cities of Montreal and Laval. Given the unbalanced distribution, and the fact that most routes are centered in one area, this gives an idea as to how many trucks are passing through just one part of the network, which is a significant factor in the deterioration of those roads.

² GHG Emission Data for Montreal HVs at various speeds can be found in Figure 52 in Appendix A.

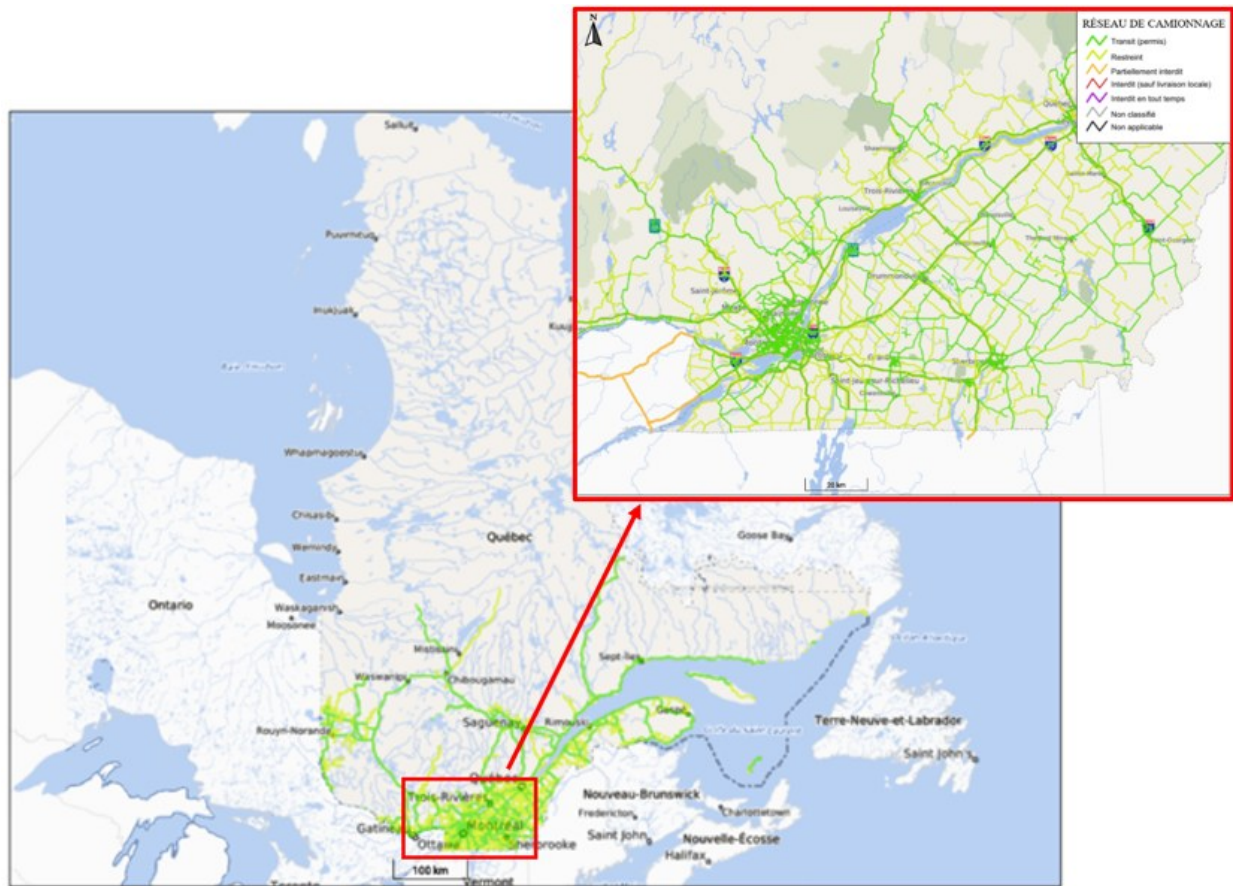


Figure 11: Quebec Trucking Route Map [85]

2.3 Pavement

Along with traffic operations and environmental factors, trucks have a large impact on pavement systems as well, thus the impact of ATs on pavement is important for consideration. Background information on pavement systems is provided below.

There are two main pavement types used as road surfaces: flexible and rigid pavements. Rigid pavements are composed of concrete that may be reinforced for additional strength with steel rebar over an aggregate base and finally the subgrade. An example of a concrete pavement system is shown in Figure 12. The stiffness of the concrete, as well as the distribution of the force over a large area, provides suitable resistance to static loads [86], and as such, concrete pavements are mainly used for pavement where vehicles are stationary such as parking lots or bus depots.

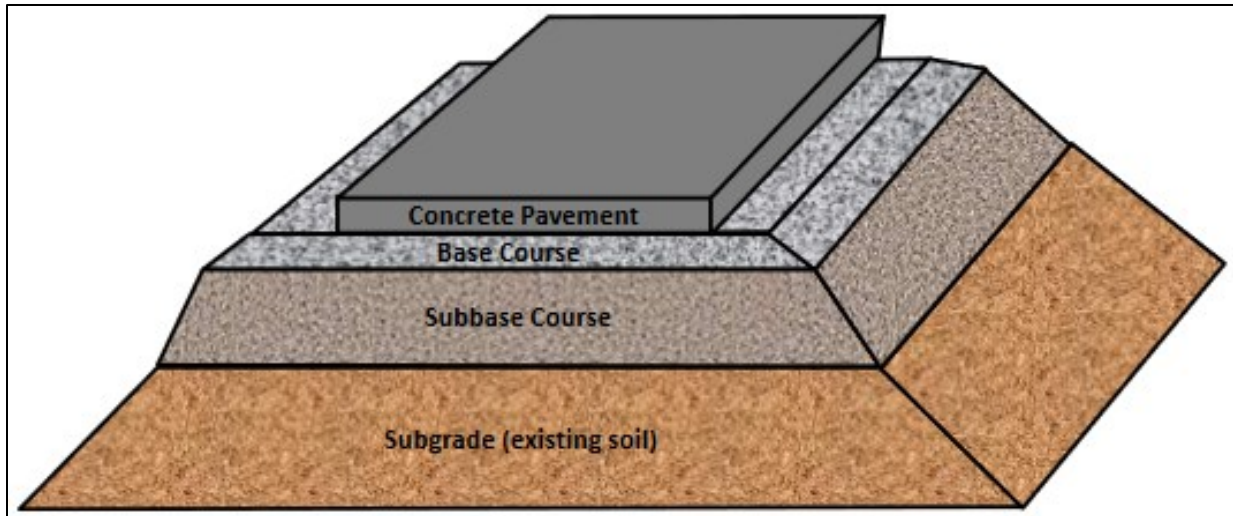


Figure 12: Concrete Pavement Layer System [87]

Conversely, flexible pavements are bituminous, or asphalt pavements and are used where dynamic loading is present, such as on roads under vehicle loading. The term bituminous refers to the binding agent of the asphalt being bitumen, which when combined with aggregate produces asphalt. When these materials are at, and combined at, higher temperatures, the resulting asphalt is Hot Mix Asphalt (HMA), which will be mainly referred to here. For the purposes of this work, the term HMA, asphalt, or asphalt pavement will be used to synonymously refer to flexible pavement or asphalt concrete. Figure 13 shows a typical asphalt pavement system, composed of asphalt surface and binder courses, followed by aggregate base and subbase layers, all on top of the soil subgrade. While the binder course allows for load distribution, and the base and subgrade layers provide for support and stability, the surface layer provides a smooth top layer, noise reduction and most importantly rutting resistance, and as such, this type of asphalt mix will be examined in this research, as opposed to the entire layered system [88]. Traffic loading applied over the pavement is distributed over a small area, but over many layers, such that the stress is lessened by the time it reaches the subgrade layer [86].

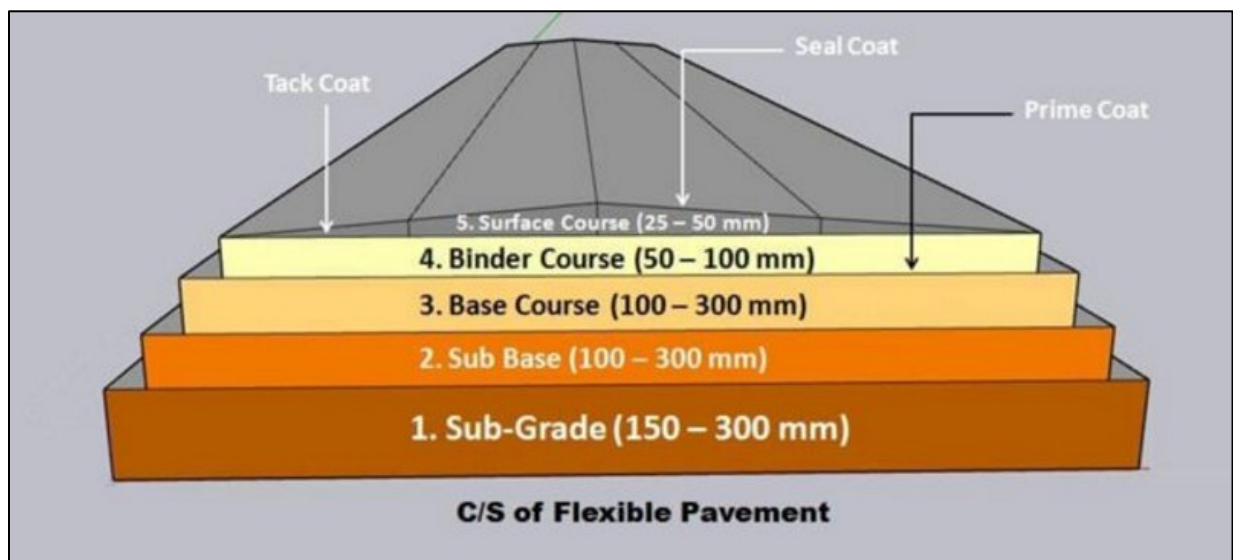


Figure 13: Flexible Pavement Layered System [89]

The reason that asphalt pavement is used for dynamic traffic loading, is due to the viscoelastic properties of the binder. A viscoelastic material, also referred to as a non-Newtonian fluid, is one whose behaviour is neither characterized as elastic, such as solids, nor viscous, such as liquids, but rather falls somewhere between the two. The viscoelastic properties of binder are described using asphalt rheology, that is, the deformation and flow of the material characterized by various temperatures and loading times. Understanding the rheological and viscoelastic properties is important in the prediction of the behaviour of the material. The two main parameters that impact the performance of the binder are complex shear modulus (G^*) and the phase angle (δ). The complex shear modulus represents the total resistance to deformation under shear loading, with a greater value of G^* indicating greater stiffness and a therefore greater resistance to deformation [90]. The phase angle indicates the time lag between the stress and strain, where no time lag is characteristic of an elastic material, while the greatest time lag is observed for a viscous liquid. For a viscoelastic material, a lower phase angle indicates a material that behaves closer to an elastic solid with greater resistance to flow and deformation, while a greater value means that the material is more similar to a viscous liquid with a greater potential for flow and therefore, increased potential for deformation [90]. Additionally, a greater rutting factor of $G^*/\sin(\delta)$ results in greater resistance to deformation [91].

Elastic behaviour refers to a solids' ability to return to its original shape or strength immediately once a load has been removed. This response is independent to the time of loading, the loading rate, or the temperature, and will occur regardless of long load times, slow loading rate, or any temperature. On the contrary, a viscous liquid will flow, or deform, when it is subjected to loading, and this is not a recoverable process. This behaviour is very dependent on time and temperature, specifically that larger loading times or rates of loading, as well as higher temperature that will induce more flow in the fluid [92]. As a viscoelastic material, whether the asphalt behaves more as an elastic solid or a viscous liquid is also highly dependent on temperature and time. At shorter loading times, as well as at lower temperatures, the asphalt behaviour is more like that of an elastic solid, and the material will recover part of its strength and shape when the load is removed. However, at longer loading times and higher temperatures, the asphalt is more likely to behave as a viscous fluid, that flows and shifts under loading, causing increased deformation and less immediate recovery [93]. Even under short loading times and low temperature where strain recovery is possible, asphalt pavement will still undergo deformation, known as creep, which the deformation of a material over time [94].

The viscoelastic properties of asphalt binder are best represented by the Burger's Model and equation [95]. Elastic materials are modeled as springs that return to their original shape once a loading has been removed, given that the loading does not exceed the elastic limit. Viscous materials on the other hand, are modeled using dashpots, where the material will deform at a constant rate while the loading is applied. The total deformation is dependent on the temperature as well as on the loading time, and once the load is removed, the deformation is non-recoverable. The Burger's model uses a combination of springs and dashpots connected in parallel and in series to represent the behaviour of viscoelastic materials under loading [95], an example of which is shown in Figure 14. G_1 represents the initial elastic behaviour, G_2 and η_1 represent the viscoelastic part of the response, followed by the viscous behaviour of η_2 . Under greater loading duration or higher temperatures, the material response is governed by η resulting in viscous behaviour, while for the reverse, G controls the response, and the behaviour is more elastic, with the ability of strain recovery [95]. The strain rate ($\dot{\gamma}$) of Burger's model is shown in Equation 2, where the applied loading is denoted by σ , the loading time denoted by t and the retardation time, λ or η/G [95].

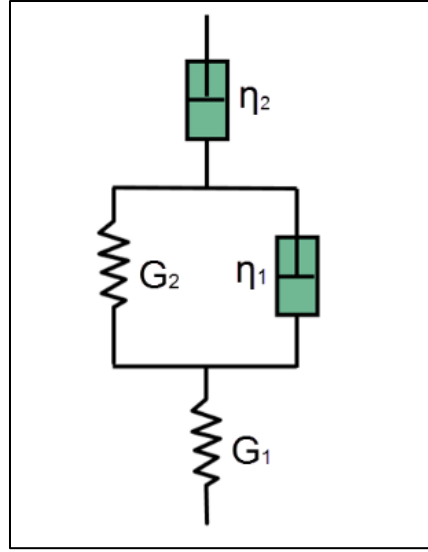


Figure 14: Burger's Model for Viscoelastic Behaviour [95]

$$\gamma = \sigma \left(\frac{1}{G_1} + \frac{1}{G_2} \left(1 - e^{-t/\lambda} \right) + \frac{1}{\eta_2} \right) \quad \text{Equation 2}$$

The viscoelastic properties of asphalt allow the material to be suitable for dynamic loading, however, the material properties are taken into account with certain factors that also influence the behaviour of the asphalt pavement. The loading time and temperature were mentioned above, along with the materials, mix design, and magnitude of loading and climate, as well as others, all have an impact on the pavement performance [96]. The loading magnitude, or weight of loading, is one of the most significant factors that influence pavement distress, with greater loads causing exponentially increased damage to the asphalt than loads of lower magnitudes [97]. Vehicle impact on pavement is characterized through the conversion of the vehicle weight to Equivalent Single Axle Load (ESALs), which is the amount of damage a vehicle does relative to that done by a standard 80 kN single axle semi-truck. Calculation of ESALs for any given vehicle are based on the 1993 AASHTO equivalency equation using pavement structure parameters, serviceability life and subgrade modulus of resilience (M_r) [98]. The load equivalency factor (LEF) can be estimated using the Fourth Power Law, using the ratio of any given vehicle weight to that of the standard raised to the fourth power, and is shown in Equation 3 [99].

$$LEF = \left(\frac{\text{Vehicle Load}}{80 \text{ kN}} \right)^4 \quad \text{Equation 3}$$

The temperature effect was mentioned briefly above, with greater asphalt distress under warmer temperature due to the viscoelastic nature of the material. Additionally, the climate of the region will also have an impact on the pavement performance, and according to some research, will most significantly impact the asphalt [100]. Warmer temperatures, as well as more humid or wet climates, tend to result in greater cracking and deformation, while colder climates and those where freeze-thaw cycles occur have accelerated cracking [101], and as such special design considerations should be given based on climate and temperature. Part of the design process is the consideration of materials. Aggregate with desirable qualities is necessary for ensuring proper pavement performance and asphalt binder should be chosen such that it is suitable for the climate and expected traffic loading [102].

There are many pavement distresses that dictate the performance and the life of the asphalt. The two main distresses that affect asphalt are fatigue cracking and permanent deformation. Fatigue cracking occurs due to tensile strain that accumulates in the top of the asphalt layers of the pavement system, due to repeated traffic loading, resulting in cracks that propagate throughout the layers and the surface of asphalt. Figure 15 shows an example of severe cracking, including fatigue cracking, along the surface of a rural road, where the cracks have propagated so much, that alligator cracking is formed. Permanent deformation, also known as rutting, is the depression or wheel path rut that is formed due to the continuous deflections of the pavement under repetitive loading, causing accumulated compressive permanent strain in the lower parts of the asphalt layers [102]. An example of an urban road with severe rutting is shown in Figure 16; the wheel path deformation can be seen in the image, as well the water pooled in the depression. Both types of distresses indicate system failure. Severe fatigue cracking reflects structural breakdown, while deep rutting can reduce driving quality by lowering comfort and maneuverability, increasing water accumulation and hydroplaning risk, and making lateral movements more difficult [103]. In Quebec, the serviceability limits are 10% surface coverage of cracks for fatigue cracking and 10 mm depth of rutting [69].



Figure 15: Cracking Example [104]



Figure 16: Rutting Example [105]

As previously mentioned, asphalt is modeled most accurately as a viscoelastic material, which recovers part of its strength when loading is removed. Figure 17 illustrates the strain response under stress-controlled conditions, i.e., when constant stress is applied and the resulting strain is measured. Upon loading, there is an exponential increase in strain, however, once the loading is removed, part of the deformation is recovered. The portion of the deformation that is not recovered is known as unrecovered, or permanent, strain. The accumulation of permanent strain can be seen in Figure 18, which depicts the stress-controlled, deformation-time curves for stresses of 100 kPa and 200 kPa over many cycles, with continuous loading and unloading, without rest. It can also be seen in Figure 17, that the two deformation-time curves follow a similar pattern, with a sharp increase in permanent strain, followed by a more gradual and linear increase. This follows the trend shown in Figure 19, where permanent strain accumulation follows a logarithmic pattern such that there is a sharp increase in permanent strain accumulation during the primary phase, followed by gradual strain accumulation following more closely a linear increase during the secondary phase, and finally undergoing another sharp and exponentially increasing strain accumulation during the tertiary phase. The tertiary phase is also recommended as an indicator of permanent deformation of the pavement [106].

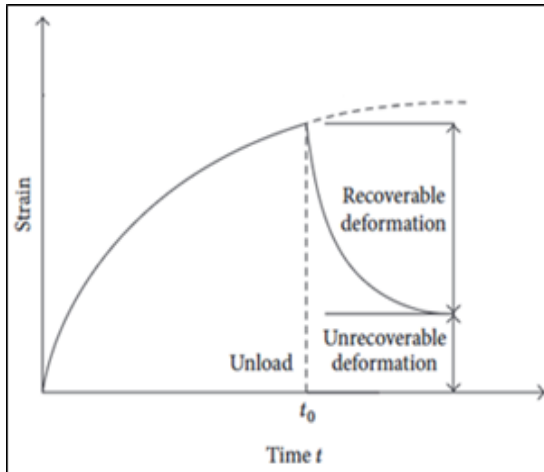


Figure 17: Stress-Controlled Strain-Time Curve [107]

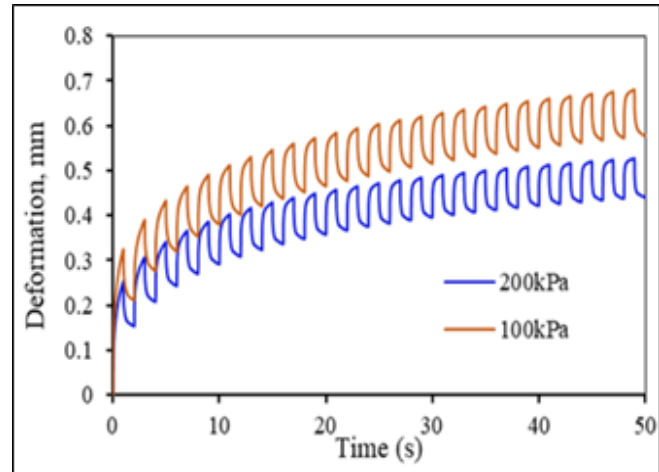


Figure 18: Stress-Controlled Deformation-Time Curves [108]

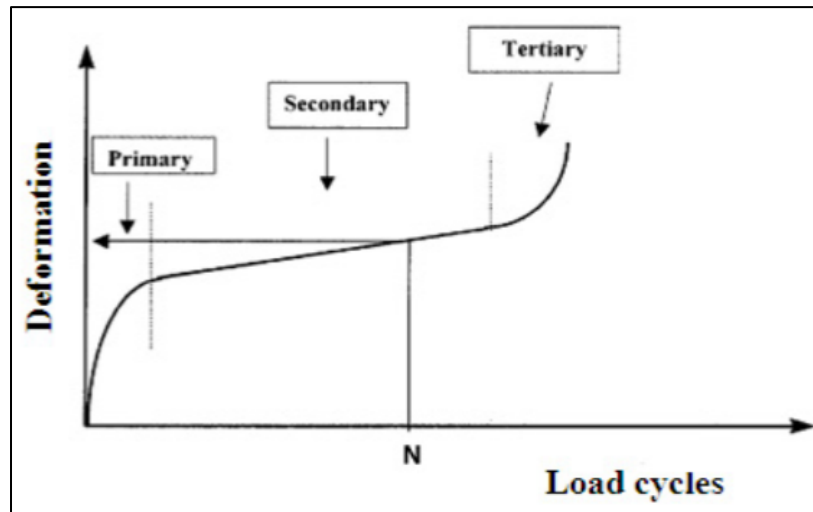


Figure 19: Typical Asphalt Deformation-Cycle Curve [109]

The permanent strain or deformation of asphalt can be tested or estimated in a number of ways, such as through simulative tests, dynamic-based tests, repeated loading testing, shear and creep testing, as well as through finite element analysis or computer-based software analysis, to name a few [110]. Simulative tests involve a wheel passing back and forth over a slab of asphalt for a predetermined number of passes to observe the deformation, simulating tire passing over pavement. Such tests include the Wheel Tracking Test (WTT) and the Hamburg Wheel Tracking Test (HWTT). Dynamic-based testing involves the application of a compressive sinusoidal loading to dynamic properties such as the dynamic modulus, flow number and flow time tests. Similarly, repeated loading tests, such as the Repeated Load Permanent Deformation (RLPD) test use repeated haversine compressive loading, and a major benefit of these types of tests is that the temperature, loading frequencies and cycles can all be programed for specific testing [110]. Haversine loading is preferred during these tests, since this waveform type most accurately represents truck traffic on asphalt pavement [111]. Creep testing, including the Multiple Stress Creep Recovery (MSCR), is used for rheological property testing of asphalt as it related to permanent strain. Since the laboratory tests for pavement testing require significant time, finite element analysis, as well as computer software analysis, such as AASHTOWARE or VISCOROUTE, and genetic programming or Artificial Neural Networks (ANN) are used to predict pavement damage based on cumulative damage equations [110].

3. LITERATURE REVIEW

With respect to infrastructure, this research will consider AVs, specifically ATs and ATs platoons, and their impact on asphalt pavement and the corresponding impact of optimized platoon configurations on traffic. First, a comprehensive literature review of trucking and the autonomous vehicle platooning effect is presented followed by a pavement review. The literature review is a summary of the recent and relevant research performed on the above topics. The review on trucking will include the effect of CTs on traffic, safety and the environment; the AVs and ATs review will look at the effects of these vehicles on traffic, safety, society, and asphalt pavement, dictated by the wander, speed, spacing and costs. This is followed by the review on platooning, which will describe the benefits and costs of truck platooning on traffic and environment. Finally, a small review on how varying vehicle parameters such as speed and headway impact asphalt pavement in terms of rutting and fatigue cracking will be presented in this section.

3.1 Trucking Review

As previously mentioned, the trucking industry is very crucial for the economy through the transport of goods all over the continent, with statistics in the previous sections illustrating the importance of this mode of freight transport in terms of percentage modal share, contribution to GDP and goods distributed. However, as this is a large part of freight, the impact of trucks and HVs should not be understated. The importance of the trucking industry in terms of the economy has been mentioned previously; therefore, the following review will discuss research done on the impact of CTs on traffic, safety and environment.

3.1.1 Truck Effect on Traffic

To begin, den Eelco den, et al. [112] analyzed Heavy Goods Vehicle (HGVs) data in the EU-27 with regards to emissions, air pollutants, noise, accidents and congestion. The report showed that road freight transport had been increasing, and was predicted to further increase, up to 60% by 2030, with a modal share of about 75%, expected to remain stable until 2030 as well. One of the components of the study was to determine the utilization factor of HGVs in the EU-27. Based on the available data from the years prior to 2009, it was found that these vehicles only had a utilization factor of 60% for the full maximum theoretical loading for both way trips. The reasons for this given by the authors were that many HGVs are only full for one-way trip, as opposed to the return trip, and many vehicles are not filled to their full weight capacity or limitation. This accounts for many trips made with empty or below-capacity trucks. Moreover, the external costs due to congestion for HGVs was determined as well, using the total cost from congestion to the studied countries with Passenger Car Equivalent (PCE) for HGVs, and was found to be around \$24 million for these vehicles. While HGVs do not contribute to the cost of congestion as much as passenger vehicles (\$81 million), the cost due to HGVs was still considered as significant. It was noted by the authors that the PCE in terms of congestion cost may be understated due to the cost related to the amount of emissions produced by larger vehicles during congestion, as well as the cost of accidents related to larger and heavier vehicles.

Al Eisaeia, et al. [113] used VISSIM simulation to study different HVs restriction strategies combined with varying percentages of HVs to determine the impact on average travel time, average speed, and vehicle delay for an Australian highway. The 8.8 km highway network was chosen as the study area due to its high concentration of HVs, as well as to consider the impact of HVs on a roadway where signalized intersections are present, as opposed to an uninterrupted freeway scenario. The restriction strategies were as follows: no restriction of HVs; restriction of HVs based on one-trailer or many-trailer truck class combinations; and restriction of all HVs

classes. All restriction strategies were simulated with varying HVs percentages from 5% to 30% in 5% steps. The simulation results showed that restricting all HVs on the roadway increased the efficiency of the traffic flow based on the performance measures, and that as more classes of HVs were restricted, the traffic flow was increasingly improved. Furthermore, as the percentage of HVs increased for any scenario, the average travel time and the delay were reduced, and the average speed increased as HVs were more restricted. This shows that with higher amounts of HVs, the greater restriction levels had more significant impacts on the traffic flow performance measures than at lower HVs levels.

The impact of trucks on driving behaviour and traffic flow was investigated by Kong, et al. [114] through a survey for passenger car drivers and through a cellular automaton model. The survey consisted of a questionnaire given to passenger car drivers in three different areas in China. Survey questions were related to the driving behaviour when following a truck, including questions about impact of trucks, and if so, what is the greatest impact, as well as lane changing, car following, and speed changing behaviour when behind a large truck. 95.77% of those surveyed indicated that they do feel an impact when following a large truck, with 34% and 39% citing that the main impacts felt are speed reduction and sight distance obstruction. 89% of those surveyed said that they would increase the gap between themselves and the truck, 72% indicated that they would reduce their speeds following a truck and in terms of lane changing, 47% of people would choose to change lanes immediately when the impact of a truck was felt, with 32% choosing to change lanes but not immediately. Moreover, when asked if they would change lanes even if the speed was constant when following the truck, more than half (54%) said that they would change lanes anyways. The results of the survey show that there is a psychological impact of trucks on passenger car drivers, and that more space, lowered speeds, and switching lanes is preferred when following a truck. The results of the cellular automaton model also showed that trucks have significant impact on passenger cars, with a greater degree of mixed traffic flow resulting in decreased traffic volumes, and that during congestion, the volume decrease and speed decrease is especially significant. Moreover, at greater traffic densities, traffic flow becomes more congested because of trucks.

Gao, et al. [115] observed traffic flow composed of varying HVs rates to look at the impact of these vehicles on average vehicle speed and average passenger car speed. The traffic volume, vehicle speed and vehicle type data were collected for a Chinese expressway, and the amount of HVs on the expressway was controlled by limiting or restricting the amount of HVs allowed to enter the expressway from on ramps. Three scenarios were implemented, the first with full restriction of HVs entering the expressway, thus having no HVs; the second scenario allowed some ramps to allow HVs, resulting in a low-moderate level of HVs on the expressway; the third scenario allowed all ramps to all HVs on the expressway, allowing the greatest number of HVs. Based on the vehicle type data collection, for the tests performed a range of 1.2% to 37.8% HVs was observed. The traffic flow patterns were categorized based on the volume-to-capacity (v/C) ratio in six levels from free flow conditions to congested ones. Similar trends were observed for different v/C ratios, such that as the HVs percentage increases, both the average vehicle speed and the average passenger vehicle speed is decreased. The most significant decrease in speed occurs with HV rates up to 20%, following this, the speed decrease is more gradual. The authors cite this as being possibly due to stabilization of speeds, and therefore traffic flow, with increasing numbers of HVs. Moreover, as the traffic volume increased, that is with increasing v/C ratio, the influence of larger vehicles on speed is more significant. A model was also made for each v/C ratio and varying HV rates for the prediction of speed and was compared to the Highway Capacity Manual (HCM) equations. The model created by the researchers based on the results of their analysis showed less discrepancy than that based of the HCM model.

The negative impacts of trucks on traffic flow have been mentioned with much agreement found by various researchers. Roh, et al. [116] set out to determine if there are any positive effects of CTs on traffic flow, or just the negative impacts of reduced speed and increased travel time and delay. The researchers performed analysis of traffic flow, including volume, speed, and vehicle type, data taken from detectors on a freeway in Korea. The results indicated that with mixed traffic flow, maximum flow rates were reduced, along with critical speeds. As the proportion of HVs was increased, there was a small decrease in speed, with the speed decreasing as well when traffic volumes or flow rates were increased. Moreover, when the proportion of HVs was increased, the impact of traffic volume on the speed was decreased. Based on the data, an increase in HVs resulted in minor speed decreases, but significant reductions of variations of speed. Platoons were found to be formed when a HV was the lead vehicle of the group, and these vehicles were found to dictate the driving pattern of the following vehicles. Specifically, for HVs leading vehicle groups, the result was found to be reduced speed variations between the fastest and slowest moving vehicles, which in turn led to fewer conflict points and accidents. The conclusion of the authors is that while HVs do have a negative impact on traffic flow in terms of reduced speed, there is also the benefit of traffic flow stabilization due to reduced speed and gap variance in the presence of HVs.

Afandizadeh, et al. [117] also looked at how HVs would impact traffic flow, choosing to specifically examine a roadway in a mountainous area and using AIMSUN simulation to model average travel time, lane changing, delay and average speed as a result of increasing HVs percentages. Data was obtained from the two-lane roadway for vehicle counts, vehicle class and speed. Different scenarios were simulated, namely varying the HVs percentage in the lane in question as well as in the opposite lane. For the lane in question, HVs percentages of 4% to 24%, 30% and 40% were examined. The simulation showed that with increasing HVs percentage, there was a decreased average speed, with a 15% decrease for a HVs rate of 40%. Moreover, up to 24% HVs, there was up to 20% more lane changing of passenger vehicles. The average travel time and delay were also increased with increasing HVs amounts. As for the increase of HVs in the opposing lane, 0, 10%, 20%, 50% and 70% increases were implemented, resulting in average speed increase until 50% HVs rate, as well as increased lane changing carried out by passenger vehicles. No significant change was observed for HVs as a result to a greater number in the opposing lane. Finally, the last simulated scenario added a lane to the highway, which led to average speed increases of 7%, however, this was offset by the speed reduction due to presence of HVs, and lane changing maneuvers were found to increase by 100% due to increased space.

Finally, Sari, et al. [118] considered the effect of HVs on traffic flow, specifically lane changing of these vehicles on the average speed of traffic on an Indonesian outer ring road. VISSIM microsimulation with strategy-based lane changing behaviour was used to model the four-lane, 600 m section of the ring road at high and low traffic densities. The lane changing scenarios implemented were high intensity HVs lane changing with no restrictions; moderate HV lane changing with one lane limiting passing for HVs; low intensity of HVs lane changing due to two-lane-passing limits for HVs. The simulation showed similar trends for high- and low-density traffic situations, specifically that as the amount of HVs increase, the average vehicle speed is reduced, with the most significant effects seen with more lane changes (high intensity lane changing with no restrictions) as opposed to where HVs are limited in their movement. As the intensity of HVs lane changing is reduced, the traffic speeds increase about 6% with each scenario. The final conclusions from the researchers were that for high density traffic conditions the speed decreased 2% with every 5% increase of HVs, while a 3% decrease in speed was observed for lower density traffic due to the same increase in HVs.

3.1.2 Truck Effect on Safety

In addition to the impacts of HVs on traffic, these vehicles also have safety implications, with more severe accidents possible, due to their size and weight. The Australian Department of Infrastructure and Regional Development [119] analyzed fatal and injury road crashes, specifically the impact of HVs on these crashes, and how they compare with lighter vehicles. Road crash data from 2008 to 2014, including crash characteristics such as severity, time and location and crash type, were the focus for the study. The results of the study showed that HVs are involved in significantly fewer crashes, however, the chances of a fatal crash were greater for crashes where an HVs is involved. It was also found that the crashes that involved HVs were predominantly multi-vehicle crashes, with about 74% to 78% of HVs crashes falling in this category, followed by crashes with pedestrians. Moreover, in terms of crash location and time regarding HVs crashes, 92% occurred during the week, with 40% to 67% on highways, and most crashes occur during the morning peak.

Similarly, Decker, et al. [120] examined road crashes involving trucks in Germany, specifically looking at truck driver injury severity. The crash data from 2000 - 2011 included injury type and severity, demographic data, collision area, and environmental circumstance for all road traffic accidents involving HVs. The researchers found that most, about 64%, of crashes with HVs involved cars, followed by other HVs. Of all the crash data collected, truck drivers were injured in only 13% of the accidents, while the injured party tended to be the driver of other smaller vehicles, and injuries occurred in most of the crashes examined. Fatalities to truck drivers only occurred in crashes with other truck drivers, not with smaller vehicles. It was also found that most of the crashes occurred during the daytime and mostly in rural areas.

Chen and Zhang [121] focused on the risk factors that contributed to fatigue related truck and HVs crashes in two cities in China, in order to determine the relationship between those risk factors and crash severity using a chi-square test. The crash location, time, vehicle and driver features, accident cause, and environmental factors for fatigue related crashes was obtained from traffic accident data bases. The accident data showed that most fatigue related crashes occurred in the morning, stated by the authors as due to higher levels of fatigue or drowsiness during this period. Moreover, up to 82% of the crashes had occurred during the week, and a majority happened on expressways or highways, and that fatigue related crashes were more likely to occur under adverse weather conditions or when there is speeding involved. Based on the statistical analysis, it was found that larger, heavier vehicles are more likely to be involved in multi-vehicle crashes, with an increased likelihood of fatal or serious injury crashes.

The relationship between vehicle weight and crash severity was the focus of the study by Xu, et al. [122]. Regression models were used to correlate WIM data such as total axle weights, total vehicle weights as well as truck counts and types, to crash severity, defined as fatal, injury, or property damage only. Crash data from 2011 to 2015 was used, and data included location and time of occurrence, geometric features and crash severity. The researcher's analysis showed that with an increase in mean vehicle weight of 1 kip, there is a 3.3% increase in the chances of the crash having fatalities or injuries. They concluded that HVs do have significant impacts on crash severity proportion for both single and multi-vehicle crashes. Also, it was found that for segments with a greater Annual Average Daily Traffic (AADT) had a greater chance of having fatal or injury crashes.

Road crash data related to pedestrian crashes with HVs and LVs was examined by Schubert, et al. [123], specifically injury patterns and frequencies, with the goal of enhancing ADS in terms of safety for VRU. All crash data, with a pedestrian and one vehicle involved, until 2021 from Germany was used for this research. The study showed that HVs and pedestrian collisions occurred significantly less frequently than with lighter vehicles and when they did occur, the collisions happened at lower speeds than when lighter vehicles were involved. However, despite the lower speeds, there was a higher chance of greater injury severity when HVs were involved, with almost 20% of severe injuries occurring due to HVs, compared to only 5% of passenger vehicles. Moreover, 49% of the pedestrians that were in head-on collisions with HVs suffered injury, and the chance of pedestrian run over was greater with HVs, likely due to their lower visibility. The regression analysis indicated that with greater speeds, there is greater risk of injury, however, HVs travelling even at low speeds have greater chances of injury in the case of a collision with a pedestrian.

Finally, Tian, et al. [124] also used a regression model to determine the relationship between different factors and how they impact truck crash severity. Crash data where at least one HV was involved was collected, including crash severity, as well as users, vehicle, road, environment and circumstantial factors, from the accident data base over five years in a region in China. Different regression models were created for urban roads, expressways and highways for the different factors affecting truck crashes. The results of the regression modelling for highways and expressways showed that the accidents with pedestrians have the highest chance of being fatal when in a crash with HVs. It was also found that HV crashes on expressways were more likely to have greater injury severity when drivers violated traffic rules, such as speeding or disregarding traffic signals.

The above research shows the impact that trucks and HVs have on safety, in terms of crash severity proportion, as well as when most accidents involving these vehicles occur. Even though collisions involving HVs are not as common as those involving lighter or passenger vehicles, fatal and severe injury crashes are more likely to occur when a truck or HV is involved, and multi-vehicle crashes are more likely when a HV is involved. When a collision with HVs occurs, it is usually the driver of other vehicles that sustains injuries or fatalities, with truck driver fatalities occurring in collisions with other trucks. HVs that were speeding or not following traffic rules were found to have higher chances of greater severity of injuries, as well as increasing weights and speeds of vehicles. Collisions involving pedestrians and HVs tended to be more severe than those with lighter vehicles, and the chance of severe injury or fatality was very high in the case of a pedestrian-HV collision. Finally, all the research suggests that most truck accidents occur on highways, freeways or expressways, during the morning periods of the week, and that adverse conditions increase the likelihood of crashes and crash severity.

3.2 Autonomous Vehicles and Autonomous Trucks Review

Over the past decade, there has been an increase in research related to AVs and their possible impacts in early-stage mixed traffic scenarios, as well as considering full autonomy. AVs will have significant effects on many aspects of society, but for the purposes of this research, the effects related to traffic, safety, environment and pavement will be looked at in this review. AVs penetration rate, reduced headway, increased speed, reduced wheel wander and other parameters related to AVs will be discussed.

3.2.1 AVs on Traffic

Friedrich [58] used macroscopic modeling as well as traffic flow theory to determine the effects that AVs will have on traffic, in terms of capacity and stability, given the efficiency of current infrastructure for both highway conditions as well as at signalized intersections. Density, volume and speed relationships were used to determine capacity values, given different penetration rates of AVs, and different truck proportions, including the capacity of a HVs only lane. For the highway conditions, the main variable changed in these scenarios was the preferred time gap of AVs versus CVs, with time gaps of 0.5 s as well as 1.15 s respectively. For mixed traffic scenarios, the time gaps were also dependent on the lead and lag vehicle types. For AVs following one another, 0.5 s was used as the time gap, for AVs following CVs 0.9 s, and finally for CVs following any type of vehicle 1.15 s was chosen. In a fully autonomous scenario, the highway capacity was found to increase by almost 80% with just passenger vehicles, or almost twice as much with 15% trucks. For mixed vehicle traffic, the capacity increases as the penetration rate of AVs increases, however, the rate of capacity increase is lower at lower penetration rates. Moreover, there will be greater stability with increasing penetration rates of AVs, due to fewer deviations of speed and headway compared to CVs, and fewer capacity drops in the network. As for the signalized intersection scenarios, the saturation flow rate and green cycle times were the deciding factors for capacity. For these scenarios, the reaction time was the main variable between AVs and CVs, as well as between trucks and semi-trailer trucks. The signalization cycle and other parameters were taken as standards or average based on German transportation networks. A 40% increase in capacity is observed in fully autonomous situations at signalized intersections, due to increased clearance speeds. Overall, capacity increases are expected, along with less lost time due to decreased headways and increased vehicle speeds.

The potential negative effects of AVs on traffic were the objective of the research by Calvert, et al. [125], where the researchers used microsimulation to observe how low automation levels, specifically vehicles with ACC or LKA, would impact capacity and travel times. The researchers used an IDM+ car following model along with Lane-change Model with Relaxation and Synchronization (LMRS) calibrated to ACC with desired time gaps, free flow speeds and desired lane change speed differences, and truck calibration was carried out as well. The desired time gaps for the ACC vehicles were in the range of 1.0 seconds to 2.0 seconds, and the different simulation scenarios also included varying penetration rates of 5% ,10%, 15%, 20%, and 30%-100% in 10% steps, as well as different truck proportions of 2%, 6% and 10%. The calibration of AVs compared to CVs included slower maximum vehicle speed, same deceleration rate but higher acceleration rate and a greater lane-change headway. Finally, the network analyzed was composed of three lanes with one on-ramp, acting as a bottleneck. The results of the simulation scenarios indicate that there is a limited negative effect on travel time and capacity until penetration rates are above 90%. For increasing gap times of ACC vehicles, a decrease in capacity is observed, and effect is more significant with increasing penetration rate, however, the breakdown capacity is higher with increased penetration rates. The results also showed that for a greater percentage of trucks the result was reduced capacity and increased travel times. A sensitivity analysis performed by the researchers indicates that maximum speed has a positive effect on capacity but the opposite effect on travel time, the lane change relaxation results in lower capacity with less relaxation and the opposite effect for travel time. The authors' conclusion to the hypothesis that low-level automation vehicles will have a negative effect on traffic is that yes, these vehicles will negatively impact traffic, however, the results are found to be minor.

Sharifillierdy [126] used VISSIM simulation to determine the effect of autonomous truck platoons on traffic flow, given different platoon configurations. Penetration rates of 0% to 100% with 10% steps, as well as headways of 0.4 s, 0.6 s, 0.8 s, 1.0 s and platoon sizes varying from 2 to 5 trucks were the variables used for different scenarios in this research. The average speed, total travel time and total network delay were the performance measures looked at for overall traffic flow impact for a freeway corridor that included multiple on and off ramps. The results showed that with an increase in penetration rate of AVs, the average speed of all vehicles is reduced, and total travel time increases with increasing number of AVs, with greater than 30% penetration having large impacts. Similarly, the total network delay increases as the penetration rate increases. Sensitivity analysis shows that the penetration rate has the greatest effect on the traffic flow, compared to the headway and the platoon sizes, however, the headway was also found to influence traffic flow at smaller spaces. The conclusions of the research indicate that for freeway sections with high traffic volume, as well as on and off ramps with high volumes, ATs platoons will have a negative effect on traffic flow, considering average speed, total travel time and total delay.

Calvert, et al. [127] also examined how platoons of ATs would impact traffic flow, with various headways, penetration rates of vehicles equipped with Cooperative Adaptive Cruise Control (CACC) technology, and platoon sizes. Using LMRS as well as using the IDM car following model, the total time spent in the system and the total network delay were determined. The simulation model also included on and off ramps, as well as major weaving sections, with penetration rates of 20%, 50% and 80%, as well as headways of 0.3s, 0.5 s, and 0.7 s, and considered 2 or 3 vehicle platoons. Through their simulation, the researchers found that with increasing numbers of CACC equipped vehicles, traffic flow deterioration was observed, with the reduction of traffic flow more significant during peak times. Only with 100% penetration, and the smallest possible headway was there found to be any reduction in network delay. Moreover, increased penetration rate of CACC equipped vehicles in a platoon resulted in increased time to merge, which was more significant on ramps with high penetration rates. Positive effects were found as a result of shorter headways; however, these results were not significant enough to offset the overall negative results of AT platoons. The authors recommend that ATs platooning be prohibited during congested times as part of their platoon management strategy.

Network capacity was similarly the objective of Lu, et al. [128], who used SUMO microsimulation to find the effect of AVs on an urban network, as well as to show how AVs would affect the macroscopic fundamental diagrams. Penetration rates of AVs were varied from 0% - 100% in 20% steps, and traffic demand was increased to congested conditions for the simulation. The default Krauss car following model in SUMO was used for the default or conventional vehicles, with the main calibration difference for AVs being the time headway, with values of 0.6 s and 0.9 s for AVs and CVs respectively. The results of the grid-network simulation showed that with full autonomy, greater speeds were achieved for the same densities, and a 16% increase in maximum flow capacity was found, showing a large variation with Friedrich (2016), who observed an 80% increase using just models. Moreover, with full autonomy, a 43% increase in critical density was observed, however, the increase in critical density is more significant for penetration rates greater than 40%. In the real-world network, the maximum flow and critical densities increase with greater penetration rates, and similar results are found to the grid-network.

Sala [129] used macroscopic modeling to determine platoon lengths given traffic demand and AVs penetration rate, as well as to estimate the capacity of a network under different platooning strategies. The researchers considered opportunistic platooning and cooperative platooning in a dedicated freeway platooning lane, where the former strategy occurs when consecutive AVs form a platoon, but may be disrupted due to CVs in the lane, whereas for the latter strategy, CVs may

be 'forced out' of the platooning lane, to allow for continuous AVs flow, and therefore uninterrupted platooning. The AVs were calibrated with shorter time gaps than CVs, based on emergency braking distances and communication abilities, with time gaps of 0.25 s or 0.6 s, compared to 1.65 s for CVs, when vehicles are travelling at 100 km/hr. The modeling results show that with a greater average platoon length, the capacity of the system is increased, and that greater platoon lengths are possible with the cooperative approach. It was found that a penetration rate of 50% in the platooning lane would increase capacities by 14% and 31% for the opportunistic and cooperative platooning strategies, and that the largest capacity increase would be possible with a dedicated fully automated platooning lane.

Elsahly and Abdelfatah [130] used microsimulation to observe AVs effects on traffic, however, they looked at the effects on undersaturated freeway conditions. VISSIM modeling of a Dubai network with varying penetration rates, as well as different undersaturated conditions was used to examine the effects of AVs on microscopic parameters, specifically average vehicle speed, travel time, and delay. A five-lane, 10-km road network with six junctions was used for the simulation, and default VISSIM calibration was used for CVs. For AVs calibration, appropriate car following and lane changing behaviour as well as lateral behaviour and signal control reaction were changed compared to CVs, with smaller headways and standstill distances, strict desired speed, and equal acceleration modelled. Penetration rates of 5%, 10%, 15%, 20%, 25% and 40% - 100% with 20% steps were used for the AVs. The results of the simulation indicated that greater penetration rates of AVs result in greater speeds for AVs and CVs in the network, however, greater speed increase for AVs and the increase in speed is more significant at greater penetration rates, or greater demand to capacity ratios. Furthermore, travel time for AVs and CVs decreases, however, once again, effect is more pronounced for AVs, due to smaller time gaps. AVs delay is reduced as penetration rate is increased with a greater reduction at greater demand to capacity ratios; up to 96.5% reduction in delay is observed. Reduced delay for CVs with increasing penetration rates as well, however, inconsistent variations of delay depending on demand to capacity ratios.

VISSIM microsimulation was also used by Park, et al. [131] to look at how AVs would impact traffic flow and capacity of urban networks, similar to the work done by Lu et al., mentioned above. The objective of their work looked at the impact of level 4 and higher AVs on traffic flow of urban roads given penetration rates, and road capacity. The simulation was based on a real-world road network in South Korea, with five lanes per directions, and 13 intersections over 4.5 km. The Wiedemann 74 car following model was used, as it was determined to be more suitable for urban networks, and CVs were calibrated with the default vehicle settings. The AV penetration rate was varied from 0% - 100% in 20% steps, along with six different traffic congestion levels, in order to see the effect on average travel time, vehicle speed, delay and stop frequency at intersections. While the congestion was kept the same, an increasing AVs penetration rate resulted in improved traffic flow over the whole network. Full AVs penetration resulted in a decrease of 17% and 31% for travel time and delay respectively, and a vehicle speed increase of 21%. With up to 84% penetration rate, the stopping frequency decreased with increasing penetration. When the traffic levels were changed in addition to the penetration rates, travel time, speed and stopping frequency all increased, however, effect is mitigated through increasing penetration rate of AVs.

Lee, et al. [132] considered the impacts of ATs platoons regarding safety and mobility of a freeway and on-ramp system. The researchers created a platoon formulation algorithm to make ATs platoons based on interplatoon distance, intraplatoon distance and varying platoon sizes, and then used the given platoon formations in VISSIM to simulate these configurations on a freeway section, including an on-ramp merging section. Mobility was considered given vehicle speed and the correlation to levels of congestion, while safety of the section was based on longitudinal safety, through rear-end conflict points, and lateral safety, through lane-change conflicts. Various penetration rates were used for different scenarios as well, ranging from 0% to 100% AVs penetration in 20% intervals, and the platoons were designated to the right-most lane. The impact of the increasing penetration rate was found to be insignificant on the mobility of both the right-most lane and the total system up to 80% penetration, however, at 100% penetration, the mobility was reduced for both, with increased levels of congestion along the section. As for the safety of the freeway section, the longitudinal safety was found to increase with increasing penetration rate with reduced conflicts, while the lateral safety was diminished.

Rather than considering capacity and traffic flow, Hamadneh and Esztergár-Kiss [133] used MATSim microsimulation to look at how fully AVs would impact modal shares in Hungarian transport networks, specifically to test the hypothesis that the introduction of AVs would decrease conventional travel modal share. Current transportation mode usage and number of travellers was determined using census data, and includes data on activity type, transportation mode, arrival and departure times, parking search times as well as sociodemographic factors. Modal utility functions for each transportation mode, including CVs, public transportation and AVs, were determined based on distances, travel time, and costs. Four main scenarios were simulated in MATSim: existing conditions based on real-world data was simulated and optimized, all conventional travel modes replaced with AVs, AVs substitution of public transit modes only, and AVs substitution of conventional vehicles only. For the scenarios with AVs, varying fleet sizes of AVs were used until a maximum wait time of 10 minutes was found. When all vehicles were replaced with AVs, the average trip time was reduced from the base scenario, however, was higher than for the other AVs replacement scenarios, and the greatest wait time, and greatest time of vehicles travelling empty was found. Also, the fleet size needed to fully replace all CVs and maintain a 10-minute wait time was the largest of the AVs replacement scenarios. With the full replacement of conventional personal vehicles, it was found that one AVs can replace 7.85 CVs based on the fleet size, and that with this scenario there would be the least empty-drive time. Finally, the full replacement of public transit by AVs showed the least average trip time and on average lower wait times, however, the vehicle miles travelled increased by more than 40%. Overall, the Vehicle Miles Travelled (VMT) is expected to increase due to empty vehicles, as well as increased trips. The marginal utility of AVs was 50%-90% of that of those for CVs. The conclusions of the authors on the effect on modal shares of AVs is that personal vehicle, and public transit use will decrease, but active transportation modes may increase. However, it may be noted that these outcomes will be dependent on the relative cost of riding in AVs or shared AVs.

Rahim [134] looked at the implications of ATs platoons on interstate highway traffic operations, environment, safety and costs. Platoons of varying size, headway, interplatoon distance, and penetration rates during peak hour and off-peak hour were simulated in VISSIM to determine network delay as well as total time to merge and diverge. The environmental impacts were determined using MOVES model to find fuel consumption and CO₂ emissions, while the effects on safety were found using SSAM software, and considered rear-end and lane-change conflicts, similar to Lee et al. above. Platoon sizes from two to five trucks were considered, with headways of 0.3 s, 0.5 s, and 0.7 s, along with penetration rates from 0% to 100% in 25% intervals. The researcher found that the total network delay was reduced with increasing penetration rates and

that fuel consumption was reduced in all scenarios with ATs. A two-to-four truck platoon with 75% penetration rate was concluded to have the best impact in terms of fuel consumption and emissions. Conversely, as the penetration rate was increased, the safety of the network was reduced during peak hour, with greater conflicts. This can be compared to the results of Lee et al. above, who found increased lateral conflict points [132]. The platoon headway impacted the total operating costs, such that a greater headway would be beneficial in terms of environmental costs, whereas the least headway was better for safety costs, however, an increasing penetration rate was found to overall decrease operational costs. The optimum headway for the platoon configurations based on different penetration rates was stated to be on the larger end of the range, with 0.7 s.

Malibari, et al. [135] compare different car-following models of a Japanese highway network to determine the impacts of varying AVs penetration rates and traffic levels on the lost time of the network. Simulation was done in VISSIM, and compared the IDM+, Helly and VISSIM default models. Real world traffic data was used, with detectors along 5.5 km highway with on and off ramp sections for comparison of results. AVs were calibrated with very short to long time gap values, as well as a faster and slower maximum acceleration value. Comparing the models and simulations to the actual data collected showed that speeds were underestimated before congestion occurred, and during congestion speed was lower than simulated. With the introduction of AVs, total lost time increased as AVs penetration increased; total lost time doubled by only 25% penetration rate, although the time lost was reduced with greater maximum acceleration. The increase in lost time is explained by the increased time gaps. Furthermore, with full AVs penetration, least amount of congestion was observed. Finally, the time gap was found to be a more significant parameter for affecting traffic and congestion than the maximum acceleration.

SUMO microsimulation was used by Babic, et al. [136] to determine the effects that increasing AVs penetration as well as varying time gaps between AVs would have on macroscopic traffic parameters, namely speed, density and total time in mixed traffic systems. The penetration rates used in the simulation are 0%, 30%, 50%, 75%, and 100%, along with time gaps for AVs of 0.3 s, 0.6 s and 1.2 s for a network 8 km in length and with increasing traffic flow. The results showed that with increasing penetration rates, the total time spend and mean travel time decrease, while the mean speed and density of the system increases, however, these effects were most significant at penetration rates of greater than 50%. Conversely, a greater time gap of AVs increased the total time spent and mean travel time as well as reduced the mean speed and density. Furthermore, with greater time gaps, queues were formed due to congestion.

Nippold, et al. [137] also used SUMO microsimulation, however, they examined the effects of AVs on capacity specifically at signalized intersections, as well as freeway conditions. The Krauss car following model was set for default vehicles for the simulation of the German road network, with only passenger vehicles considered. AVs penetration rates of 0%-100% with 10% steps were used for the simulation, and AVs time gaps were set to 1.5 s compared to 1.0 s for CVs. AVs were also calibrated to have no speed deviations and set with a lower deceleration rate than CVs. For penetration rates greater than 70%, the capacity given a corresponding density was reduced. Freeway conditions were observed as well, and the results showed that there is a maximum flow rate at 100% AVs penetration, however, the increase was only slightly more than the fully CVs scenario. Moreover, the worst flow conditions occurred with 50/50 split of AVs and CVs. The conclusion of the researchers is that free-flow traffic with larger AVs headways resulted in decreased capacity only at signalized intersections, not during freeway conditions.

Dai, et al. [138] used regression models based on personal trip surveys completed in three separate cities in China to determine how AVs will affect vehicle travel demand and VKT considering a fully autonomous scenario. Personal trip surveys were conducted, and the information obtained included trip data, as well as household and personal information. For the purposes of the research, only trips made by personal vehicles were considered for VKT calculations, which may be a limitation of the research, especially given the results found by Hamadneh and Esztergár-Kiss [133], who showed that public transportation modal share would be impacted as well by AVs. Dai et al looked at three different cities, a medium tourist city, a political and cultural center and an economic center. The results of regression analysis as well as extrapolated trends show that on average, the VKT increased by 33%-46% for all populations in the three cities, with the greatest increase for the age group of 61-year-olds to 80-year-olds, indicating that AVs provide the greatest increase in convenience for older populations. The results for the teenage or young adult population also show an increase in VKT, however, it is proposed by the researchers that there may not be acceptance of parents to allow their kids to take AVs alone, therefore limiting the increase with this age group. The researchers also suggest that despite the fact that AVs will provide the greatest increase in mobility and accessibility for older populations, this age group may be the most resistant or least trusting to the technology, which may result in not so increased VKT, at least at the beginning of the technology.

A summary of the research on AVs and their impact on traffic is presented in Table 2. The variables examined by the researchers, as well as the Measure of Effectiveness (MOE) and the results are displayed. The summary table shows that the Penetration Rate (PR) of AVs in mixed traffic scenarios has been researched extensively, with capacity, speed, travel time and delay parameters as common MOEs. As the PR increases, there is an increase in capacity of the system due to increased potential speeds and shorter headways, with the greatest capacity increase at 100% AVs penetration. Other than Sharifillierdy [126], a greater penetration rate of AVs results in an average speed increase, however, for average travel time and delay the impact of AVs is less conclusive. An increase in VKT or VMT has also been found to be a likely outcome of greater rates of AVs, especially for elderly and younger people. The impact of reduced AVs headways was also looked at, with greater capacity found to be possible as a result, while other research found that headway variance is not as significant as an increasing penetration rate. An increased headway was also found to cause greater travel times, delays and vehicle speeds. Further investigation on the effect of reduced headways and greater speeds on traffic parameters is necessary for the safe and efficient integration of AVs.

Table 2: AVs Impact on Traffic- Research Summary

Author; Year	Variables	MOE	Results
Friedrich, 2016	Penetration Rate % Trucks Headway	Capacity Stability	↑PR = ↑Capacity ↑PR = ↑Stability, ↓ Speed Variations ↓Headways = ↑Capacity
Calvert et al., 2017	ACC Penetration Rate % Trucks Desired Gap	Travel time Breakdown/ discharge Flow	↑PR= negative traffic flow effect ↑%Trucks = ↓Capacity, ↑Travel time > Gap = ↓Capacity
Sharifillierdy, 2018	Penetration Rate Headway Platoon Size	Average Speed Travel Time Delay	↑PR = ↓ Average Speed ↑PR = ↑Total Travel time and Delay PR more significant than Headway
Calvert et al., 2019	Penetration Rate Headway Platoon Size	Total Time spent Total Delay Merging	↑PR = ↑Total Time Spent and Delay Headway not Significant ↑PR = ↑Time to Merge
Lu et al., 2019	Penetration Rate Traffic Density	Traffic Diagrams	↑PR = ↑Capacity 100% PR= >Speed, at same Density
Sala, Soriguera, 2020	Headway Penetration Rate	Capacity	↑PR = ↑Capacity ↓Headways = ↑Capacity
Elsahly, Abdelfatah, 2020	Penetration Rate Demand-Capacity Ratio	Speed Travel Time Delay	↑PR = ↑ Average Speed ↑PR = ↓ Travel time More significant effects for AVs than CVs
Park et al., 2021	Penetration Rate Traffic Volume	Average Speed Travel Time Delay Stopping Frequency	↑Traffic Volume = ↑Travel time, ↓Speed ↑Traffic Volume = ↑Stopping Frequency ↑PR = ↓ Travel time, delay
Lee et al., 2021	Penetration Rate	Congestion Rear-end Conflicts Lane-Change Conflicts	100% Penetration = ↑ Congestion ↑PR= ↓Lateral safety ↑PR=↑Longitudinal Safety
Hamadneh, Esztergar-Kiss, 2021	AVs replacement of CVs, Public transit	Travel Time Fleet Size Waiting Time	Full AVs replacement = ↓Trip Time ↑VMT when Public Transit replaced Negative impacts on CVs, Public Transit
Rahim, 2022	Penetration Rate Platoon Size Headway Congestion	Network Delay Time to Merge/Diverge Fuel Consumption CO ₂ Emissions Conflict Points	↑PR = ↓ Network Delay ↑PR= ↓Fuel Consumption, CO ₂ Emissions ↑PR=↑Conflict Points
Malibari et al., 2022	Penetration Rates Traffic Scenarios	Time Lost	↑ PR = ↑ Time Lost 100% Penetration = least congestion
Babic et al., 2022	Penetration Rate Headway	Total time in system Travel Time Average Speed Density	↑ PR= ↓Travel Time and Time Spent ↑ PR= ↑ Average Speed and ↓ density ↑ Headway = ↑ Travel Time and Time Spent ↑ Headway = ↑Average Speed, ↓ density
Nippold et al., 2022	Penetration Rates Intersection vs Freeway	Capacity	100% Penetration = greatest Capacity 50% Penetration= minimum Traffic Flow
Dai et al., 2024	Survey Results	VKT	↑VKT overall and for elderly/youth

3.2.2 AVs and Safety

As previously mentioned, AVs are expected to have beneficial safety implications in the future. 94% of traffic accidents are the result of human error, and eliminating this source of error through autonomous, self-driving vehicle is predicted to decrease traffic accidents by 90%, as well as to reduce injury severity of crashes. The technology is still far from this point, and more research and testing of these vehicles is required to ensure that these goals are met in the future. Recent AVs research on the effects of these vehicles on safety, crash rates and severity are presented. Much of the research focuses on findings found from California transportation departments, since AVs testing is predominant in this state, with AVs legally allowed on roads, and mandatory accident or disengagement reports in the event of an incident.

Wang, et al. [139] examined crash and disengagement reports from the California DMV to analyze safety risks and factors associated with AVs. In California at the time of the study, over 3.7 million miles had been travelled by various AVs, with 621 disengagement reports, and 128 accidents reported. 63% of the accidents reported involving AVs happened while the vehicle was in some type of self-driving mode, resulting in the conclusion by authors that there is still much research to be done and advancements to be made before this technology is truly safe for roads. A significant finding of the research showed that for almost 94% of the accidents involving AVs, the at-fault party was the driver of another vehicle or other road user, such as pedestrians or cyclists, not the self-driving vehicle. The authors of the study state that perception errors, decision-making and action-taking performance of AVs are some of the challenging areas regarding AVs safety implications, and will need to be further researched, tested and improved.

AVs interactions with active road users, specifically cyclists and pedestrians, is the focus of the research by Alozi and Hussein [140]. The researchers looked at sensor and camera data from AVs in Canada, USA, and Singapore, where collisions or near-collisions occurred with cyclists and pedestrians in order to model and predict the risks associated with these road users and AVs. The models were categorized based on cyclist or pedestrian interaction, as well as based on clear or adverse weather conditions and type of AVs movement type (i.e., through movement, right turn movement, or left turn movement). The extreme value theory model showed that risky collisions were more likely with pedestrians than with cyclists, however, clear or adverse weather had no significantly differing effect on the likelihood of collisions. For weather conditions specifically, there were more, however not significantly more, collisions that occurred in clear weather than during adverse weather conditions. The reasoning for this was stated by the authors as increased caution by active road users during poorer weather, and therefore fewer accidents during adverse conditions. For pedestrians, the right turn movement was the riskiest interaction with AVs, while for cyclists the left turn maneuver had the most predicted collisions. It should be noted, that the analysis of camera and sensor data allowed the researchers to examine specific conditions resulting in conflict points or collisions, and it was shown that many of the potential collisions occurred due to pedestrian or cyclist traffic violations, such as crossing when or where they are not supposed. The AVs were found to have difficulty to predict the behaviour of pedestrians and cyclists when they performed these violations, indicating that understanding driving behaviour and road-use behaviour is still an area of improvement needed for AVs.

Similar to Wang et al. mentioned above, Almaskati, et al. [141] performed an analysis of California DMV crash and disengagement reports, as well as a critical review of related literature, to determine the relationship with AVs and safety risks and crash severity. The crash and disengagement reports from the California DMV included information such as details leading up to the collision, injuries or damage caused, location, type of crash, weather and road conditions, and timing of occurrence. The disengagement reports include weather and road conditions as

well as any other considerations relevant to the disengagement. The authors found similar results to Wang et al., such that AVs were not the responsible party for 87% of accidents they were involved. AVs were most likely found to be involved in rear-end collisions with other vehicles; however, the other vehicle was found to be at fault for most of the collisions. This is attributed to human driving behaviour when following AVs, specifically that human drivers do not expect AVs to follow every and all traffic rules, such as slowing down or stopping for a yellow light, instead of accelerating through it, as a human driver may do. Moreover, AVs were less likely to be involved in other types of collisions; it was specifically found that AVs collisions involving pedestrians only compromised 6% of total collisions, compared to 42% of conventional vehicle collisions. The effect of weather on traffic conditions involving AVs was also found to be insignificant, however, it is mentioned by the researchers that this may be a limitation due to the data and research available, all from California, which does not experience very much adverse weather, and therefore, the effects of adverse weather conditions are likely downplayed in results and discussions regarding AVs collisions.

3.2.3 ATs Platooning

A joint study carried out by Transport Canada, along with the U.S. Department of Transportation (DOT) and FHWA, examined the potential fuel savings related to reduced aerodynamic drag as a result of shorter gap spacing between vehicles in a heavy-duty vehicle platoon. The track-based fuel economy tests were performed at the Motor Vehicle Test Center (MVTC) in Blainville, Quebec, according to the SAEJ1321 Type II procedure. The first study [142] involved three-truck platoons, with testing parameters of two different speeds, 89 km/h and 105 km/h, spacing gaps ranging from 17.4 m to 43.6 m (space gaps correspond to 0.6 s to 1.5 s time gaps at 105 km/h), and truck-trailer weights of 14 000 kg and 29 400 kg. Moreover, the truck-trailers were modified with aerodynamic enhancing features and were compared to the standard truck-trailer configuration. The total difference in fuel consumed between each of the three vehicles in the platoon, the lead, middle and trailing vehicles, was determined as a result of the track testing. Fuel savings of at least 6% were obtained for the middle and trailing vehicles, regardless of the spacing, and the fuel savings of these vehicles tended to increase with decreasing spacing, and the greatest fuel savings occurred for the trailing vehicle. On the other hand, the lead vehicle experienced negligible fuel savings at the tested distances. Moreover, for the aerodynamically modified truck-trailers, there was an increase in fuel savings compared to the standard configuration. The speed effect was found to be insignificant in terms of fuels savings, and the parameter that had the greatest effect on the fuel savings was determined to be the vehicle's position within the platoon.

In the second part of the joint study by Transport Canada, USDOT and FHWA [143], closed-track tests were performed to observe the fuel savings of two-truck and three-truck platoons with greater variability in spacing compared to the first study. Time gaps of 0.14 s, 0.21 s, 0.31 s, 0.41 s, 0.6 s, 1.2 s, 1.5 s, 2.0 s, and 3.0 s were tested with the above parameters. The increased variety of spacings was considered to see if there is a spacing that would provide fuel savings to the lead vehicle in addition to the middle and trailing vehicles. Also, the truck platoons were tested with speed changes every 100 s to observe the impact on non-constant speed on the fuel consumption. It was found that the shorter headways did have an impact on the lead vehicle, which exhibited fuel savings at headways of up to 20 m (~0.7 s at 105 km/h) with up to 10% fuel savings at 4 m headway (0.14 s at 105 km/h). The greatest fuel savings occurred for the middle truck at the least separation distance, which showed increasing fuel savings with decreasing spacing, consistent with the results of the previous test. The trailing vehicles showed the greatest fuel savings around the 0.5 s headways. Overall, a net fuel savings of 13% was found among all vehicles in a three-truck platoon at the smallest separation between vehicles, down to only 4.5%

at the greatest distance. For the truck platoons at varying speeds, a decrease in fuel savings is observed. As for the two-truck platoons, the lead vehicle had similar fuel savings at different spacings compared to the three-truck platoon, and the trailing vehicle had similar trends as to that of the three-truck platoon, however at lower magnitudes. At a 6 m separation distance (0.21 s at 105 km/h) the three-truck platoon exhibited greater net fuel savings compared to the two-truck platoon at the same distance, with fuel savings of 11.5% and 7% respectively, therefore the researchers came to the conclusion that a three-truck platoon has greater fuel savings than a two-truck platoon. The two studies show the environmental benefits of automated truck platoons travelling at shorter spacings and provide the insight that the greatest benefits are achieved at constant speeds and small spacings for a three-truck platoon.

Alzahrani [144] examined the impacts of heavy-duty platoons on traffic operations, environmental factors and safety given vehicles capable of CACC and V2V communication. VISSIM microsimulation was carried out to determine traffic impacts, specifically, capacity with changing penetration rates, percent trucks, percent HVs in a platoon, different headways, desired speeds, and number of vehicles per platoon. MOVES software was used to examine CO₂ emissions, and the safety assessment was carried out through the number of conflict points from SSAM software. The microsimulation results showed that as the penetration rate of platooning vehicles increases, the capacity is increased as well, with significant increases after 70% of automated heavy-duty vehicles in platoons. The desired speed was the only parameter of those mentioned above found to be insignificant in terms of capacity change, and an increased time gap was the only parameter which was found to result in a decreased capacity. The percentages of trucks and percentages of trucks in a platoon were found to be the most statistically significant parameters when it came to CO₂ emissions, with desired speed having no significant change in CO₂ emissions. Finally, the number of conflicts was reduced with increased platooning, indicating possible fewer accidents. Rear end conflicts had the greatest reduction in number of conflicts, with similar trends for lane-change and front-end conflicts. Once again, the percent of trucks and those in a platoon had the greatest impact on safety factors.

Similarly to Alzahrani, Hassan et al. [145] looked at the broad effects of automated truck platooning on traffic operations, environmental factors, and safety using VISSIM simulation, as well as the impact that a lack of wander of trucks in a platoon may have on the asphalt pavement using finite element analysis. Truck platoons of platoon sizes varying from two-to-five trucks, with headways of 0.3 s, 0.5 s and 0.7 s, and penetration rates of 25%, 50% and 100% during peak and off-peak periods were simulating using VISSIM. The impacts on traffic operations were measured by the total network delay, and time to merge and diverge; the environmental impacts measured through the fuel consumption, and CO₂, NO_x, and PM₁₀ emissions; the safety of the platoons was assessed through time to collisions. The results of the simulation showed that truck platooning may be beneficial during off-peak hours in terms of improved traffic operations, safety and reduced emissions, however, during peak-periods truck platoons may cause reduced traffic operations and safety, but with still improved environmental effect. The recommended platoon characteristics suggested by the authors, based on cost analysis for the off-peak period is a four-truck platoon with a headway of 0.3 s and 25% penetration of AVs or a headway of 0.5 s and a penetration of 50%. During the peak period, a two-truck platoon should be implemented with 50% penetration and 0.7 s of headway between successive vehicles. The finite element analysis of fatigue and permanent damage of the pavement system under platoon loading along the same wheel path showed that the damage done to the pavement system would be significantly accelerated compared to loading that is distributed along a lane.

3.2.4 AVs Wander Effect on Pavement

As mentioned before, there has been much research on AVs: technology, safety, AI, etc., however, one area of lacking research is how AVs are predicted to interact with, and impact existing infrastructure, with asphalt pavement chosen as the focus for this work. AVs will differ from CVs in numerous ways, which may be advantageous or detrimental to pavement life. First, AVs will be able to be programmed to follow a certain wheel path, reducing or nearly eliminating their wheel wander, as opposed to CVs which follow a normal distribution with a standard deviation of 25 cm [146]. The following review will focus on the research on the impact of the lack of wheel wander on rutting and fatigue damage.

To begin, Chen, et al. [147] used finite element analysis to examine rutting of asphalt pavement. In addition to increased capacity and increased speeds, the researchers looked at normally distributed wheel wander with a deviation of 0.26m for CVs versus a range of wander, 0 m to 0.2 m, for AVs and the resulting permanent deformation. The increased speeds and increased capacity were considered due to the predicted smaller headway between adjacent vehicles, and the improved emergency braking resulting in greater achievable safe speeds. Their work showed that increased lane capacity would have detrimental effects on rutting, since there would be greater number of load repetitions on the pavement and therefore, an accelerated rate of rutting, with up to a 39% increase in max rut depth when the capacity is increased by 100%. Similarly, zero wander would have increased rutting as well, since there would be more repeated loading following one wheel path. Finally, it was shown that rutting decreased as a result of increasing traffic speed, due to the reduced load time of the vehicle on the pavement surface, with a decrease of 50% in maximum rut depth due to a speed increase from 10 km/h to 50 km/h.

Next, Noorvand, et al. [148] used Mechanistic Empirical Pavement Design Guide (MEPDG) for ATs wander impact on rutting. Normally distributed wheel wander was used for CVs, and normally distributed, uniformly distributed and zero wander scenarios were considered for ATs. Additionally, the researchers examined integrated versus segregated scenarios with varying truck distributions amongst lanes, and varying ATs penetration rates, as well as how the design pavement thickness would need to be altered. The scenarios that were examined were a reference scenario with no ATs, an integrated scenario with all trucks in the design lane or trucks evenly distributed amongst lanes, and a separated scenario where ATs use a dedicated lane or two depending on the volume. Equivalency factors were used to relate the number of passes provided by a vehicle with no wander to the standard normal CVs wander. The MEPDG results show that ATs with uniform wander would cause less damage and a reduced pavement thickness compared to a normally distributed, and especially a zero-wander case. With zero-wheel wander, the detrimental effects to the asphalt would occur at an ATs penetration rate of just 10%. The benefits of uniformly distributed ATs are greatest at penetration rates greater than 50%, and when trucks are distributed among all lanes, rather than confined to one lane. The implementation of a segregated lane for ATs is suggested by the researchers given a cost analysis, and since pavement reduction of a segregated lane may occur with as low as 70% penetration, while for an integrated lane, 90% ATs penetration would be required for pavement thickness reduction.

Zhou, et al. [149] investigating wheel wander of ATs and rutting, specifically pertaining to possible vehicle hydroplaning. Experiments performed on wheel wander of AVs showed that there is 3 cm to 7.5 cm of deviation from a normally distributed wander, compared to a deviation of 25 cm for CVs. The ATs wander used in the study was a normal distribution with 7.5 cm of deviation, and two scenarios were considered, 100% ATs or no ATs. Pavement performance was simulated for these scenarios using the Texas Mechanistic-Empirical (TxME) program. Rut depth increased under 100% ATs, with 32% faster time to critical rut depth found and a 20% reduction in fatigue

life; uniformly distributed wander was recommended as way to improve pavement life and safety, specifically to reduce the increased risk of hydroplaning due to increased rutting as we a result of increased ATs penetration rate.

Lateral position was also the focus of the works by Gungor and Al-Qadi, who looked at the overall impact on pavement damage through modified MEPDG to account for wheel wander, as well as Life Cycle Cost (LCC) Assessment considering agency cost only. The first approach followed an 'All for One' approach [150], wherein the lateral position of the entire platoon was optimized for best pavement performance, as opposed to the second method which followed a 'One for All' approach, where the lateral position of individual trucks within a platoon was optimized for a given number of trucks [151]. The optimization of the lateral position of both the platoon and the individual vehicles resulted in improved pavement performance and life cycle cost, specifically a 50% and 9% reduction in LCC from the 'all for one approach' and 'one for all' approaches respectively. Moreover, in the 'one for all' approach, where the individual lateral positions were optimized, different longitudinal spacings, or headways, were considered as well in finding the optimum platoon configuration given LCC, and for all truck platoon combinations, the optimum headway was found to be nearer the lower bounds (i.e., 3 m), than to the upper bounds of 10 m, for instance with 3.3 m for a ten-truck platoon being the optimum for that case. However, for the case of individual vehicles in a platoon with varying lateral positions within the platoon could reduce the benefits that platooning provides regarding fuel-consumption and emission reduction, since the string stability of the system will be reduced.

The energy savings associated with platooning, and the effect of different configurations had been the topics of previous research, however, Song, et al. [152] were one of the first that looked at optimizing platoon configurations, specifically for ATs, with the goal of minimizing pavement damage without compromising the energy savings. The authors used finite element analysis to assess the pavement fatigue damage given lateral and longitudinal offset, as well as computational fluid dynamics to assess the fuel savings. The longitudinal position of the truck was used to determine the air drag coefficient, which is increased as the spacing is increased, indicating greater drag. Given these parameters, it was determined that maximum fuel saving rate occurred when the spacing between adjacent vehicles is at its minimum (0.5 times the length of the truck). The research also looked at varying platoon numbers, with an increase of trucks in the platoon resulting in reduced air drag, with a slower increase in fuel-savings rate given greater than four trucks. Finally, varying lateral offsets from 0.21 m to 1.05 m were investigated as well, with results showing that an increase in lateral offset results in a decrease in pavement damage. Given a lateral offset of 150 mm between lead and following vehicles, pavement fatigue damage was reduced by 41%, with only a 1% decrease in fuel-savings rate. A four-or-less truck platoon, at 0.5 times the truck length headway and with 150 mm of lateral offset is recommended by the authors for optimum pavement performance and energy savings considerations.

The impact of wander on fatigue damage was also studied by Marsac, et al. [153], who used numerical simulation by Viscoroute 2.0 software and the French pavement design method to assess strain at the bottom of the asphalt concrete layer due to different wander patterns. Lack of wandering resulted in about six times more damage for wheel path distance greater than 0.25 m for a three-truck platoon.

Similar to Noorvand et al, Rana and Hossain [154] considered wander, integrated versus separated lanes, truck distribution amongst lanes and penetration rates of ATs, and took time of day into consideration on the effect of pavement performance, specifically rutting, bottom-up and top-down fatigue, as well as the International Roughness Index (IRI). The pavement distresses were simulated using AASHTOWare software and MEPDG. The wander that was considered by

the researchers was zero wander, normally distributed wander with 7.5 cm and 25 cm deviation, and uniformly distributed wander. The results showed that asphalt concrete rutting increased with increasing ATs penetration rate, as well as fatigue cracking, however, at high penetration rates, even distribution over lanes, and therefore more evenly spread rutting, can mitigate the increased damage. For all penetration rates, even distribution of truck among lanes resulted in reduced pavement distress. Moreover, their experiment showed that at low temperature, for instance under nighttime driving conditions, would result in 24% and 24%-30% reductions in rutting and fatigue cracking respectively than daytime driving endeavors. The researchers recommend a separated lane for uniformly distributed trucks for minimized pavement distress, with uniform distribution resulting in 30% and 94% reduction in rutting and fatigue cracking respectively.

In a report for the Illinois Center of Transportation, Al-Qadi, et al. [155] examined overall effects of ATs platoons on pavement, including a Life Cycle Assessment (LCA), Life Cycle Cost Analysis (LCCA) and 3D FE analysis for pavement performance, as well as a genetic algorithm to determine the optimum platoon configuration. The effect of wander was looked at by comparing normal distributions with 25 cm deviation to represent CVs, and ATs represented by zero wander and uniformly distributed wander; MEPDG damage calculations used for rutting and fatigue cracking. Channelized, or zero wander traffic was found to increase rutting and fatigue cracking by 38% and 212% compared to the researcher's optimized configuration, with an approximate 150% increase in fatigue cracking from zero wander compared to uniform wander. Based on the LCCA, the optimum platoon configuration consisted of six trucks with 20 ft spacing between and the LCA showed that platoon configurations consume less fuel and produce fewer emissions. The optimally configured platoons led to 46%, 36% and 42% decreases in life cycle cost, energy consumption and emissions respectively.

Guo, et al. [156] used FE analysis to examine the traffic volume under increasing AVs penetration rates and coupled these results with different wheel wander distributions to examine the impact that AVs would have on asphalt rutting. The aim of their study was to find speed control strategies for AVs considering rutting, pavement roughness and bumpiness as well as vibration, and their correlations to comfort. The analysis of traffic showed that increasing AVs penetration rates resulted in increased traffic volume due to higher vehicle speeds and decreased vehicles spacing. A speed increase of 45%-72% is found for 100% AVs penetration rate, with a traffic volume increase of 114%. The distribution modes that were coupled with the increasing penetration rates consist of a centralized distribution, bimodal Gaussian distribution as well as uniformly distributed, and uniform distribution at both ends for the distribution of the wheel trajectory. The results show that increasing penetration rates resulted in increased rutting due to greater load repetitions, however, this effect could be mitigated through the implementation of uniform distribution at both ends for the wheel trajectory. Moreover, their results verified previous works that showed that channelized, or centralized in the case of the Guo et al, would have a very negative impact on pavement permanent damage, with any type of wheel distribution resulting in less rutting than zero wander, channelized distribution.

Okte and Al-Qadi [157] introduced an approach for incorporating lane position into pavement performance prediction and verified their model and framework using different mixed traffic scenarios. The rutting and fatigue cracking models from MEPDG were adapted to include wheel position through the use of quintiles within the lane, with normal distribution of wheel passes over each quintile. The expected response framework gives stress or strain as a function of lane position. The human driven vehicles were assumed to have a zero mean normally distributed wander with a standard deviation of 10 in, while the AVs followed a uniform distribution within the lanes. Thin and thick, weak and strong pavement structures were analyzed under different traffic compositions as well as different wheel distributions. Wheel distributions included channelized,

normally distributed and uniformly distributed. Results of the study showed that the framework was suitable for predicting both fatigue cracking as well as rutting. Moreover, the researchers also confirmed that channelized traffic would result in an increase in fatigue cracking and rutting, with a 60% and 25% increase in the respective distresses, however, a distributed later position resulted in a decrease of 40% and 20% for fatigue cracking and rutting respectively.

Similarly, Georgouli and Plati [158] also looked at the impact of wheel wander on pavement performance using modified critical responses from MEPDG and 3D FE analysis software 3D-Move. The researchers examined different wheel wander distributions including zero distribution, or channelized flow, normal distribution with zero mean and a standard deviation of 25 cm representing CVs, uniformly distributed wheel wander, and finally Laplace distribution, on pavements of varying thicknesses and stiffnesses. Similar to Okte and Al-Qadi, the MEPDG equations for fatigue cracking and rutting were modified using the same 'distribution amongst quintiles' technique to determine the damage equivalency factor, which the authors defined as the ratio of damage for wheel wander vs zero wander. The results are consistent with other research; greater rut depth was found for zero-wheel wander, with the greatest reduction of rut depth from a uniformly distributed wheel path. Moreover, the results indicate that the wheel path distribution has more of an impact on rutting than on fatigue cracking and the impact was more significant for thinner, stiffer pavements.

Rutting performance was considered by Yeganeh, et al. [159] by means of 2D FE analysis considering lane width, lateral wander, penetration rate for integrated as well as segregated scenarios. Penetration rates from 20% to 100% for the integrated case, coupled with wheel path distributions of zero wander, normal wander and uniformly distributed wander for both uniform time wander and uniform frequency wander. The standard deviation of the normally distributed wheel wander was dependent on the lane widths. For the reference scenario with no AVs, an increase in lane width resulted in less rutting, and for all cases with all distribution types, narrower lanes resulted in greater rutting. In all scenarios other than the uniform time distribution, an increase in AVs penetration resulted in greater rutting, which is consistent with previous research. Notably, at high penetration rates, specifically 80%-100% AVs, there is a significant increase in the amount of rutting that occurs. The recommendation of the authors is uniform frequency wander for AVs penetration rates up to 80%.

Fahad and Nagy [160] used FE analysis to examine a few ATs platoon parameters' effects on fatigue and rutting of pavement. The parameters included axle configuration, headway and interplatoon distance, lateral truck distance, and the results of the analysis provided optimum headways for varying wander modes. Three truck platoon types were analyzed: semi-trailer only, rigid body trucks and equally distributed random traffic mix, with zero wander and uniformly distributed wheel paths. The results indicated that the random mixed traffic scenario with uniformly distributed trucks would result in the least accumulated strains. For zero wander mode, the fatigue life of the pavement was reduced by 40% for 5m headway. Channelized traffic also resulted in an increase in rutting with a 38% increase in rut depth for platoons at 2 m headway. Therefore, the researchers recommend 3 m headway and uniform traffic distribution among the lanes. The platoon configuration that is recommended by the authors is a 4 -truck platoon with 4 m between trucks and 10 m between subsequent platoons.

This section presented an in-depth review on AVs wheel wander, or lack thereof, and the impact on asphalt pavement rutting and fatigue cracking. The ADAS features, such as lane centering and LKA will allow AVs to maintain their lateral position within a lane, however, this type of channelized driving increases the rate of damage compared to a distributed wheel load since there is an increased number of load repetitions over the same area. Uniformly distributed,

normally distributed, or lateral offset of AVs should be considered to avoid the increased damage to the pavement.

3.2.5 AVs Speed and Spacing Effect on Pavement

There has been much research carried out regarding the impact that wheel wander, or lack thereof in the case of AVs, will have on pavement performance, however, research on other parameters related to AVs driving and their impact on pavement performance has been less thorough. The next section will discuss the literature regarding AVs parameters such as speed and constant time or distance headways. These parameters are related to the loading time and rest period on the asphalt respectively. AVs and CAVs will be able to maintain certain headways and speeds, smaller headways and greater speeds than human driven vehicles and therefore, this should also be considered for pavement performance at different penetration rates.

In addition to examining the lack of wheel wander of AVs and this impact on pavement rutting performance, Chen, et al. [147] also looked at increased traffic volume and greater vehicle speeds, as these are predicted feature of AVs driving, and the result on permanent deformation. The results showed that an increase in traffic volume would increase rutting due to increased total load repetitions and therefore, total longer loading times, whereas, an increase in speed had a decrease in rutting for the opposite reason, where faster speeds resulted in shorter loading times. A 100% increase in capacity for any given number of load repetitions was found to increase the rutting by about 40%, while for a speed increase from 10 km/h to 50 km/h the researchers found a reduction in rutting of almost 50%.

A comparison between individual ATs and platoons of ATs was carried out by Leiva-Padilla, et al. [161]. The researchers examined the impact on pavement performance, specifically fatigue strain, of truck platoons with varying speeds and wheel wander, given constant headway between trucks in a platoon as well as considering summer and winter conditions. Only two wandering distributions were considered; normal distributions with standard deviations of 15 cm or 25 cm, along with speeds varying from 40 km/h to 80 km/h. The results showed that platoons, compared to individual trucks, induced greater strain in the pavement and it was found that truck platoon configurations during the summer have a greater effect on the fatigue strain than during the winter, due to the viscoelastic behaviour of the material at higher temperatures, compared to elastic performance at lower temperature. Moreover, an increase in the wander interval could reduce the tensile strain values and therefore increase pavement life. For the winter scenario, the strain values were unchanged with varying load configuration, speed and platoon number, however, during the summer, all these parameters were found to influence the strain.

In addition to the previous study where the speed and wander effect of truck platoons during winter and summer conditions is considered, Leiva-Padilla et al also looked at varying time gaps and number of trucks per platoon along with the above parameters [162]. Truck time gaps of 0.5s, 2.0s and 3.5s were considered, along with up to six trucks per platoon at varying temperatures. The researchers found that when the number of trucks is increased at temperatures greater than 35 °C, there is a logarithmic increase in strain values, and that the rate of increase in strain accumulation is greatest at temperatures above 55 °C and when the time gap is reduced. Moreover, the results showed that for all load levels a reduced time gap, or headway, between successive vehicles in a platoon, there was an increase in maximum strain values. For summer or warmer conditions, the authors suggest fewer trucks per platoon at a greater time gap and with a reduced load as the best strategy for mitigating pavement damage.

Fahad and Nagy [163] also considered vehicle speed coupled with wheel wander, with speeds of 5 km/h, 50 km/h and 80 km/h for each wander type on fatigue damage using 3D FE element analysis. Their results indicated that the highest maximum strain occurred at the lowest speeds, similar to the results found by Chen et al. mentioned before, and that the magnitude of the strain decreases when the vehicles decelerate. Also, the rate of increase of strain is greater at lower speeds, compared to higher speeds. Moreover, the significance of the speed on the fatigue performance was greater for the zero-wander wheel path compared to the uniformly distributed wheel path. Therefore, for greater pavement performance, the researchers recommend AVs travel at higher speeds, 80 km/h, with uniform distribution amongst the lanes.

In their 'One for All' study previously mentioned, Gungor, et al. [151] aimed to optimize the location of each truck in a platoon, thereby, taking into consideration the spacing between subsequent vehicles, also known as the headway. In their optimization, an upper bound of 60 m between vehicles was used from agency limits for trucks, and the lower limit of 3 m spacing was used as the minimum space required for ATs to emergency brake, and the optimization was carried out to find the ideal platoon configuration considering minimized fuel costs as well as optimal pavement performance, specifically rutting. The results obtained from the optimization gave platoon skeletons for varying platoon sizes, with the optimized configurations resulting in 9% reduction in life cycle cost. It should be noted that for all of the platoon configurations, the spacing between the trucks is closer to the minimum lower bound of 3m than higher values. For instance, given a 10-truck platoon, the optimal spacing is 3.3 m between truck in the platoon.

Elwardany, et al. [164] used reliable multivariate regression and artificial neural network (ANN) models to examine mix parameters, temperature variation, and varied rest period, and how these parameters would influence the fatigue life of asphalt. Their main aim was to look at the effect that ATs platoons would have on the fatigue life of pavements, specifically, the effect of reduced rest period on fatigue performance. A reference rest period of 10 s, to simulate human driven trucks from previous studies, was compared to rest periods of 0, 1, and 5 s after each 0.1 s loading cycle. The results of a previous study carried out by the National Cooperative Highway Research Program (NCHRP) [165] showed that rest periods following a loading cycle, instead of another load cycle immediately resulted in stiffness reduction at a lower rate. Different regression models were used, linear regression, quadratic regression and ANN regression. The ANN model proved to be superior to the linear and quadratic models due to greater correlation coefficient, R, and lower errors. Analysis using the model predicted a 7% reduction of fatigue life due to an 85% decrease in headway between ATs in a platoon, for a final rest period time of 1.5 s. The number of load repetitions to failure decreased at lower temperatures and higher applied strain as shown by the greater stiffness ratios. The authors recommend a four-second rest period between trucks, or a 60% reduction in truck following distance, as this result was found to have a negligible reduction in fatigue life of the asphalt.

In addition to considering wandering parameters associated with ATs, Al-Qadi, et al. [155] in their report for ATs platoon impacts on pavement for the state of Illinois, also take truck spacing, or headway, into account when conducting their life cycle assessment, life cycle cost analysis and 3D FE analysis models. Moreover, different pavement thicknesses and corresponding speeds, varying temperatures as well as five different loading magnitudes were analyzed as well. Asphalt distress for fatigue cracking and rutting was based off of modified MEPDG distress equations for rutting, and both bottom-up and top-down fatigue cracking. Based on the results for the predicted horizontal tensile strain, given both the longitudinal and transverse directions at the surface and bottom of the HMA respectively, it was shown that a rest period of greater than five feet (~1.5 m) does not have any impact on the fatigue cracking of the material, since the strain after this amount of rest period was found to have been recovered almost completely. Similarly, the influence of the

rest period on the vertical compressive strain, related to rutting, was found to be insignificant for spacing greater than five feet. Due to the spacing of five feet presenting a 'safety issue', according to the authors, the conclusion of the research is that rest period does not have affect rutting or fatigue cracking. However, these results should be verified experimentally, since only FE analysis was used for distress prediction, and due to the assumptions made by the researchers, namely that the asphalt was a perfectly linearly viscoelastic material. This assumption is made to simplify calculations; however, it is not perfectly accurate in describing the characteristics of the material, and therefore the conclusion that Al-Qadi et al came to will be used as a hypothesis and verified experimentally through this research. From their optimized platoon configurations, the authors recommend six-truck platoons, with 20 ft (~6.1 m) spacing between trucks.

Ramakrishnan, et al. [166] continued the work of Al-Qadi and colleagues for a report for the Center for Connected and Automated Transportation, with a specific goal of examining the effect of rest period on fatigue and permanent deformation through mechanistic approaches, as well as to validate those results experimentally through various tests. Finite element analyses were also carried out, with similar findings as Al-Qadi et al under similar conditions, specifically, that rest period is ineffective for truck spacings greater than 10 ft. The experimental tests included the MSCR, complex modulus and RPLD tests on two different mix designs with four different binders. Different stress levels, temperatures and rest periods were considered as well, specifically of note, rest periods of 0.18, 0.57 and 2.5 seconds. The MSCR results showed that an increased rest period resulted in improved recovery of viscoelastic strain, which would result in less accumulated damage. However, the RPLD test showed increasing asphalt concrete permanent deformation with increased rest period, showing inconsistencies between the two tests. The data collected from the tests was used to create mechanistic shift models that relate the permanent deformation to rest period, in addition to temperature and load time.

Table 3 presents a summary of the research presented on the AVs speed and spacing effect on asphalt pavement rutting and fatigue cracking. Most of the previous work considers fatigue life, however, the impact on permanent deformation is only taken into account by a couple of the researchers. A decrease in rest period or time gap has a proportional effect on the fatigue life, while increased speeds show benefits for both rutting and fatigue life improvements. Al-Qadi, et al. [155]. and Ramakrishnan Ramakrishnan, et al. [166] found that reducing rest periods beyond five feet (1.5m) and ten feet (3 m) respectively, would have negligible impacts on pavement damage, while from a cost perspective, Gungor, et al. [151] observed that shorter headways were optimum in terms of minimizing pavement damage without sacrificing fuel efficiency. Finally, Leiva-Padilla found that platoons induced greater fatigue damage compared to individual vehicles [161], and that with increased trucks per platoon, greater damage was observed [167].

Table 3: AVs Effect on Pavement Summary

Author; Year	Variables	MOE/Test	Results
Chen et al., 2016	Wander Traffic Volume Speed	Rutting	↑ Traffic Volume = ↑ Rutting ↑ Speed = ↓ Rutting
Gungor et al., 2020 [151]	Longitudinal Spacing Lateral Spacing	Pavement Damage Fuel Savings	Shorter Headway= Optimum
Elwardany et al., 2021	Rest Period	Fatigue Life	↓Rest Period = ↓ Fatigue Life
Al Qadi et al., 2021	Longitudinal Spacing Lateral Spacing Wander	Pavement Damage	Rest Period <5 ft = Negligible
Ramakrishnan et al., 2021	Rest Period	Permanent Deformation	Rest Period <10 ft = Negligible MSCR: ↑ Rest Period = ↑ Strain Recovery RPLD: ↑ Rest Period = ↑ Permanent Strain
Fahad and Nagy, 2022	Speed Wander	Fatigue Life	↓Speed = ↑ Strain
Leiva-Padilla 2022, [161]	Platoon vs. Individual Trucks Season/Temperature Speed	Fatigue Life	Platoons = ↑ Pavement Damage
Leiva-Padilla 2022, [167]	Temperature Time Gaps Number of Trucks	Fatigue Life	↓ Time Gap = ↑ Max Strain > # Trucks = ↑ Max Strain

3.2.6 AVs and Pavement Costs

In addition to the research and work done relating AVs, specifically ATs, on pavement performance, there has also been research relating these vehicles with associated infrastructure costs. The costs that can be considered include agency costs, which are costs related to design, construction, and maintenance or rehabilitation of roadways, as well as the user costs which are the costs for the users of the system in terms of fuel, vehicle maintenance and also the cost associated to delay or congestion as well as cost of crashes and accidents.

Hoque, et al. [168] investigated the agency and user costs associated with different lane management strategies of connected and autonomous truck platoons through a LCCA. Specifically, their main objectives were to the lane management strategy with the least cost considering construction and rehabilitation from the agency side and fuel costs from the user side. The lane management strategies considered were a prohibitory approach, where truck platoons were not allowed to form, a conditional approach, where platoons could be formed if certain conditions were met, such as time of day or number of trucks, as well as truck platooning permitted unconditionally on one lane, two lanes, a truck only lane or on any lane. It was found that truck platoons resulted in reduced construction costs and user costs and that there was a decreased fuel consumption of trucks travelling in a platoon. However, it was also determined that if trucks are allowed to platoon in inner lanes that were not considered as the design lane the rehabilitation costs will increase as part of the agency cost. To prevent this, pavement would have to be

designed with a greater distribution of trucks to inner lanes. The greatest savings in cost was found when truck platoons were allowed to form unconditionally on any lanes, with a decreased agency cost due to lower rehabilitation costs when trucks are distributed amongst all lanes.

Finally, Al-Qadi, et al. [155] in their report mentioned previously, performed a LCCA and LCA to examine the effect of ATs platoons and different pavement and traffic conditions on the environment and associated costs. For the LCCA, user costs were considered, specifically costs related to fuel consumption and maintenance costs, for four different scenarios: a reference case with only CVs, truck platooning with channelized driving, 50% mixed traffic scenario and a scenario based on their optimized platoon configuration. Based on the LCCA, the best performing truck platoon configuration consisted of 6 trucks spaced at 20 feet (~6.1 m). The results of the analysis showed that there is a fuel reduction due to platooning, therefore a reduced user cost, and that for the mixed traffic scenario compared to the human driven case, the former had a lower user cost than the latter. A LCA was also carried out to quantify environmental effects of platoons. The results showed that platoons always had decreased fuel consumption. Based on the results of the analysis and assessments carried out demonstrated that the authors' optimum platoon configuration would reduce life cycle cost, energy consumption and emissions by 46%, 36% and 42% respectively.

Table 4 shows a summary of the works presented related to AVs and pavement. The focus of the research, whether it be about rutting or fatigue or both, and which parameters were considered, whether wander or speed and/or spacing are shown in the table. Moreover, the research with work validated experimentally or just through mechanistic or empirical methods such as models or FE analysis is shown as well. It can be seen that the majority of the previous research considers the effect that a lack of wheel wander will have on rutting and fatigue. There is, however, much less work focusing on the vehicle speeds and headways, or in other words the loading time of trucks and the rest period between successive trucks, and how these parameters would affect the stresses. While lateral wander is a critical factor is how AVs will impact asphalt pavement, for the purposes of this work, the wander will serve as a confounding variable that is being controlled, and an emphasis is placed on the impact of speed or spacing of AT platoons on asphalt to fill the gap in the research. The next section describes how these parameters, specifically loading time and rest time as they are related to speed and headway respectively, affect asphalt concrete fatigue cracking and rutting.

Table 4: AVs Effect on Pavement Literature Summary

Authors, Year	Rutting	Fatigue	Wandering	Speed/Spacing	Validated Experimentally
Chen et al., 2016	X		X		
Noorvand et al., 2017	X		X		
Zhou et al., 2019	X	X	X		
Gungor et al., 2020 [150]	X	X	X		
Gungor et al., 2020 [151]	X		X	X	
Song et al., 2020		X	X		
Marsac et al., 2020		X	X		
Hoque et al., 2020					
Al-Qadi et al., 2021	X	X	X	X	
Ramakrishnan et al., 2021	X			X	X
Elwardany et al., 2021		X		X	
Rana and Hossain, 2021	X	X	X		
Guo et al., 2022	X		X		
Leiva-Padilla et al., 2022 [161]		X	X	X	
Leiva-Padilla et al., 2022 [167]		X	X	X	~
Fahad, and Nagy, 2022		X	X	X	
Okte and Al-Qadi, 2022	X	X	X		
Georgoul and Plati, 2022	X	X	X		
Yegeneh et al., 2022	X		X		
Fahad and Nagy, 2023	X	X	X	X	

3.3 Pavement Review

3.3.1 Speed/Frequency and Fatigue

To begin, Jiang-Miao, et al. [169] examined fatigue performance of asphalt concrete with varying load weight, temperature, vehicle speed and asphalt aging using a four-point bending beam test. The vehicle speeds were varied through different test frequencies: 5, 10, and 25 Hz. Their results showed that a 1 Hz increase in frequency results in a 2.2% increase in stiffness, however, based on regression analysis, frequency changes from 5 Hz to 15 Hz have little impact on fatigue life. The order of significance of the different parameters on fatigue life was found to be loading, test temperature followed by long-term aging, while the short-term aging and frequency were found to be statistically insignificant.

Al-Khateeb and Ghuzlan [170] performed Indirect Tensile Tests (IDT) to determine fatigue life given varying parameters, specifically, frequency, temperature and stress level as well as the coupling effect. A stress-controlled test with haversine loading of six different stress levels, two temperatures and four loading frequencies from 3 Hz to 10 Hz were tested. The experimental results in addition to the power fatigue model showed that an increase in loading frequency results in a longer fatigue life. In other words, a greater speed and therefore lower loading time will result in better fatigue performance. Moreover, there was an increased stiffness for higher speeds, which would result in greater rut resistance, however could result in a reduced fatigue life at lower temperatures.

Shafiee, et al. [171] examined the effect of varying speeds and wheel wander on the longitudinal and compressive strains in the different layers of pavement in a field experiment carried out in the University of Alberta integrated road research facility and compared with the MEPDG performance predictions. The four speeds tested in the field experiment are 10, 20, 40, 60 km/h, while the software analysis was carried out with speeds from 10 km/h to 110 km/h in 10 km/h increments. The experimental test results that an increase in speed resulted in up to 84% reduced stress measured on the pavement for an increase from 10 km/h to 60 km/h. The lower stresses in the pavement at higher speeds was concluded to be due to the viscoelastic behaviour of HMA, which indicates that there will be higher stiffness at higher loading frequencies and therefore less stress is transferred to lower layers. In addition to the change in stress levels with different speeds, the measured compressive strains were found to be lower at higher speeds, indicating greater rutting resistance at higher speeds. The wheel wander was found to be insignificant on the strain due to low truck loading. As for the results of the software analysis, the MEPDG predictions indicated that the 20-year fatigue cracking decreased as a result of increased speeds, however the rate at which the fatigue cracking decreased was less significant for speeds greater than 70 km/hr. The results also showed only a 6% reduction in rutting, which was not consistent with the large difference in compressive strains measured experimentally. For this reason, the authors concluded that the MEPDG predictions did not agree with those trends measured in the field, as well as because all of the MEPDG scenarios failed to meet 90% reliability.

Similar to Al-Khateeb and Ghuzlan mentioned above, Zhang, et al. [172] looked at loading frequency and stress level and how variations of these parameters affected fatigue damage characteristics and the damage accumulation. The IDT with a haversine load at 5, 10, and 25 Hz showed that with constant load rate, a lower loading frequency resulted in faster strain rate, however the final strains at failure were not much different between the different frequencies. The fatigue life of the pavement was found to be increased from greater loading frequency and from reduced loading levels. The authors also looked at the Pavement Condition Index (PCI) of in-situ

pavement sections for comparative analysis. Results showed that the PCI of the faster lane was larger and that there was less fatigue cracking on sections with higher vehicle speeds.

3.3.2 Rest Period and Fatigue

Kim, et al. [173] performed controlled-strain torsional fatigue tests with varying strain to create damage models with a relationship between strain and load repetitions. The objective of the authors' work was to illustrate fatigue and healing characteristics of asphalt. In order to demonstrate the healing characteristics of the asphalt concrete, a two-minute rest period was introduced in addition to continuous loading for one of the strain levels, with ten rest periods throughout the loading period followed by continuous loading until failure. Given two-minute rest periods, the dynamic modulus increased by up to 24% depending on the binder used, which would indicate a longer fatigue life where a rest period is introduced.

Similarly, and Castro and Sanchez [174] compared the fatigue life given testing with and without a rest period. A three-point flexural test with a continuous sinusoidal loading of 0.1 seconds was compared to a sinusoidal loading of 0.1 followed by a rest period of ten times the loading time, or 1 second. The comparison of the two testing scenarios showed that the rest period increased the fatigue life of the pavement by five to ten times that of a pavement tested without rest period.

Souliman, et al. [175] looked at how rest period, among other factors, would impact the fatigue response of asphalt pavement to attempt to validate the endurance limit of the pavement. The endurance limit of asphalt is the value of stress or strain below which no fatigue damage occurs, and this research in particular looked to relate this endurance limit to the healing concept of asphalt, more specifically the effect of rest periods on the healing effect. Beam fatigue tests were performed on asphalt samples with varying binder grades, air voids, binder percentages and temperatures under continuous loading of 10Hz or a 0.1 s loading time followed by a 5 s rest period. Additionally, more tests were carried out to test different rest periods, specifically no rest, 1, 5, and 10 s of rest for asphalt under various parameters mentioned above. For any case that was tested, there was no observed increase in healing with greater than 5 s of rest between loading applications.

Baaj, et al. [176] also looked at healing properties and fatigue characteristics of asphalt given rest time at various temperatures. A tension-compression fatigue test loaded sinusoidally was carried out for different loading times and rest periods. However, instead of using smaller rest periods between loading cycles, which would simulate actual traffic conditions more accurately, the load and rest times were quite large to clearly show the effect of the long rest period on the healing properties. The loading times were 4 to 8 hours with rest periods for a total of 24 hours cycle time. The results of the tests showed that modulus recovery of samples with rest period was greater, and that during the unloaded period the phase angle was found to be decreased, indicating stiffer material. Also, an analysis of the number of load repetitions to failure for the different loading and rest scenarios showed that the rest period increased the number of load repetitions to failure, therefore, increasing the fatigue life, of the samples.

Loading frequency coupled with temperature were the parameters of focus of Zeiada, et al. [177], who measured their effects on asphalt fatigue life using the Viscoelastic Continuum Damage Model (VCED). The dynamic modulus test was carried out with different frequencies tested, specifically, 25, 10, 5, 1, 0.5, 0.1 Hz over a range of temperatures from 10 °C to 54 °C to obtain the moduli for the VECED model, while axial fatigue testing was done to show the difference in haversine and sine loading conditions as well as under the condition of continuous loading or with an intermittent loading and 3 sec rest period. A 3-second rest period was chosen from previous work that showed a rest time greater than 3 seconds would not have any impact on the fatigue

life. The uniaxial test with intermittent sinusoidal loading showed improved fatigue life, however, this effect was less prominent for haversine loading. Moreover, since the intermittent loading with a 3-second rest period had improved fatigue performance it was concluded by the authors that a 3-second rest period was sufficient for the recovery effect to occur in the asphalt.

Li, et al. [178] et al performed Dynamic Shear Rheometer (DSR) tests and measured the crack length, as well as the dissipated pseudo shear stiffness and pseudo shear strain, to look at the healing properties of asphalt binder. The latter two parameters were looked at in order to differentiate the shear modulus due to recovery versus that due to healing, and used to define the healing index. The effect of rest periods, ranging from 5 seconds to 40 minutes, on the healing index of the binder was examined as well. Three different loading times were tested as well, 5 min, 10 min and 20 min. The authors found that with a greater loading time, there is greater cracking in the binder, and furthermore, the healing index is increased due to the recovery of pseudo strain energy following the rest period. For shorter rest periods, it was found that the rate of healing, or recovery, was increased, compared to longer rest periods which had reduced rates of healing.

3.3.3 Speed/Frequency and Permanent Deformation

The objective of the research done by Qi and W.Witzcak [179] was to create permanent deformation models for asphalt that took into consideration loading time and rest time. Cyclic creep tests with square shaped loading waveforms were performed using an MTS hydraulic press. The loading times were 0.1, 10 and 1000s, with rest periods of 0.9, 10, and 1000 seconds for the different cases tested at three stress levels and two temperatures. The results showed that the permanent strain increased with an increase in loading time and decreased with rest time, and that the loading time had a more significant effect on the permanent strain than the rest time.

The following year, Pereira, et al. [180] aimed to validate methodology set by Strategic Highway Research Program (SHRP), specifically, the SHRP A-698 methodology for permanent deformation prediction in asphalt layers, specifically, based on repeated simple shear test at a constant height done on cored samples from low-speed road sections in Portugal. Hilly sections with reduced speeds, as low as 20 km/h, were used to validate the methodology for conditions other than regular higher speed highway sections. Three different load times were tested on the cored samples, 0.1, 0.2 and 0.3 seconds of haversine loading with a 0.6 rest period for the repeated simple shear test. The effect of the loading time on the rate of permanent deformation was shown through the greater permanent strain observed for the longer loaded specimens for all cycles. The model that resulted from the experimental result was not found to be statistically significant to correlate the permanent deformation characteristics based on the in-situ measured rut depths.

Similar to the experiments carried out by Al-Khateeb and Ghuzlan, mentioned before, that looked at the effect of different frequencies, stress levels and temperatures on fatigue performance, Zou, et al. [181] looked at similar parameters, as well as different binder types, and the effect on the rutting resistance of asphalt pavement. The WTT was performed on samples with two binders, at three different temperatures and four different frequencies, namely, 42cycles/min, 35 cycles/min, and 28 cycles/min which is equal to 60km/h, 50km/h, and 40 km/h respectively. The authors denoted a parameter as the dynamic stability, or the number of loads within a 15-minute period that would induce a one-millimeter deformation which was used as performance indicator for rut resistance. The results of the orthogonal experiment as intuitive analysis showed that the asphalt mix type and temperature were more significant in their effects on the rutting than the frequency,

followed by the loading magnitude. Moreover, a lower frequency, increased temperature and increased loading magnitude resulted in greater rutting.

Wang, et al. [182] used finite element analysis to examine single-axle, double-axle, tire pressure, and speed effect on rutting. A reduced speed of 60 km/h was compared to a base scenario of 80 km/h for the same number of load repetitions and the analysis showed that this decrease in speed could result in a 60% increase in rutting for the section. Also, it was determined that dual-axle loads result in greater rutting than single-axle, and a higher tire pressure would result in greater maximum rut depths.

Once again, temperature, load magnitude and load frequency were the parameters of focus for Zhang, et al. [183] as well as moisture, and how they would impact the permanent deformation. A HWTT on samples with different anti-rutting agents under different conditions was carried out for 20 000 passes or until 12.5 mm of deformation in the middle of the sample. The frequencies tested were 44 passes/min, 52 passes/min and 68 passes/min at temperatures varying from 40°C-60°C. At lower frequencies, the rut depth was found to be greater and there was a decreased resistance to deformation. Moreover, an increase in temperature, as well as overloading cause greater rutting, with the effect of moisture found to be more significant than the effect of the overloading.

Khan, et al. [184] used pavement data from in-situ networks as the input data to simulate strains in asphalt under varying speeds using the BISAR program. Four speeds, 30, 50, 70 and 80 km/h were used as input variable to find the horizontal tensile strain at the bottom of the asphalt layer and the vertical compressive strains at the top of the subgrade surface layer, correlated to the fatigue and rutting performances respectively, of the pavement system. The simulation results showed that with increasing speed, there was an increased stiffness of the top layer indicating a longer fatigue life and better performance. Moreover, at higher speeds, there is less accumulated strain, for both the horizontal tensile strain and the vertical compressive strain, indicating that there would a greater number of loading repetitions can be withstood by the material before the end of the fatigue life or rutting life.

The speed effect, coupled with the temperature effect, on in-service asphalt was examined by Chen, et al. [185] to modify the evolution law rutting model. Three different speeds, 3.5 km/h, 4.5 km/h and 5.5 km/h and four temperatures from 30 °C to 60 °C were the varying parameters for the accelerated loading test performed on cut samples from a Chinese expressway at the end of its service life. The results of the testing show that for the lowest speed, there is the greatest permanent deformation. However, the rate of rut depth decrease was more significant for an increase in speed from 4.5 km/h to 5.5 km/h. Also, it was found that there was up to 2.15 times more rutting for temperature increases of up to 15 °C, with this effect more significant at higher temperatures. As a conclusion of their work, the authors presented a modified rutting prediction model based on the rutting under different speeds.

In addition to the speed effect on permanent deformation, Alkaissi and Banyhussan [186] also looked at loading waveform and load durations, utilizing 3D finite element analysis for simulation. Three speeds were chosen to represent slow to fast moving traffic, specifically 40, 60, 80 km/h, along with 200 or 400 ms of loading time and triangular or trapezoidal waveform. The simulation showed that a speed decrease from 80 km/h to 40 km/h resulted in over 55% more permanent deformation. Moreover, this effect was found to be more significant at lower speeds under higher load repetitions. A decrease in load duration was found to increase pavement life, and a trapezoidal waveform was found to better represent loading conditions than a triangular waveform.

Zhuang, et al. [187] aimed to modify the rutting prediction model to include temperature, loading frequency and duration as well as stress level. The results of an MTS test were used for regression analysis and to compare the results of finite element analysis of the new modified model. A sinusoidal rectangular wave at five frequencies from 1 Hz to 10 Hz under loading times from 1000 ms to 5000 ms and two different stress values was tested for three hours of continuous loading at three different temperatures. The results from the experimental testing showed that for a lower loading frequency and greater loading time, there was increase in measured rut depth. Moreover, an increased temperature would have the same effect of greater rutting compared to lower temperatures. The modified rut prediction model was shown to be statistically a good predictor of the rut depth based on the finite element analysis and the MTS test.

3.3.4 Rest Period and Permanent Deformation

In addition to testing various frequencies, load times or speeds, the effect of rest periods has also been looked at with respect to how the rutting will be affected. Nejad, et al. [188] performed uniaxial compression tests with haversine loading and different rest time scenarios to determine how the permanent deformation was impacted. The testing conditions were continuous loading, intermittent loading, where a cumulative rest period of 2.25 times the loading time was applied between each loading, as well as continuous loading with a short or long rest period after a specified number of cycles. The short rest periods after continuous loading repetitions started with 60 seconds, with the rest period doubling after every 100 seconds of loading, and for the longer rest period after 2000 seconds continuous loading was constant at 4500 seconds. The observations made were that for samples under intermittent loading had less strain for the same number of cycles than under continuous loading, however, the rest period was determined to be too short for viscoelastic recovery of strain. Conversely, for continuously loaded specimens with short increasing rest period the resistance to permanent deformation in the specimen is reduced, with the authors citing the reason for this as due to the hardening-relaxation principle of asphalt. The hardening-relaxation effect was found to be reduced, with an increase in the recovery effect, with longer rest periods after loading cycles.

Taherkhani, et al. [189] looked at how different frequencies, rest periods, temperatures, and moisture conditions would affect the resilient modulus of a polymer modified asphalt. The resilient modulus of asphalt is similar to the elastic modulus, with the difference being that it is a ratio of stress to recoverable strain, and it should be noted that an increase in resilient modulus results in better rutting performance. From the tests carried out, a model was developed for the resilient modulus variation based on temperature and loading time, or frequency. Dynamic creep tests at 0.5, 1, 5, and 10 Hz, with a triangular loading and loading times of 0.5, 1, 0.05 and 0.01 seconds, as well as set periods of 1.5, 0.9, 0.15 and 0.09 seconds at various temperatures and under dry or saturated conditions were performed on asphalts samples with and without polymer modified binders. The results showed that the polymer modified binders allowed more repetitions prior to failure, and it was found that the loading time was a significant factor affecting the resilient modulus, with increased the modulus due to higher frequencies or lower loading times. For instance, for an increase from 0.5 Hz to 10 Hz, there was an increase of eight times in the resilient modulus, indicating better rutting performance. As a result of decreased temperature, the resilient modulus increased, and the rate of frequency effect decreased.

The rest period effect on asphalt concrete has been mentioned, however, this effect was also looked at by Alnedawi, et al. [190], but on the unbound granular materials, specifically the permanent deformation of the base and subbase, not the asphalt course. They performed triaxial tests with trapezoidal repeated loading with 1 second load followed by 2 seconds rest or a 3-

second total load without any rest period, and compared the results to finite element analysis. The triaxial test showed that there was a lower resilient modulus when a rest period was included. Also, the finite element analysis showed an increase in rutting given rest period, however, the difference in deformation was found to be very small, on the order of 10^{-5} . The rest period loading scenario showed greater maximum elastic strains, however, once again the difference was found to be minimal.

The experiments of Motevalizadeh, et al. [191] were performed to create an experimental model to predict the permanent deformation of asphalt concrete. The model was based on results from the repeated load permanent deformation test (RPLD) under haversine loading with different loading times, rest periods, as well as two stress level and various temperatures. The loading times were 100, 500 and 1000 ms with rest period to load time ratios of 1/9 and 1/1 for each loading time. An increase in rest period and load duration resulted in greater deformation, and an increase of permanent deformation was observed for an increase in both rest and load duration separately. The load duration, followed by stress magnitude and finally, rest period duration were the factors in order of significance of their effect on the permanent deformation.

Alrajhi, et al. [192] performed many different types of tests to experimentally determine the rest period of effect on permanent deformations of asphalt concrete made with and without modified binders. First, flow number experiments at three stress levels and temperatures were carried out for determination of complex modulus and linear viscoelastic characteristics. The flow number was reduced with an increase in rest period, with a 70% decrease in flow number for only a 2.5 second increase in the rest period. A decreasing flow number indicates worse rutting performance. The RPLD test was performed with uniaxial haversine loading of 0.05 and 0.1 seconds of loading followed by 2.5, 0.57 and 0.18 seconds of rest period. These rest periods were chosen to correspond to the safe sight distance of trucks traveling at 96 km/h, the headway of trucks in a platoon, and the expected headway for a platoon of ATs respectively. Larger rest periods of 5 seconds and 10 seconds were also used to show the significance of the rest period effect, and three different test temperatures were used as well. The results of this experiment showed that permanent deformation increased when rest period increased and that the rest period effect is reduced after greater than 2.5 seconds of rest time. Next, the authors used the same rest periods for the MSCR test, which showed converse results to those of the RPLD. An increasing rest period was found to have greater percentage of recovery and therefore a greater permanent deformation resistance for small number of load repetitions. Finally, complex modulus tests at six different frequencies from 25 Hz to 0.1 Hz and a 60 seconds rest period between different frequencies was carried out at five different temperatures to find the complex modulus master curves.

Table 5 and Table 6 present summaries of the previous research described, specifically the effect of frequency, speed or loading time, as the only factor or along with rest period or spacing, and their effects on fatigue and rutting of asphalt pavement respectively. It can be seen in Table 5, that there are many works focused on the fatigue life of asphalt concrete, and how it is impacted by both varied speeds or frequencies as well as rest periods. Overall, the results of the previous work show that an increase in speed or frequency was found to increase the fatigue life, through reduced strain accumulation rate or reduced stress on the asphalt. The rest period effect on fatigue life is less researched than the speed or frequency effect however. For those studies that did consider rest periods or spacing, there is a focus on the effect of a single rest period versus no rest period [174, 177], or with long term rest periods [176] which are not very representative of real-life situations. More research should be done on shorter, more realistic rest periods on the effect of the fatigue life.

Table 5: Summary of Fatigue Related Research

Authors, year	Speed/ Frequency	Rest Period/Spacing	Results
Kim et al., 2003		X	Rest Period = ↑ Fatigue Life
Jiang-Miao et al., 2005	X		↑Frequency NEGLIGIBLE on Fatigue Life
Castro, Sanchez, 2006		X	Rest Period ↑ Fatigue Life
Al-Khateeb, Ghuzlan, 2013	X		↑Speed = ↑Fatigue Life
Souliman et al., 2013		X	
Shafiee et al., 2015	X		↑Speed = ↓ stress
Baaj, et al., 2017		X	no further healing with > 5s rest
Abdelaziz et al., 2018	X	X	rest period = ↑ fatigue life
Zhang et al., Lu, 2019	X		↓Frequency = ↑ strain rate
Li et al., 2019	X	X	↑Rest Period = ↑healing rate ↑loading times = ↑ damage

Table 6: Summary of Rutting Related Research

Author, year	Speed/ Frequency	Rest Period/Spacing	Results
Qi, Witczak, 1998	X	X	↑ loading time= ↑strain ↓ rest period = ↑ strain
Pereira et al., 1999	X		
Nejad, et al., 2015		X	rest period = ↓ strain
Wang et al., 2017	X		↓ speed = ↑rutting
Zou, Xu, Wu, 2017	X		↓ speed = ↑rutting
Taherkhani et al., 2017	X		↓loading times = ↓rutting
Alnedawi et al., 2018		X	rest period= ↑deformation, Mr
Motevalizadeh, et al., 2018		X	↑rest period =↑deformation ↑loading time =↑deformation
Zhang et al., 2020	X		↓ speed = ↑rutting
Alrajhi et al., 2022	X	X	↑rest period =↑deformation
Chen et al., 2022	X		↑speed =↓ rutting
Khan et al., 2022	X		↑speed =↑ rutting life
Alkaissi, Banyhussan, 2023	X		↓ speed = ↑rut depth
Zhuang et al., 2023	X		↓ frequency = ↑ rut depth

The summary of rutting-related research is presented in Table 6. Much of the rutting related research has been focused on the speed or frequency effect, with very few considering the rest period effect the results presented in the table above show that at reduced speeds or frequencies, and as well at greater loading times, the deformation or rate of rutting is increased. Chen et al. [147] had similar findings with regards to greater speeds reducing rutting. These results are reasonable, given that more damage is done to the pavement under increased loading time, which is a product of reduced speed or frequency, due to the viscoelastic properties of the material as mentioned in the Background section of this report. As for those studies that did consider rest period on rutting, the research is not too thorough, for instance, Alnedawi, et al. [190] looked at rutting without or with only one rest period, and Motevalizadeh, et al. [191] had large loading times and only two rest ratios, which once again, are not very representative of actual traffic loading conditions. Regarding the rest period effect, Qi and W.Witzcak [179] found that with increasing rest period, there would be decreased permanent strain for greater rest periods, whereas Motevalizadeh, et al. [191] and Alrajhi, et al. [192] concluded that a greater rest period would result in more rutting or permanent strain. From the previous section specific to AVs, the optimized platoon configurations from Gungor, et al. [151] had headways closer to lower bounds, while Al-Qadi, et al. [155] and Ramakrishnan, et al. [166] found that rest periods less than five feet (1.5 m) and ten feet (3m) respectively, was negligible to pavement damage. The results of Ramakrishnan et al. MSCR test showed that an increased rest period improved strain recovery, while RPLD results indicated greater deformation due to increased rest periods. The lack of studies related to the rest period effect on rutting is a gap that will be addressed in this work.

The above literature review has examined previous research related to the impact of trucking on traffic and safety, the impact of AVs and ATs on traffic, safety, the implications of platooning, and the impact on asphalt pavement that wheel wander, vehicle speed and spacing will have as well as an in depth look at pavement performance given speed and spacing, or frequency and rest period. AVs will have impacts on the environment, traffic operations, safety and infrastructure, and it is therefore important that all these impacts be carefully considered before these vehicles are deployed. Much of the research presented above only considers one or some of these impacts when discussing AVs, thus, this work focuses on determining optimal autonomous truck platoon configurations and penetration rates considering the effect on asphalt pavement, traffic operations and emissions.

4. METHODOLOGY

To examine the impact that ATs platoons will have on asphalt pavement, traffic operations and emissions, two methodologies are considered in this research. Figure 20 illustrates a summary of the methodologies for the asphalt testing and VISSIM traffic simulation. In the first part, asphalt samples are tested under compressive loading meant to simulate ATs platoons travelling at different speed and headway combinations and how this will affect asphalt permanent deformation or rutting. The optimum headway and speed combination from the asphalt testing section is used for model calibration for ATs platoons during the traffic simulation, to find the effect of this platoon configuration on total travel time and network delay as well as on GHG emissions, specifically CO₂, NO_x, PM₁₀ and PM_{2.5}. Different traffic volumes and penetration rates of ATs are considered for simulation as well.

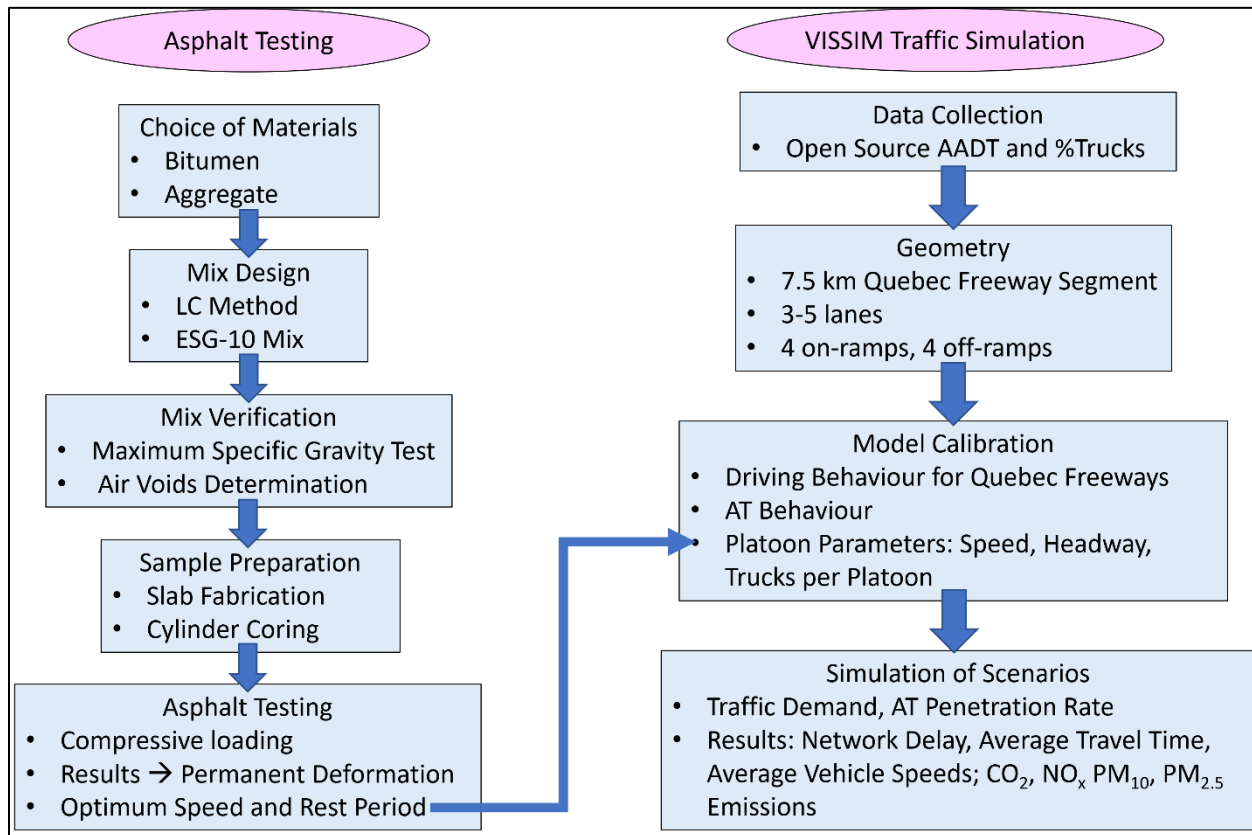


Figure 20: Methodology Summary

4.1 Pavement

4.1.1 Asphalt Binder

The bitumen used in this asphalt mixture is a PG 64E-28 binder, indicating, according to the SuperPave Performance Grade system, that the binder is sufficient for temperatures of up to 64°C and as low as -28°C. Moreover, the binder is polymer modified for extreme traffic conditions (ESALs > 30 million) and is the binder that is used in the southern part of Quebec, as well as in the major cities in the province. Figure 53 Appendix B illustrates the climactic zones of Quebec, as well as the accompanying table that shows which binder is used in each zone. The mix was prepared according to an ESG-10 surface layer mix type, which is a standard surface layer mix used in Quebec [69]. An ESG-10 mix indicates a dense-graded mix recommended over others for engineering structures, since it is a less permeable mix [193]. Binder properties, including viscosities indicating mixing and compaction temperatures, are known from previous laboratory testing. Table 7 shows the properties of the binder, including specific gravity and mixing and compaction temperatures. The amount of binder used in the mixture is determined from the maximum specific gravity test described in Section 4.1.3.

Table 7: Binder Properties

Property	Value
Binder	PG 64E-28
Specific Gravity, G_b	1.02
Proportion, P_b (%)	5.64
Mixing Temperature, T_{mix} (°C)	168
Compaction Temperature, $T_{compact}$ (°C)	160

4.1.2 Aggregate

The aggregate used in the asphalt mix is local crushed stone, with size ranges of 0-5 mm and 5-10 mm for fine and coarse aggregate respectively, without the use of additional filler material. The coarse and fine aggregate were obtained from a local quarry with proper sampling techniques used, in order to maintain the gradation of the aggregate. Ten separate samples from a flat level plane of aggregate were taken, then recombined and split to maintain the gradation. Aggregates were split according to LC 26-015 [194] using the mechanical separation method. The gradation of the coarse and fine aggregates was checked through mechanical sieving of both aggregate types. Moreover, two separate samples of both fine and coarse aggregate were sieved for a comparison of the similarity of the gradation between the samples, and both were found to be within the limits set by the code. The proportion of the coarse and fine aggregates of the total aggregate amount was determined through the adjustment of previous asphalt mix designs for ESG-10 asphalt mix with PG 64E-28 binder to respect the gradation of this type of mix. The resulting mix design consists of equal amounts, or a 50/50 ratio, of fine and coarse aggregates as part of the total aggregate amount.

Table 8 shows the properties of the coarse and fine aggregate, including aggregate type, specific gravity, absorption and percentage of total aggregate mass for each aggregate type. The combined specific gravity is shown as well, found from the individual specific gravities and the proportion of the total aggregate mass.

Table 8: Aggregate Properties

	Coarse Aggregate	Fine Aggregate
Type	St. Phillips 5-10 mm	St. Phillips 0-5mm TV
Proportion	50%	50%
Bulk Specific Gravity	2.716	2.677
Absorption (%)	1.21%	0.69%
Combined Specific Gravity	2.696	

The coarse aggregate was first sieved through coarse sieves for ten minutes, followed by another ten minutes of fine sieving for the material that passed through the 5 mm sieve. The mass retained on all sieve sizes from 10 mm to 80 μm was measured. Prior to sieving the fine aggregate, the dry mass of the material was weighed, and then washed to remove particles that are finer than 80 μm . The washed sample was then dried at 110 °C and weighed again to find the mass lost. Next, the fine aggregate was sieved for ten minutes using sieves from 5 mm to 80 μm , and then the mass of the retained material on each sieve size was measured. The mass passing the 80 μm sieve was added to the washed filler amount to obtain the amount of total filler as part of the aggregate sample gradation. The sieving procedure described before, and corresponding analysis was carried out in accordance with LC 21-040 [195] to verify that the gradation of the coarse and fine aggregate would respect the limits set by the standard.

Table 30 in Appendix C shows the initial masses for the two samples of coarse aggregate, as well as the cumulative mass after sieving. The percent difference in the masses should be less than 0.3 as set by the standard. Similarly, Table 31 shows the initial mass of the fine aggregates samples prior to and after washing. The difference in the masses before and after washing is shown as well, representing the mass of filler, which is material that is less than 80 μm , as well as the total mass of filler once the sieving was completed. Table 32 and Table 33 show the cumulative mass retained, the cumulative percent of material passing each sieve size and the percent of material passing each size as well as the average percent passing of the two samples for both coarse and fine aggregate respectively. The precision for both the fine and coarse aggregates between the two samples respects the limits set by the standard.

Next, in order to verify the gradation, the combined percent passing of both aggregate types, based on aggregate proportion in the total mix, the cumulative percent passing per sieve is plotted against the size of the sieve openings raised to the 0.45th power. The combined percent passing is shown in Table 9 and the plot is shown in Figure 21. This figure also illustrates the maximum density line, which the percent passing should fall below, the specification, where the percent passing should fall between, and finally, the restricted zone, where the percent passing should not fall at all for proper mix design as indicated by LC 21-040 [195]. Moreover, in Table 34 and Table 35 in Appendix C, the specification values and restriction zones for the aggregate gradation are shown as well, with the restriction zone indicating a range of values where the combined percent passing should not fall. The specification values indicate a zone that the combined gradation should fall between for each sieve size.

Table 9: Aggregate Gradation Percent Passing

Sieve Size (mm)	10	5	2.5	1.25	0.613	0.315	0.16	0.08
Sieve Size $^{\wedge 0.45}$	2.82	2.06	1.51	1.11	0.80	0.59	0.44	0.32
5-10 mm % Passing	97.21	8.36	2.42	1.75	1.46	1.34	1.26	1.08
0-5 mm % Passing	100	96.96	66.12	44.92	32.35	24.64	19.59	15.83
Combined % Passing	98.61	52.66	34.27	23.33	16.91	12.99	10.43	8.45
Limiting Values MAX	100	65.00	46.10	30.7-36.7	22.8-26.8	18.10	--	--

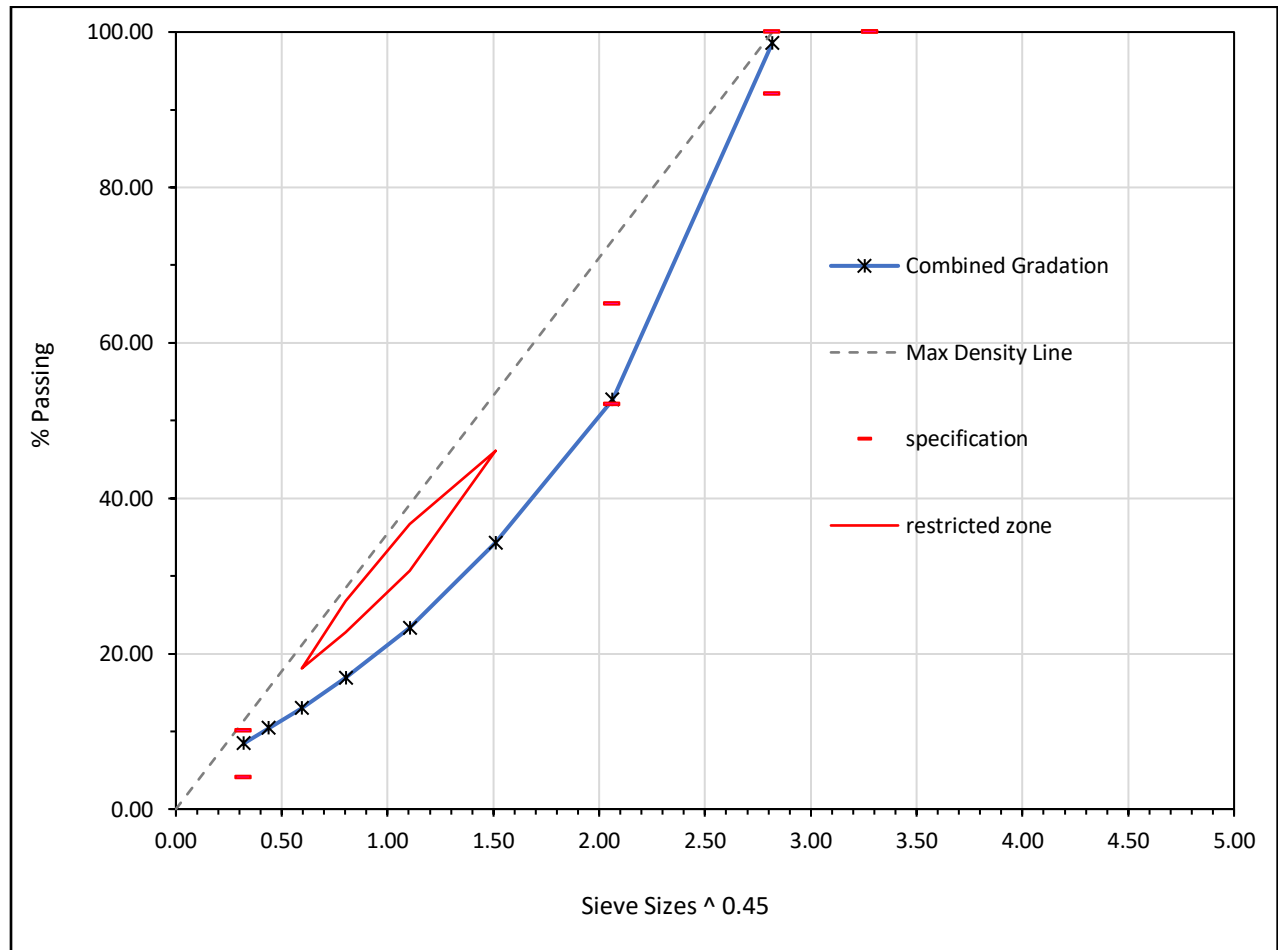


Figure 21: Combined Aggregate Gradation

4.1.3 Mix Design and Verification

The design of the asphalt mixture was done according to the *Laboratoire des Chaussees* (LC) method of design. This method is the standard for asphalt mix design in Quebec, with the method itself a mixture of SuperPave method of design, as well as the French LCPC design method. A surface layer ESG-10 asphalt mix with PG 64E-28 grade binder and 0-5 mm and 5-10 mm aggregate was designed and tested according to these standards.

The LC 26-004 [196] standard is the specific method followed for mix design. In this method, an initial binder content is estimated given the specific gravities of the binder and aggregate (G_b and G_s , respectively), as well as the effective asphalt binder volume (V_{be}) and estimated percent binder absorption (P_{ba}). The specific gravities are known as part of material properties, and the effective binder content for an ESG-10 mix is 12.2%. The binder absorption is initially assumed to be half of the total aggregate water absorption, or half of the proportional water absorption of the combined aggregate and using this estimated value the estimated binder content is found. The steps, as well as sample calculations, can be found in Appendix D.

Next, the maximum specific gravity (G_{mm}) test is performed for binder contents at the estimated binder content and at 0.3% greater, according to the LC 26-045 [197] standard. The maximum specific gravity, or the specific gravity of the mix not including air voids, is found through by finding the specific gravity of two batches of the loose, noncompacted mix. Volumetric calculations are then carried out in order to find the effective binder volume, and linear interpolation is used to find the binder content at which the V_{be} is equal to 12.2%. This binder content is then verified through one final repetition of the maximum specific gravity test. After the G_{mm} tests, the binder content that resulted in 12.2% effective binder volume was 5.64%, with a maximum specific gravity of 2.508. Table 10 illustrates the final verification of G_{mm} for a V_{be} value of 12.2%. Previous trials of the G_{mm} test to obtain this value, as well as calculations and linear interpolations, are shown in Appendix D.

Table 10: G_{mm} Test Verification

	Sample A	Sample B
P_b (%)	5.64	5.64
M_{dry} , (g)	1157.4	1224.2
$M_{equipment}$ (g)	5163	5156.5
$M_{equip+sample}$ (g)	5858.5	5893
$T^{\circ}C$	24.7	24.7
G_w	997.1	997.1
G_{mm}	2.506	2.510
$G_{mm_average}$	2.508	

The asphalt mixture was designed and compacted in accordance with LC 26-003 [198] with the SuperPave Gyratory Compactor (SGC) to verify that air voids are within 4% - 7% at 80 repetitions and within limits at different number of compactions as well as to find the Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA). Two separate trials with two samples each were designed and compacted using the SGC. The results, as well as the limiting values are shown in Appendix E. All asphalt samples were split according to LC 26-010 [199], to keep the mix gradation between batches. Table 11 shows the relevant properties of the final asphalt mix design, including specific gravities, absorptions, voids and effective volumes.

Table 11: Asphalt Mix Properties

Property	G_{mm}	G_{sb}	P_{bi} (%)	G_{se}	P_{ba} (%)	M_{be}	V_{be} (%)	G_{mb}	V_a (N=10)	V_a (N=80)	V_a (N=200)	VMA	VFA
Value	2.508	2.696	5.64	2.748	0.71%	4.972	12.23	2.346	16.485	6.656	3.280	15.08	78.25

4.1.4 Sample Preparation

Once the mix design was verified and within all limits set by the standards, large asphalt slabs, with dimensions of 600 mm x 400 mm x 125 mm, were fabricated and compacted according to the LC 26-400 [200] standard. An example of a slab being compacted is shown in Figure 22 and the final constructed asphalt slab is shown in Figure 23. First, the aggregates were weighed and split into two batches, using the proper splitting methods as in LC 21-015 [201], in order to respect the limit of the mixer, and the aggregate and binder were heated at the mixing temperature of 168 °C for the time required by the standards. Once the large slab had been manufactured and compacted, the slab was left for a two-week curing period. Following the curing period, each slab was split into three sections, then cored, to obtain multiple 150mm x 75 mmØ cylinders needed for testing. Figure 24 illustrates the slab cutting and coring process. First, the extremities of the slab were cut off, to ensure proper and consistent air voids throughout, since the air voids at the extremities of the slab will be higher than in the middle, due to the compaction process. About 60 mm from each side were sliced off, followed by the remaining slab being split into three even sections of approximately 160 mm. All slices of the slab are in the direction normal to the wheel path of the compactor (shown in blue in Figure 24). Next, four cylinders of diameter 75mm were then cored from these sections, for a total of twelve cylinders per slab. The cylinders were cored closer to the middle and top of each sectioned slab piece, for more uniform and representative air voids. Finally, the ends of each cylinder were sanded to ensure straight, flat edges. Figure 25 shows an example of a cored and sanded cylinder with the appropriate dimensions.

The bulk specific gravity, G_{mb} , of each cylinder was determined using the methods described in LC 26-040 [202]. The dry, submerged and saturated surface dry weights of each cylinder were found and used to calculate the specific gravity, as well as with the dry weight and the volume of each cylinder. The air voids (P_a), VMA, and the effective volume of asphalt (V_{be}) were calculated, with a target V_{be} of 12.2% and to ensure precision among samples. Moreover, the heights and diameters of each cylinder were found for further volumetric calculations, using the average of three measurements around the circumference of the cylinder. Four total slabs were manufactured, and 45 total cylinders were cored, measured and tested. The properties of each slab mix as well as the individual cylinders and their bulk specific gravity results can be found in Tables 41 to Table 52 in Appendix F.



Figure 22: Asphalt Slab Compaction

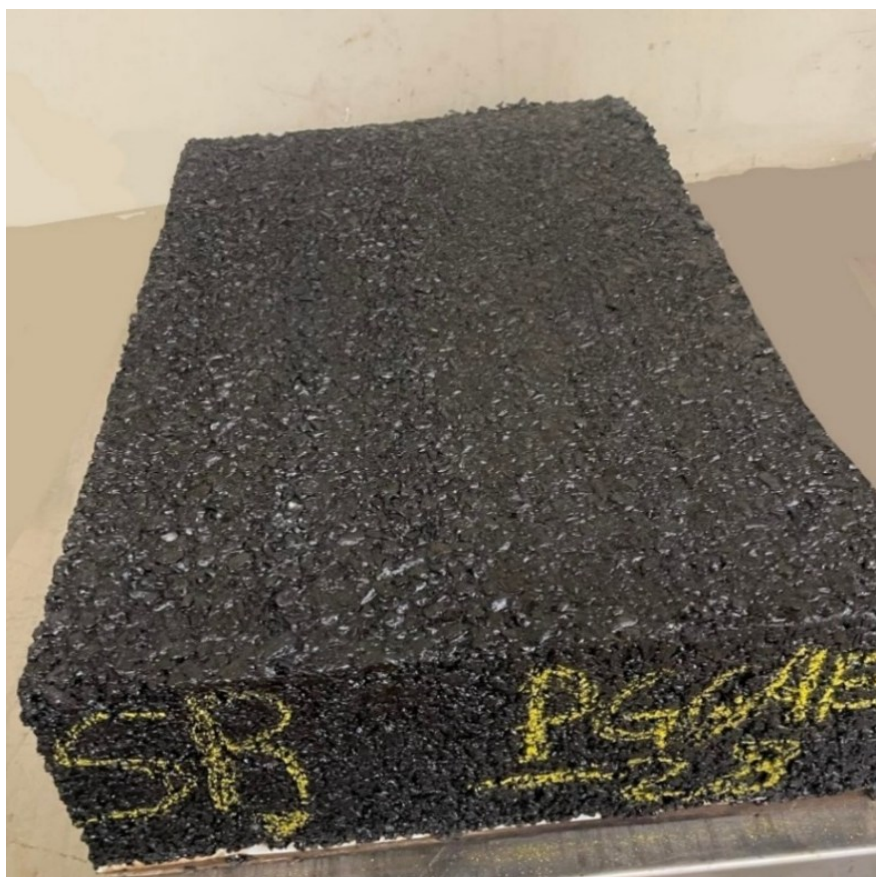


Figure 23: Constructed Asphalt Slab

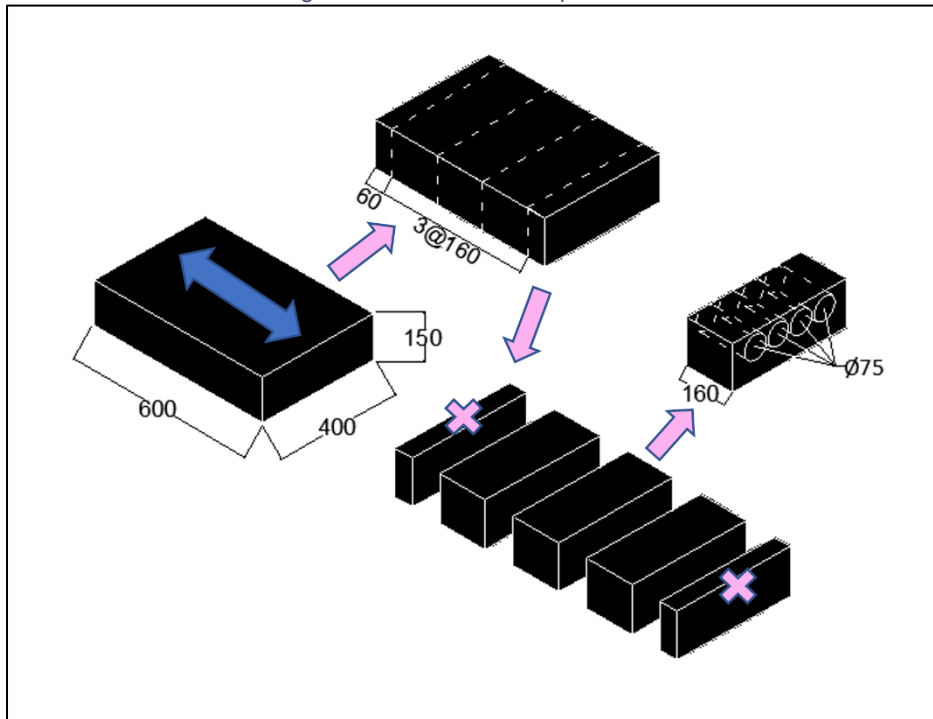


Figure 24: Slab Slicing and Coring

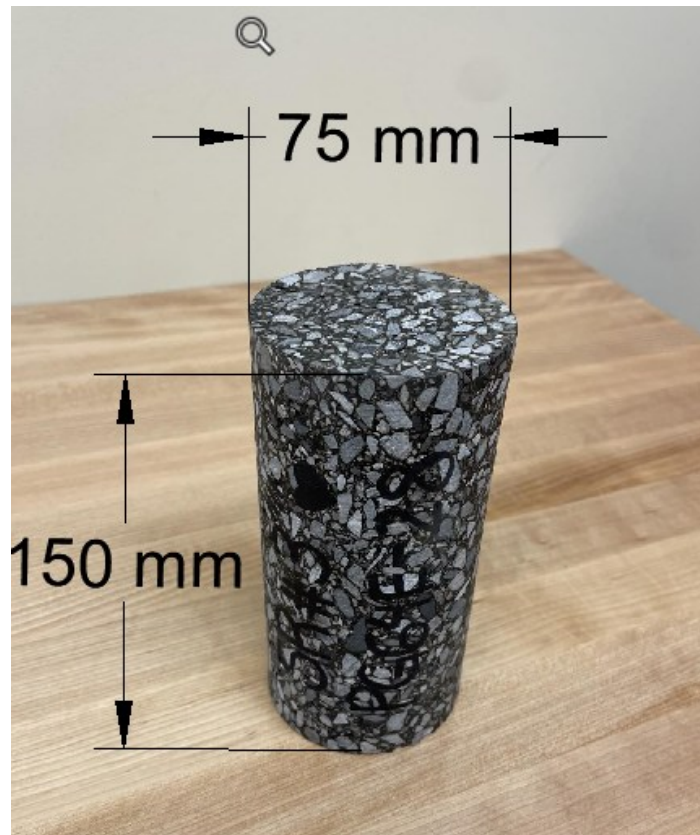


Figure 25: Cored Cylinder with Dimensions

4.1.5 Test Parameters

The MTS 810 testing system is a hydraulic press that can apply a variety of different loading types and control modes, under different conditions such as variable temperatures, frequencies and rest period for materials testing. To simulate and evaluate the effect that AVs will have on asphalt systems, the asphalt is tested under uniaxial compressive haversine loading. This loading type was chosen as it is representative of dynamic traffic loading, specifically, since it most closely resembles actual truck traffic loads. Moreover, the haversine wave form was chosen such that the sample would be only under compression, as this is the main loading that influences permanent strain. The temperature and applied axial force were kept constants for all tests, with a temperature of 50 °C and a load of 1.75 kN, while the frequency and rest periods were varied. Two samples for each frequency and rest period combination were prepared and tested for 30 000 loading-unloading cycles each.

Figure 26 illustrates the loading, unloading and rest periods for the testing cycles; 10 Hz and a rest period of 0.5 s is shown in the figure. For the purposes of this research, one cycle will be defined as a loading, unloading and rest period, while the term solicitation will be used to refer to just a loading and unloading period. During the loading period the sample is loaded with increasing force until the peak of around 1.75 kN is reached, after which the sample is still under load, but the force is decreasing back to 0. For the rest period, the force applied on the sample is 0. The rest period represents the headway between successive vehicles on the road, assuming vehicles travelling at a constant rate. From an asphalt point of view, the rest period during which the pavement is unloaded, provides a chance for the viscoelastic material to regain its strength that was lost during the loading period. An optimum rest period would allow for strength recovery of the material as shown. Figure 27 illustrates this point; the peak cycle strains, as well as the recovered and permanent strains are shown for an example cycle under 10 Hz and 0.5 second rest time.

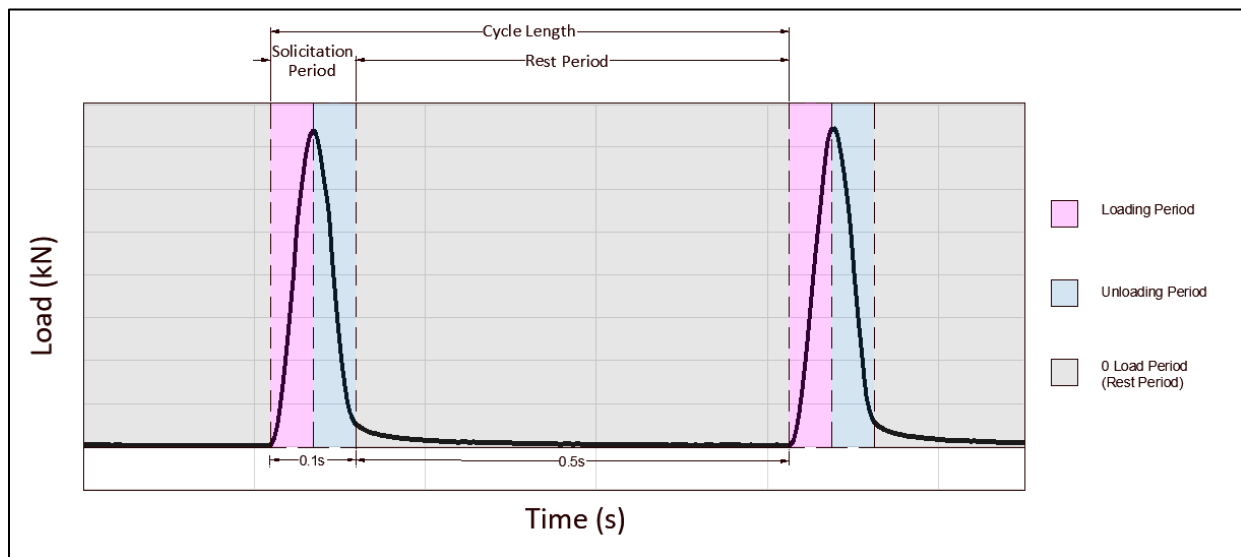


Figure 26: Haversine Loading Force-Time Diagram: 10 Hz, 0.5 s Rest

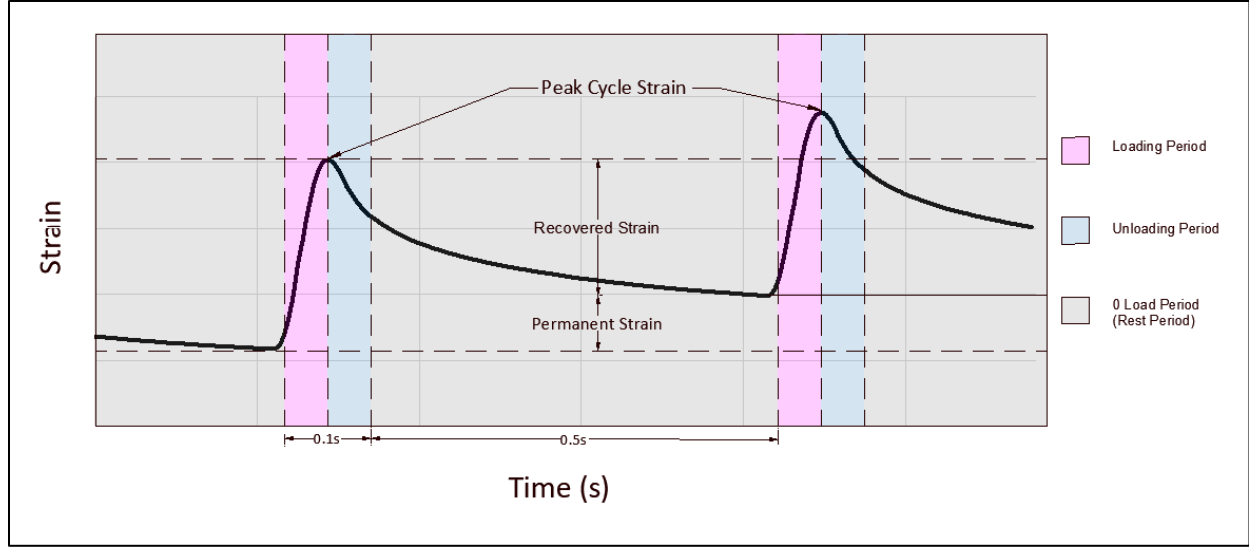


Figure 27: Strain-Time Diagram: 10 Hz, 0.5 s Rest

For the purposes of this research, the temperature was maintained constant for all tests at 50°C and the applied axial force was 1.75 kN. The applied loading of 1.75 kN corresponds to a stress level of approximately 0.4 MPA considering a 75 diameter for the stress formula given in Equation 4. This stress was chosen based on the work done by Nejad, et al. [188], who also examined the rest period effect on permanent deformation with a similar test. In order to determine the optimum ATs platoon configuration in terms of speed and headway, those parameters are correlated to pavement through the frequency of loading time and the rest period between successive loading applications. Equation 5, derived by Primusz and Tóth [203], is used to obtain the frequency, f (Hz), of loading corresponding to a given speed, V (km/h), based on stress wavelengths and wheel contact area. Based on this equation, and to compare speeds of 80 km/h and 100km/h, frequencies of 8 Hz and 10 Hz are used for testing.

$$\sigma = \frac{\pi * D^2}{4} \quad \text{Equation 4}$$

$$f \approx 0.1V \quad \text{Equation 5}$$

$$f = \frac{1}{t} \quad \text{Equation 6}$$

The frequency is inversely related to the loading time [203], specifically the amount of time that the vehicle tire is in contact with the pavement system at any given point, and is shown in Equation 6, resulting in loading times of 0.125 s and 0.1 s for frequencies of 8 Hz and 10 Hz respectively. Each frequency was tested along with four different rest periods, namely, 0.5 s, 1.2 sand 1.5 s, to simulate the headways of the trucks in a platoon. This configuration represents vehicles continuously travelling at a constant speed and at constant headways from one another during congested traffic conditions. Table 12 summarizes the testing parameters used for these experiments, while Table 13 illustrates the different frequency and rest period combinations, as well as the total cycle time for each combination.

Table 12: Pavement Test Parameters

Test Parameters	Values
Force (kN)	1.75
Temperature (°C)	50
# Cycles	30 000
Frequency (Hz)	10
	8
	0.5
Rest Period (s)	1.2
	1.5

Table 13: Test Variables and Cycle Times

Frequency (Hz)	Loading Time (s)	Rest Period (s)	Total Cycle Time (s)
10	0.1	0.5	0.6
10	0.1	1.2	1.3
10	0.1	1.5	1.6
8	0.125	0.5	0.625
8	0.125	1.2	1.325
8	0.125	1.5	1.625

4.1.6 Testing Procedure and Data Acquisition

Prior to testing, a cored cylindrical sample is glued on each circular face to metal plates necessary for the connection with the MTS press system. The samples are glued with epoxy and set in place for a minimum of four hours for the epoxy to set. The gluing and setting configuration of the concrete cylinders is shown in Figure 28, and the final cylinder sample glued with metal plates, ready for testing is shown in Figure 29.



Figure 28: Cylindrical Sample Epoxy Gluing



Figure 29: Glued Sample Prepared for Testing

Following the four-hour setting time, the sample is then prepared to be placed in the MTS press. First, two rubber bands are placed around the sample, one in the middle and one closer to the top, to hold the temperature gauge and the extensometers respectively. Two 100-mm long extensometers are held in place, evenly spaced, by the upper rubber band and secured with two equal springs at the top and bottom of the extensometers. Following the initial calibration of the testing machine and process, two extensometers with a deflection range of 10 mm were used for the deflection measurements. Figure 30 shows a cylindrical sample with elastic bands and springs holding in place two - 10 mm extensometers. The sample is then screwed into the testing apparatus, with care taken not induce excess force during this process. Finally, the temperature gauge is placed under the middle rubber band to measure and record the temperature of the sample throughout the test. The testing chamber is then sealed, and the testing procedure can begin.



Figure 30: Cylinder Testing Set-up

The testing procedure begins with heating the sample until it reaches 50°C throughout. Standard lab practice is to allow for at least four hours of heating; thus, 4.5 hours of heating time was programmed for each sample. Following the heating phases, the testing begins. The testing phase consists of 30 000 cycles, with one cycle being the loading-and-unloading period plus the rest period. The loading and unloading period each consist of half of the total loading time, as will be referred to throughout this text.

During the heating period, the data acquisition rate is every 60 seconds, while for the testing phase, the acquisition rate is every 0.05 seconds. A gap acquisition, used in order to minimize the number of data points, is as follows: for the first 100 cycles, all cycles are recorded; for the 100 to 1000th cycles, 4 out of every 30 cycles is recorded; for the 1000th to the 10 000th cycles, 4 out of every 60; for 10 000 to 30 000, 4 out of every 100 cycles. The first one hundred cycles are all recorded since this is where the greatest rate of change of results occurs and the acquisition rate of 0.05 seconds was chosen to get an accurate representation of the data and the pavement response at small intervals. Each frequency and rest period combination were tested at least twice, or until the difference in final strain values was within 5% of the previous iteration.

4.2 Traffic Simulation

4.2.1 Simulation Description

In order to model the traffic and environmental impacts that ATs platoons will have on Quebec freeway traffic, microsimulation using PTV VISSIM® is carried out. VISSIM is a tool used for microsimulation, meaning that vehicles are considered and simulated individually, and is based on driving behaviour such as car following and lane changing behaviour. Autonomous driving or drivers assist features are also available, as well as the possibility of vehicle platooning, making VISSIM a very useful tool for the simulation of AVs and ATs platoons [204]. As this simulation tool is stochastic, as opposed to deterministic, there is an increased accuracy due to the randomness in behaviour [205]. Finally, VISSIM provides emissions and fuel consumption information for different vehicle types and is used to compare the difference in emissions between different platooning scenarios.

4.2.2 Model/Network Description

A 7.5 km segment of *Autoroute 15* or Highway 15 in Quebec, with traffic in the northwest direction, was chosen for simulation as part of this research, to determine the effect of ATs platoons on highway traffic flow. Autoroute 15 is a major highway in southern Quebec, and one of the main connections to Laval and Montreal from the towns north-west of the city. The selected highway section for this research an approximately 7.5km stretch, off the bridge from Laval to Exit 25, with a total of five-on-ramps and five-off-ramps throughout the section. Neighbouring towns of Boisbriand, Sainte-Thérèse and Blainville are adjacent to the highway, with a combined population of about 100 000 people [206-208], many of whom travel to Laval or Montreal for work or leisure. The bridge connecting Laval and the mainland has an AADT of 145 000, based on summertime travel values [209]. Figure 31 shows the highway section (the highway within the red designated area) used for simulation, as well as the three largest neighbouring towns. Along the highway in the neighbouring towns are residential areas, along with commercial, industrial and manufacturing sectors. Moreover, the large shopping center *Faubourg Boisbriand*, is located just off the highway at Boisbriand, along with a large wholesale shopping center nearby, as well as various manufacturers and transportation and logistics companies.



Figure 31: Autoroute 15 (Source: Google Earth®)

This highway section was chosen for analysis due to the large amount of passenger car traffic, as well as presence of trucks and HVs, with more stable and consistent driving and traffic patterns than in the cities of Laval or Montreal, while still having the influence of being located near the two large cities. As previously mentioned, the southern part of Quebec is subject to heavy truck traffic, and this highway is no exception. Many industrial, commercial and manufacturing hubs are in this area, along with transportation and logistics companies, providing transport of many goods from nearby distribution centers to shopping centers and wholesale centers in the smaller towns, as well as in the larger cities of Montreal and Laval. Given the large amount of commercial and industrial business in the area, as well as the presence of logistics and shipping companies, this highway section would be an excellent candidate for the hub-to-hub case of ATs deployment. ATs platoons would be able to travel from the manufacturers or commercial locales to distribution centers, both for the smaller adjacent towns, and also for the larger cities nearby. In addition to the large presence of commercial and industrial areas, this highway segment is also located near the MVTC in Blainville, Quebec, described as Canada's largest and most advanced automotive research and test center' [210], where current autonomous vehicle and platooning tests are being performed [211]. Figure 32 illustrates the location of the MVTC (shown within the orange boundary) with respect to the highway segment under consideration (within the red boundary).

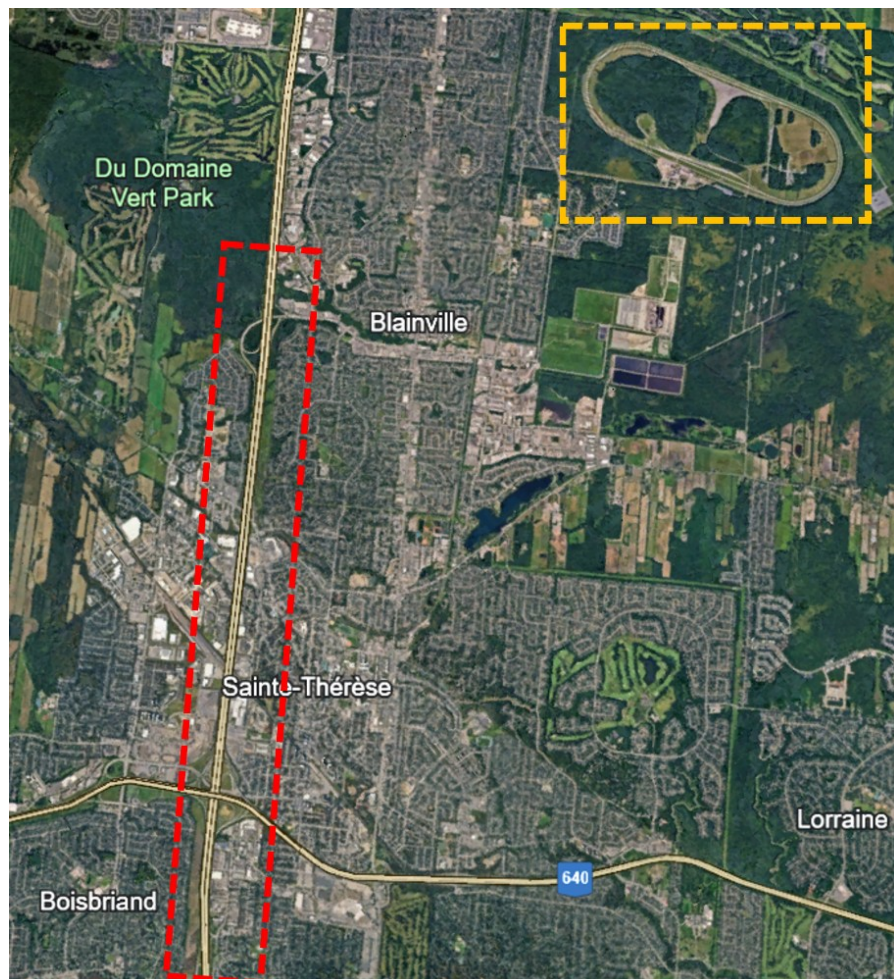


Figure 32: MVTC Location Off Highway 15, Blainville, Qc (Source: Google Earth®)

Moreover, in recent years there has been an increased rate of pavement deformation along Autoroute 15. Figure 33 illustrates a section of the highway in May 2021 and Figure 34 shows the same highway section in June 2024. The dark wheel paths present in the latter image show the increased deformation, or rutting, along this route, in just three years' time, showing significant road deterioration during that period. This may be due, in part, to heavy truck traffic (as mentioned before, due to the commercial and industrial area surrounding the highway). As such, this is another reason as to why this roadway was chosen; the implementation of ATs platoons, especially in a hub-to-hub scenario, could have drastic impacts on pavement systems which already may be under excess stress. Studying the impacts of platoons on pavement for optimized configurations and how those corresponding platoons will affect traffic, along with improved pavement design, the negative impacts on both systems could be mitigated.



Figure 33: Autoroute 15 - May 2021 (Source: Google Earth®)



Figure 34: Autoroute 15- June 2024 (Source: Google Earth®)

The highway segment used for analysis is shown in greater detail in Figure 35 . The segment is split into four sections between major interchanges, based on data collection. Section 1 includes the first three exits and entry ramps, with the fourth exit and entry ramps just following Section 2, Section 3 between the fourth and fifth on-and-off-ramps, followed by Section 4.

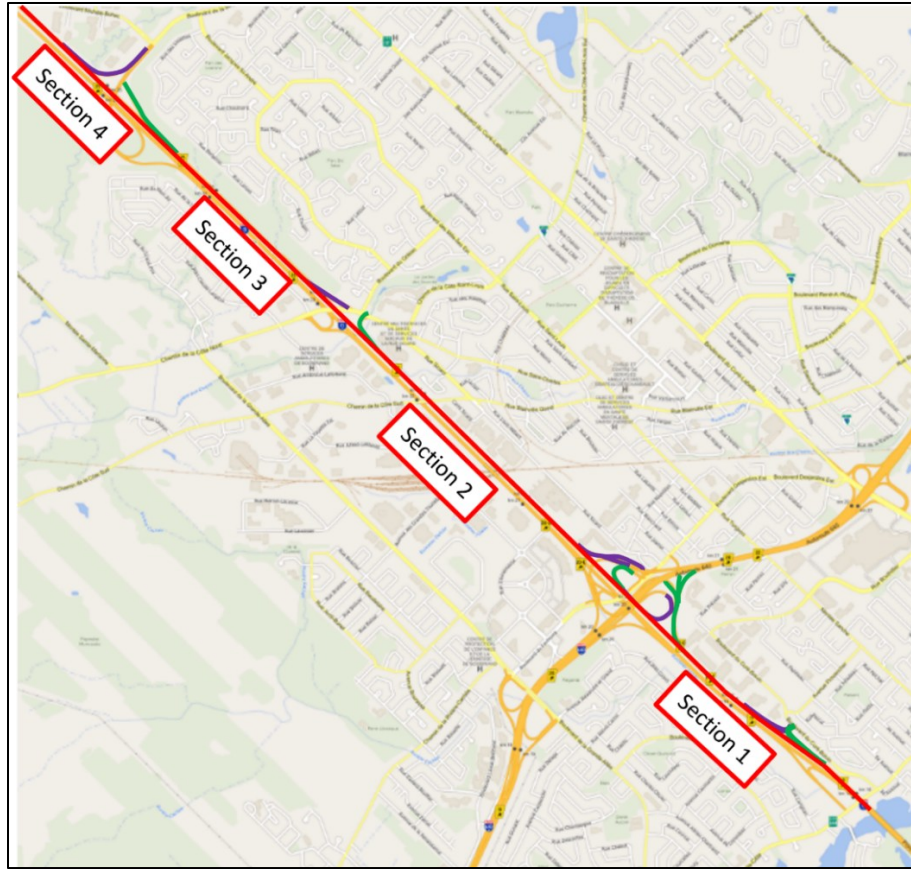


Figure 35: Autoroute 15 Modeled Section [209]

4.2.3 Data Collection

Figures 54 through Figure 61 in Appendix G show the traffic data used in the VISSIM simulation, obtained from open-source data from Transport Quebec. All data is from the interactive map, provided by *Infrastructure Géomatique Ouverte* (IGO), or the Open GIS Infrastructure developed by the *Gouvernement du Québec* [209]. The interactive map provides traffic data, including AADT (denoted as *DJMA* in the figures in the Appendix), top 30th hourly traffic (denoted as *30e heure*), and truck percentage (denoted as *%cam*) among others. The highlighted values in the figures indicate the traffic counts used in the simulation of this research. Data was taken from 2023 as much as possible, with interpolation or assumptions used for missing data. Detailed traffic data was available for four main sections along the highway; however, on-ramp and off-ramp data availability was more varied, with some ramps having limited hourly traffic counts and some having no data at all. Where hourly counts were available, the greatest values from peak hours were taken, and rounded up, meanwhile for segments with no data available, values were assumed based on traffic flow.

The 30th highest hourly flow rate counts were obtained for the year of 2023 for four highway sections, or interpolated where the data is missing, however, since this data represents the traffic flow in both directions, a directional split of 0.5 was used to determine the peak flow rate in the design direction (North-bound). Equation 7 shows the formula for determining the Directional Design Hourly Volume (DDHV) with the Design Hourly Volume (DDV) and the directional split (*D*). Table 14 shows the 30th hourly volume, or DHV, for the four main sections of Autoroute 15 under consideration, as well as the percentage of trucks, and the final corresponding DDHV, with data calculated from IGO data [209].

$$DDHV = DDV * D$$

Equation 7

Table 14: Highway 15 Sections' Traffic Data

Section	1	2	3	4
DHV	10538	14100	13350	12150
%Trucks	4.55	7.7	7.7	5.29
D	0.5	0.5	0.5	0.5
DDHV	5269	7050	6675	6075

Figure 36 shows the first highway section, along with all exits and on-ramps, as well as traffic inputs, for each on-ramp, and outputs for each exit and truck percentages where available. Section 2 is shown in Figure 37, and Sections 3 and 4 are displayed in Figure 38. All vehicle inputs are hourly values, with estimated or obtained greatest hourly flow rates to represent peak hours, as well as truck percentages, where data was available from the open data interactive map. Table 15 illustrates all traffic data used for the highway section for simulation, including the DDHV for the four main highway sections, the hourly counts for vehicles on off-ramps and on-ramps, and percent trucks where available.



Figure 36: Autoroute 15 Section 1 [209]

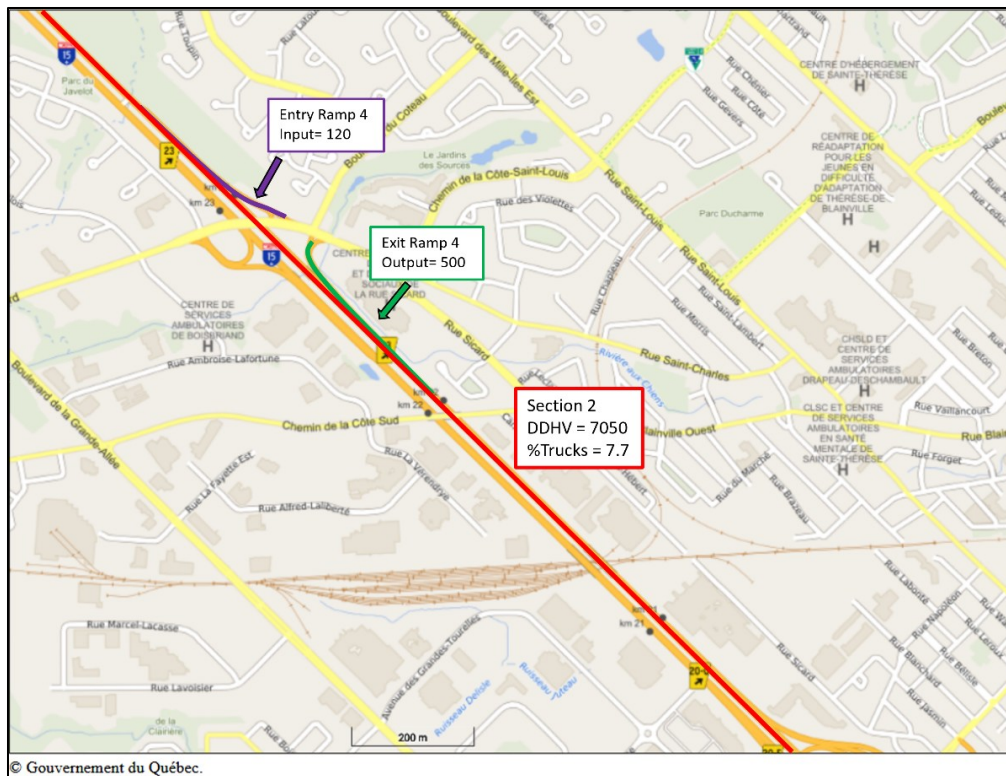


Figure 37: Autoroute 15 Section 2 [209]

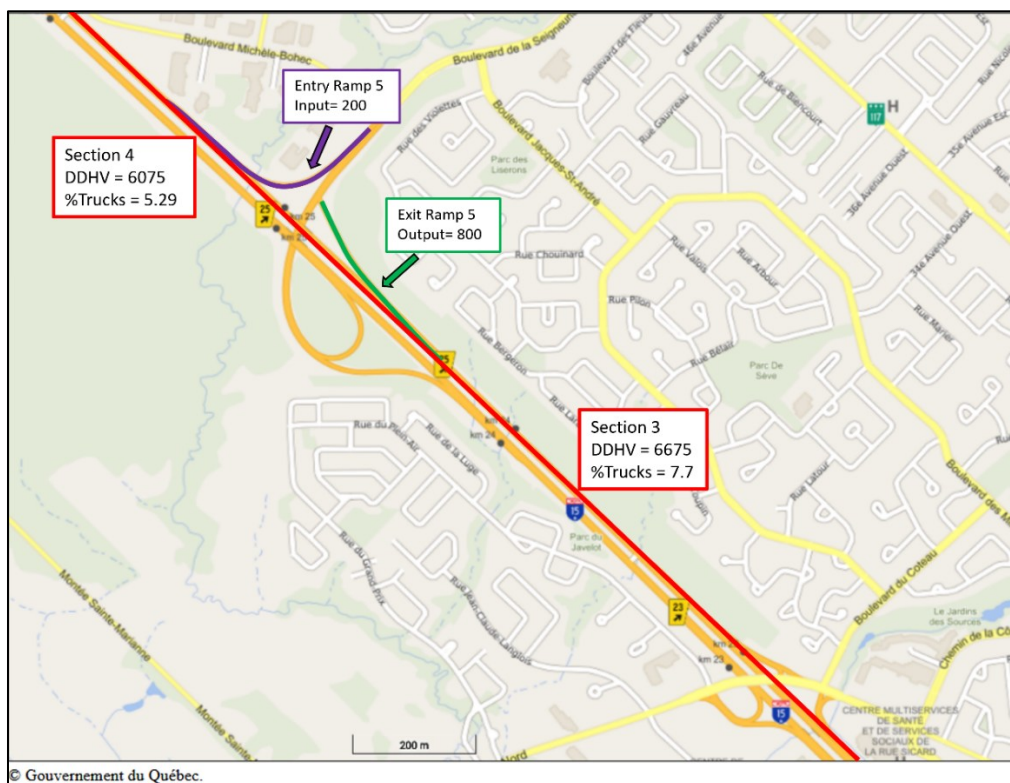


Figure 38: Autoroute 15 Sections 3 and 4 [209]

Table 15: Autoroute 15 Simulation Traffic Data

Traffic Section	Input	Output	%Trucks
Off-Bridge Entry (Section 1)	5269		4.55
Exit Ramp 1		99	4.44
Entry Ramp 1	830		
Exit Ramp 2-Left Exit		850	
Exit Ramp 2-Middle Exit		665	
Exit Ramp 2-Right Exit		1260	
Entry Ramp 2 (Clover)	1790		8.9
Exit Ramp 3		140	
Entry Ramp 3- Upper	780		3.5
Entry Ramp 3- Lower	1400		8.2
Section 2 DDHV	7055		7.7
Exit Ramp 4		500	
Entry Ramp 4	120		
Section 3 DDHV	6675		7.7
Exit Ramp 5	800		
Entry Ramp 5	200		
Section 4 DDHV	6075		5.29

4.2.4 Model Calibration

VISSIM microsimulation is carried out according to the default Wiedemann 99 car following model, which is recommended for freeway modeling, as well as for the car-following behaviour of AVs [89, 213]. The model is a psychophysical one, where driving behaviour is based on the perceived distances and speeds of the vehicle ahead [214]. Table 16 shows the nine main parameters used in the Wiedemann 99 car following model in VISSIM, along with the units and default values.

Table 16: Wiedemann 99 Default VISSIM Parameters for CVs under Freeway Conditions

Parameter	Unit	Default Value	Description
CC0	m	1.5	Standstill Distance
CC1	s	0.9	Gap Time Distribution
CC2	m	4.00	Following Distance Oscillation
CC3	s	-8.00	Threshold for Entering Following
CC4	m/s	-0.35	Negative Speed Difference
CC5	m/s	0.35	Positive Speed Difference
CC6	1/(m*s)	11.44	Distance Dependency of Oscillation
CC7	m/s ²	0.25	Oscillation Acceleration
CC8	m/s ²	3.50	Acceleration from Standstill
CC9	m/s ²	1.50	Acceleration at 80 km/h

In order for the simulation to be as representative of real-world conditions and provide the most accurate results, calibration of the model is carried out prior to the simulation analysis, where certain factors are modified to minimize the difference between observed and simulated results. The parameters *CC0*, *CC1* and *CC2* are the most significant parameters that should be changed from the default settings in VISSIM, as they have the most impact on the driving behaviour. The initial model calibration of Wiedemann 99 was carried out for Montreal-area freeway conditions in a previous study by Esmaeeli [215], where the researcher used Particle Swarm Optimization (PSO) to determine the model factors that most closely represent actual Montreal freeway driving conditions. The objective of the optimization function was to minimize the difference between simulated and observed results, and a sensitivity analysis was carried out to determine the most important parameters that should be calibrated. The results of the PSO are shown in Table 17 for the *CC0*, *CC1* and *CC2* values, as well as for the Maximum Trailing Deceleration (MTD) value, which was also determined through the PSO for Montreal freeway conditions. All CVs, including CTs, were modeled using these parameters.

Table 17: Montreal Freeway Calibrated Parameters [215]

Parameter	Units	Default Values	Calibrated Values
CC0	m	1.5	1.49
CC1	s	0.9	1.1
CC2	m	4.00	3.70
MTD	m/s ²	-3.00	-2.88

In addition to having default driving behaviour for CVs under freeway conditions, VISSIM also provides driving behaviour for AVs under freeway conditions, and more specifically for cautious, normal or aggressive AVs. This research will consider AVs as 'normal' and the default AVs driving behaviour for this condition on freeway driving will be used. Table 18 shows the default car following parameter for normally aggressive AVs. In addition to car following behaviour of AVs, autonomous driving behaviour was also selected to allow platooning with up to four vehicles per platoon, a maximum desired speed of 100 km/h, and a gap time of 0.5 s, to simulate the effect of ATs platoons with the parameters determined from the optimized pavement damage configuration.

The VISSIM model includes calibration for ATs and platooning and for Montreal driving behaviour, as well as for emissions based on Quebec vehicle types. Nouri and Morency [216] found that the emissions of any given vehicle are based on vehicle type, fuel type, speed and acceleration, mileage and maintenance, as well as temperature, pavement condition and asphalt concrete use, and it is based on their findings that an external .dll file was created specific to Quebec vehicles to determine emissions. As previously mentioned, only 0.0065% of trucks and buses in Canada are electric or hybrid vehicles, and for the purposes of this research, the CTs as well as the ATs platoons are to be considered as gasoline or diesel vehicles. The emissions results are therefore likely to be inflated, since it does not consider emission-less vehicles such as electric and hybrid vehicles, even though it is likely that AVs will also be electric. This research considers emissions based on current conventional vehicles.

Table 18: Wiedemann 99 Default VISSIM Parameters for Normal AVs

Parameter	Unit	Default Value	Description
CC0	m	1.5	Standstill Distance
CC1	s	0.9	Gap Time Distribution
CC2	m	0.00	Following Distance Oscillation
CC3	s	-8.00	Threshold for Entering Following
CC4	m/s	-0.10	Negative Speed Difference
CC5	m/s	0.10	Positive Speed Difference
CC6	1/(m*s)	0.00	Distance Dependency of Oscillation
CC7	m/s ²	0.10	Oscillation Acceleration
CC8	m/s ²	3.50	Acceleration from Standstill
CC9	m/s ²	1.50	Acceleration at 80 km/h

4.2.5 Testing Scenarios

In order to determine the effect that ATs will have on traffic operations given the specifics of the study area, varying penetration rates of ATs coupled with different traffic volumes are tested with VISSIM microsimulation. Based on the traffic data available for the selected highway segment, the percentages of trucks or HVs is in the range of about 4.5% to 8.9%, however, for simplicity of the simulation 10% HVs is used as an input, since this represents a worst-case scenario in the present condition and also accounts for the predicted increase in truck percentage in the future. For the purpose of this research, only the only AVs that are considered are ATs; ATs are predicted to be among the first types of vehicles to have higher level automation due to the simpler nature of highway driving and the possibility of hub-to-hub transportation and it is trucks and HVs that are the greatest consideration for asphalt pavement damage, and were considered in the previous tests.

As for the platoon configuration, the headway and speeds of the platoon are obtained from the optimum rest period and frequency from the asphalt testing section, with a headway of 0.5s between trucks in a platoon and a desired speed of 100 km/h. The maximum truck speed is limited to 105 km/h, since trucks in Quebec are limited to this speed [217]. The number of trucks in a platoon is chosen based on previous literature that considered platooning on traffic, fuel consumption, safety, and asphalt pavement. From a fuel savings point of view, a greater number of trucks results in more efficient fuel consumption [143, 144], however it is suggested fewer than four trucks per platoon for safety benefits without the loss of fuel savings [134, 152]. Smaller platoons are better with respect to the damage done to pavement [162] and research based on traffic operations suggests two-or-three-truck platoons during peak hours for optimum performance [134, 145]. Given the previous research, a platoon consisting of three trucks is considered here, due to the greater fuel benefits compared to a two-truck platoon, without compromising safety, traffic operations, or increased asphalt damage. The platoon configuration parameters are shown in Table 19.

The introduction of ATs will not come all at once, but will rather gradually happen in the future, resulting in a mixed-traffic case where the traffic consists of both AVs and CVs. For this reason, varying penetration rates of 25%, 50% and 100% of ATs have been simulated, to look at the possible effects at the different stages in the deployment of these vehicles in the future. Moreover, the traffic volume is also varied to observe what happens to traffic operations during peak hour and off-peak times, as well as at an increased demand to represent future peak conditions with traffic volumes of 100%, 75% and 125% of current peak-hour conditions. It should be noted that for the results, the traffic volumes are represented as 1.0, 0.75 and 1.25 respectively, based on the amount of the current traffic demand. The base scenario consists of 10% HVs with 100% of the current peak hour traffic demand and no ATs. The testing scenarios consist of the three traffic regimes coupled with each of the penetration rates and are summarized in Table 20 .

Table 19: Platoon Configuration and Simulation Parameters

Parameter	Value
Headway	0.5 s
Desired Speed	100 km/h
# Trucks per Platoon	3
% Heavy Vehicle	10%

Table 20: Simulation Scenarios

Scenario	Traffic volume	ATs Penetration Rate
Base	100%	0%
1.a.	100%	25%
1.b.	100%	50%
1.c.	100%	75%
1.d.	100%	100%
2.a.	75%	25%
2.b.	75%	50%
2.c.	75%	75%
2.d.	75%	100%
3.a.	125%	25%
3.b.	125%	50%
3.c.	125%	75%
3.d.	125%	100%

4.2.6 Evaluation and Measures of Effectiveness

In total, for the base scenario, as well as the three traffic volumes along with each of the three ATs penetration rates, 13 total scenarios are modeled, with the average values for each used for comparison. Each scenario was run a total of ten times, with 15 minutes of warm up time, 60 minutes of simulated run time, followed by a 15-minute cool down period. The traffic volumes are as shown above for the 60-minute run period and are proportionally scaled down for the warm-up and cool-down periods. Table 21 summarizes the simulation times for each run.

Table 21: Simulation Run Times			
	Warm Up	Simulation Run	Cool Down
Range	0-900	900s - 4500s	4500s - 5400s
Duration (s)	900	3600	900
Duration (min)	15	60	15

Vehicle performance measures were recorded during the simulation run period, and include average delay, average speed, total travel time and number of vehicles arrived, as well as emissions counts for CO₂, NO_x, PM₁₀ and PM₂₅. All values are obtained from VISSIM software, and total average travel time per vehicle is determined from the total travel time and the vehicles arrived. The averages of these results for each scenario and variable are determined, and a t-test is performed to determine significance of each parameter, as well as relative significance.

5. RESULTS AND DISCUSSION

5.1 Pavement Testing Results

As previously mentioned, the asphalt cylinder samples were tested under repetitive haversine loading-unloading and rest period cycles, for a total of 30 000 cycles, with a maximum loading of 175 kN, at 50 °C with different frequency and rest period combinations. Frequencies of 8 Hz and 10 Hz were tested in combination with rest periods of 0.5 s, 1.2 s, and 1.5 s. The notation for the results below follows the pattern of 'FXRY', where X indicates the frequency in Hz, and Y indicates the rest period in seconds. The raw data obtained from the testing includes the time and cycle, load magnitude, temperature and deformation. The load magnitude and temperature information were used to verify testing parameters as well as to ensure consistency among tests. The strain was determined using Equation 8, where, ε is the strain, δ is the measured deformation, and L_{exten} is the length of the extensometer (100-mm extensometers were used for the purposes of this research).

$$\varepsilon = \frac{\delta}{L_{\text{exten}}} \quad \text{Equation 8}$$

The data acquisition mentioned above, allowed for the stress, strain and corresponding cycle to be obtained for the extent of the experimentation. Once this information was obtained, data treatment allowed for summarization of the data, with information per every cycle, instead of at 0.05 second intervals, for easier analysis. For each cycle, the strain immediately following the loading period, as well as the peak strain and the strain after the rest period were computed based on load applications and cycle timings, and the loading period, rest period, and total cycle lengths were obtained as well. The loading time, rest period and cycle time were all checked such that these time periods were within 0.05 seconds of the set values. Table 22 shows the expected, average (denoted with *ave*), and standard deviation (denoted with *dev*) times for the loading period and rest period for each frequency-rest period combination, as well as the total test time. The standard deviations for the loading and rest periods range from about 0.0018 to 0.0055 indicating that there is little variation from the expected and average values for the time periods parameters set in the experiment.

Table 22: Test Times

Test	t_{loading} (s)	$t_{\text{loading_ave}}$ (s)	$t_{\text{loading_dev}}$ (s)	t_{rest} (s)	$t_{\text{rest_ave}}$ (s)	$t_{\text{rest_dev}}$ (s)	t_{total} (s)
F8R1.5	0.125	0.126	0.0043	1.5	1.504	0.0018	49 041.05
F8R1.2	0.125	0.126	0.0040	1.2	1.204	0.0034	40 049.55
F8R0.5	0.125	0.127	0.0050	0.5	0.505	0.0032	19 094.40
F10R1.5	0.1	0.101	0.0055	1.5	1.505	0.0031	48 325.25
F10R1.2	0.1	0.101	0.0039	1.2	1.204	0.0030	39 302.37
F10R0.5	0.1	0.101	0.0049	0.5	0.504	0.0024	18 293.94

Using the treated strain values, the permanent strain for each cycle was determined as the difference in the strain immediately following the rest period and the strain at the beginning of the cycle. The recovered strain was found as the difference in the strain following rest and the maximum strain measured at the peak of loading, when the maximum stress occurred. The equations for the permanent and recovered strain for each cycle can be seen in Equation 9 and Equation 10, respectively. The total cumulative strain and total recovered strain were found by

summing up the accumulated permanent strain and the accumulated recovered strain, respectively. Finally, the percent strain recovered is the ratio of the total accumulated recovered strain to the total accumulated permanent strain, represented in Equation 11.

$$\varepsilon_{\text{permanent}} = \varepsilon_{\text{after}_{\text{rest}}} - \varepsilon_{\text{initial}} \quad \text{Equation 9}$$

$$\varepsilon_{\text{recovered}} = \varepsilon_{\text{after}_{\text{rest}}} - \varepsilon_{\text{max}} \quad \text{Equation 10}$$

$$\% \text{Recovered} = \frac{\sum_{i=1}^n \varepsilon_{\text{recovered}}}{\sum_{i=1}^n \varepsilon_{\text{permanent}}} \quad \text{Equation 11}$$

The average values for the two iterations of each frequency and rest period were taken and used for analysis and comparisons. Table 23 shows the final accumulated strain for each of the two iterations for each frequency-rest period combination, as well as the average accumulated strain for each combination. The 10 Hz frequency with any rest period combination resulted in lower average accumulated strain than at 8 Hz, and the 0.5 s rest period combined with either frequency also had the least average accumulated strain for each frequency. For both 8 Hz and 10 Hz, the difference in rest period between 0.5 seconds and 1.2 seconds or 1.5 seconds is significant, however, the difference between 1.2 and 1.5 seconds is much more minimal.

Table 23: Strain Accumulation Values and Averages Among Iterations

Frequency (Hz)	Rest Period (s)	Accumulated Strain	Average Accumulated Strain
8	1.5	39.991	40.087
	1.5	40.182	
8	1.2	42.243	41.431
	1.2	40.618	
8	0.5	38.424	38.515
	0.5	38.605	
10	1.5	39.177	38.352
	1.5	37.527	
10	1.2	38.695	38.908
	1.2	39.122	
10	0.5	35.162	35.354
	0.5	35.545	

For comparison among tests, the total accumulated strain in 2000 and 30 000 cycles is displayed, as well as the total accumulated recovered strain and the percent of recovered strain is shown in Table 24 . 2000 cycles were chosen as a comparison point, since based on the strain-cycle curves this is within the area of rapid change of strain accumulation, while 30 000 cycles show the final values. The total accumulation of the permanent strain after 30 000 cycles was displayed and discussed above in Table 23, with the *F10R0.5* combination having the least total strain with 35.162 strain, whereas the greatest overall strain recorded was with the *F8R1.5* combination. This trend is consistent with the accumulated permanent strain at 2000 cycles as well. The greatest percent strain recovered at both 2000 and 30 000 cycles occurred at a frequency of 10 Hz with a rest period of 1.5 seconds, while the least percent recovered are also at a frequency of 8 Hz and 1.5 seconds of rest. Finally, the greatest total recovered strain occurred for both frequencies with the largest rest period.

For both frequencies, between 0.5 seconds of rest period and 1.5 seconds of rest period there is an overall increase in the percentage of recovered strain at both 2000 and 30 000 cycles. However, for both frequencies, the middle rest period value of 1.2 seconds, shows a decrease in recovered strain compared to the 0.5 second rest period. Moreover, there is greater percentage of recovered strain for the tests at 10 Hz frequency compared to the 8 Hz frequency for each similar rest period. Regarding the total accumulated strain, the lower rest periods for both frequencies are found to have lower strain at the end of testing, despite a lower percentage of strain recovered. The total accumulated strain is also lower for the combinations of 10 Hz frequency with any rest period.

Table 24: Permanent and Recovered Strain Comparison

	Total Accumulated Strain		Total Recovered Strain		%Recovered	
	2000 Cycles	30000 Cycles	2000 Cycles	30000 Cycles	2000 Cycles	30000 Cycles
F8 R1.5	21.614	40.528	346.365	1445.922	16.03	35.68
F8 R1.2	22.155	41.430	285.815	1294.491	12.97	31.28
F8 R0.5	21.183	38.515	278.632	1279.811	13.15	33.23
F10 R1.5	20.466	38.352	341.779	1504.069	16.7	39.18
F10 R1.2	20.297	38.908	301.496	1311.357	14.81	33.73
F10 R0.5	19.129	35.162	297.183	1347.971	15.54	38.34

The strain-cycle diagrams for the different rest period and frequency combinations are shown below. The average final strain following the rest period for each cycle between the two iterations was found for each combination. Figure 39 and Figure 40 display the strain-cycle diagrams for frequencies of 8 Hz and 10Hz, respectively, in combination with all the rest periods, while Figure 41, Figure 42 and Figure 43 show the diagrams for rest periods of 1.5 s, 1.2 s and 0.5 s in combination with the two rest periods, respectively. The strain-cycle diagrams are shown for all 30 000 cycles tested. It should be noted that the strain values are shown to be increasing in the negative direction, this is due to sign convention wherein compressive force is denoted as negative. The absolute values may be considered for strain, despite being shown as negative.

It may be observed from the curves below that all follow the same pattern and trends, as mentioned before. For the curves comparing the rest periods for each frequency, the 0.5 second rest period results in the lowest strains, while the 1.5 second and 1.2 second rest period curves for both frequencies are closely matched for all cycles. There is a greater significance of strain accumulation difference between rest periods for 10 Hz as opposed to 8 Hz. For the curves comparing the frequency per each rest period, the similar logarithmic trend is observed, and for each different rest period the frequency of 10 Hz has noticeably less strain throughout the testing, and this effect is especially significant for 0.5 seconds, in Figure 43. For each curve, the logarithmic trendline and R^2 values are displayed as well. A logarithmic trendline was used for each of the curves, since the stress-strain relationship of asphalt follows a logarithmic or exponential trend, as it is a viscoelastic material, as previously mentioned. It can be seen from the curves in the graphs below that the R^2 value for all curves is above 0.96, indicating that these curves are fitted well. The strain-cycle curves below follow the trend of strain-time diagram shown in Figure 19, however, since only 30 000 cycles were completed, instead of running the test to failure, the second sharply increasing strain portion of the curve was not obtained in this experiment. If samples had been tested to failure, it is likely the same pattern would have been observed.

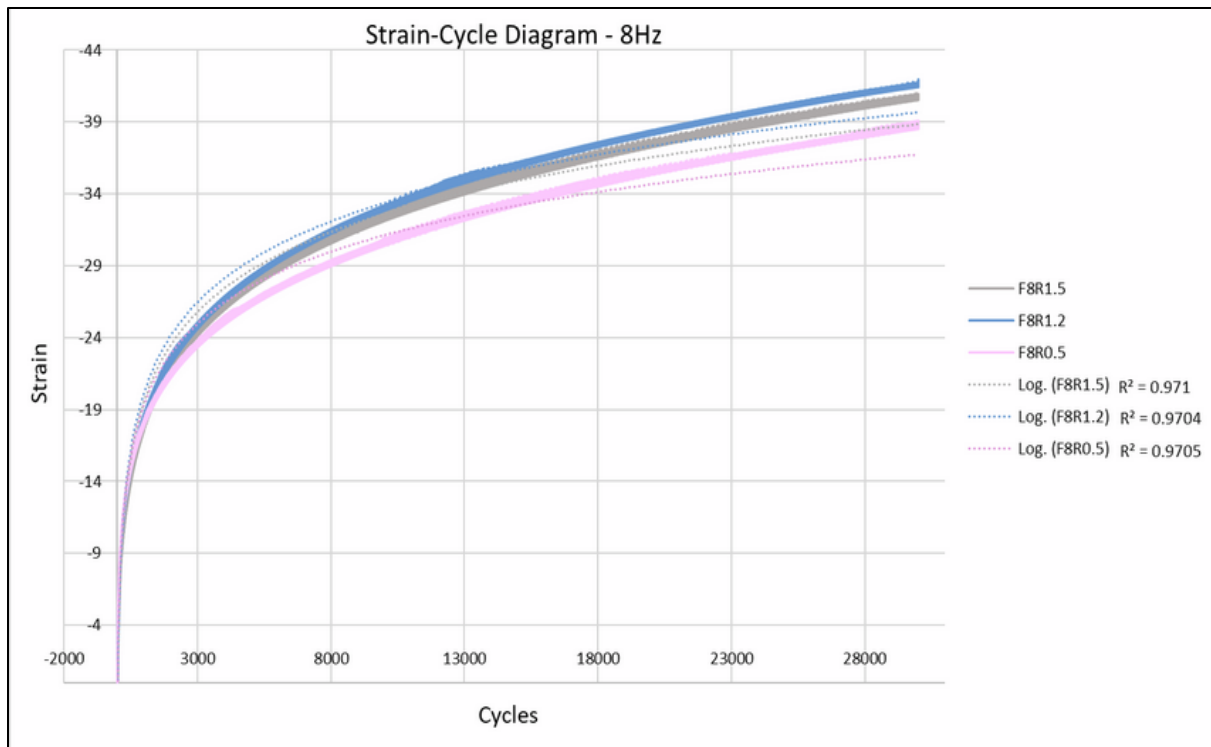


Figure 39: Strain-Cycle Diagram at 8 Hz

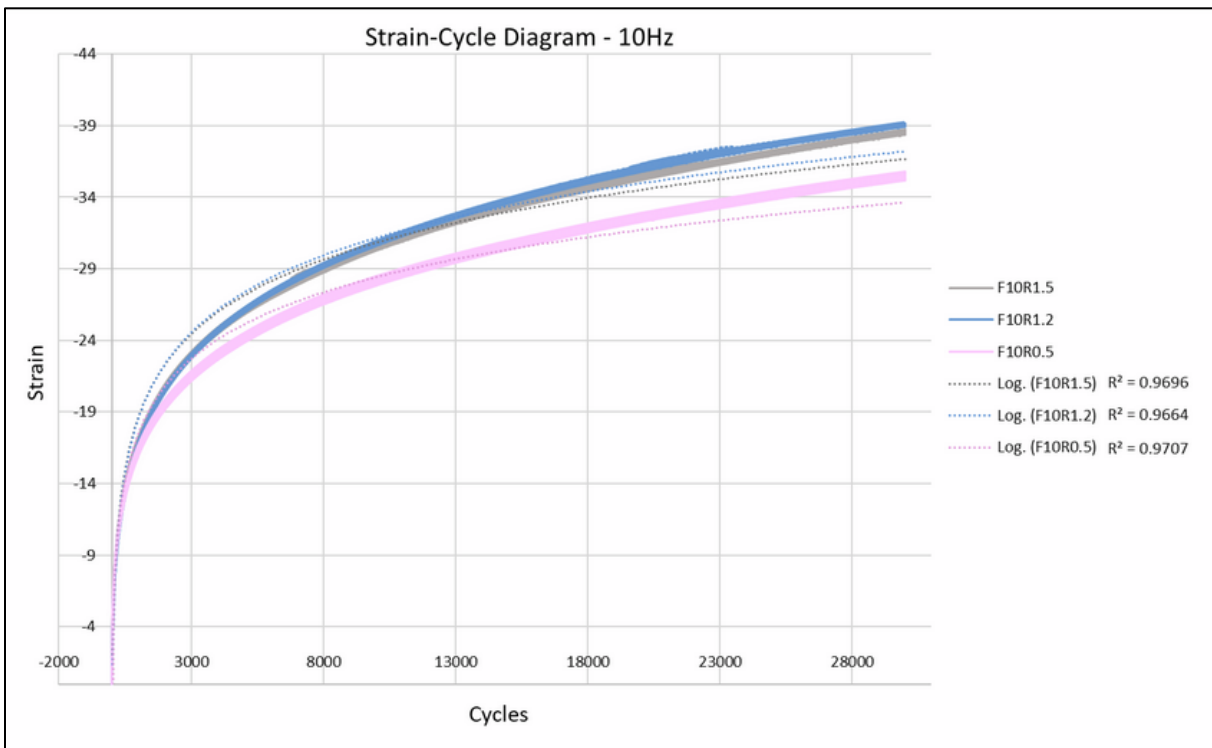


Figure 40: Strain-Cycle Diagram for 10 Hz

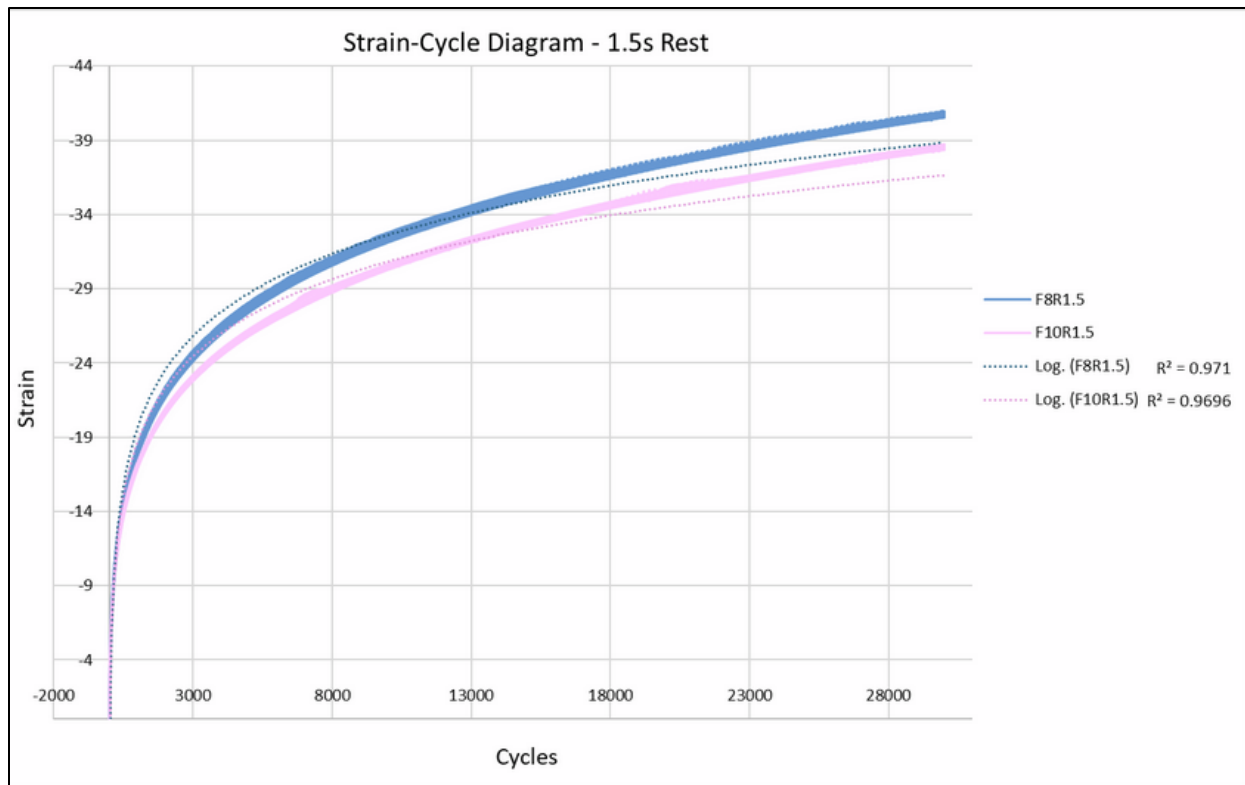


Figure 41: Strain-Cycle Diagram with 1.5 s Rest

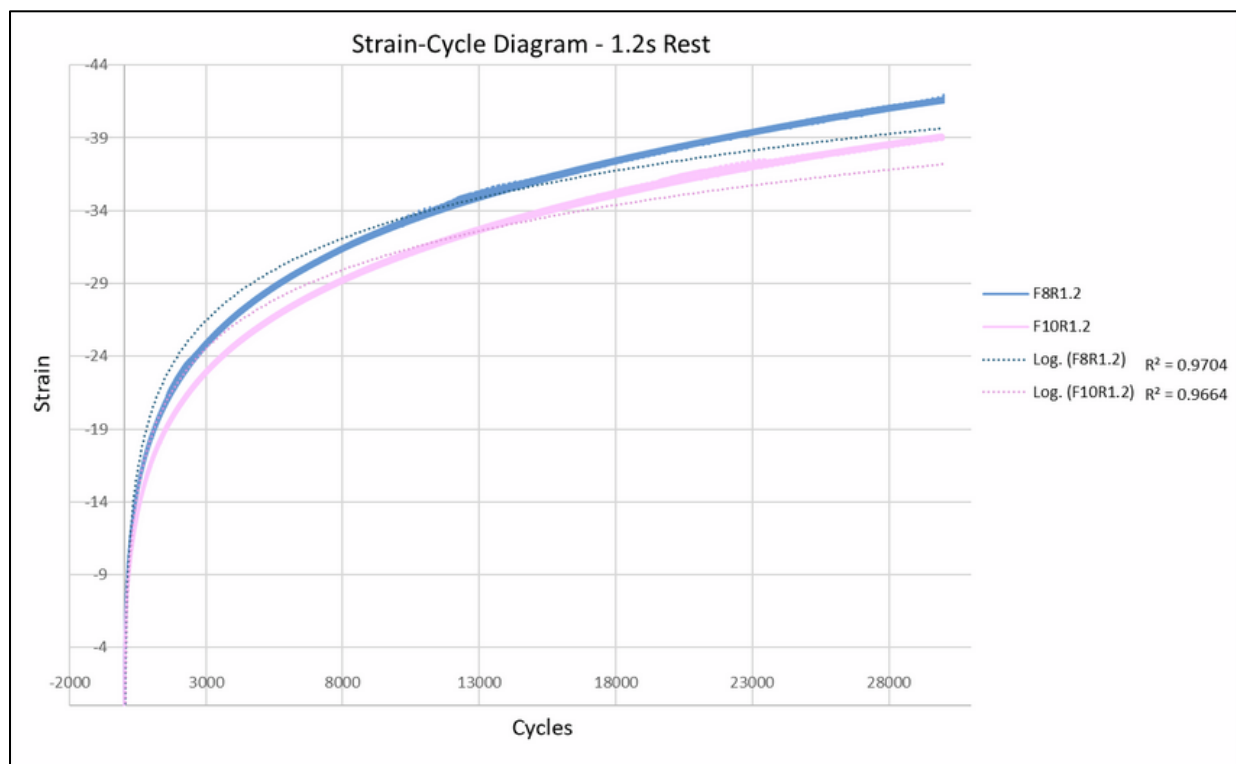


Figure 42: Strain-Cycle Diagram with 1.2 s Rest

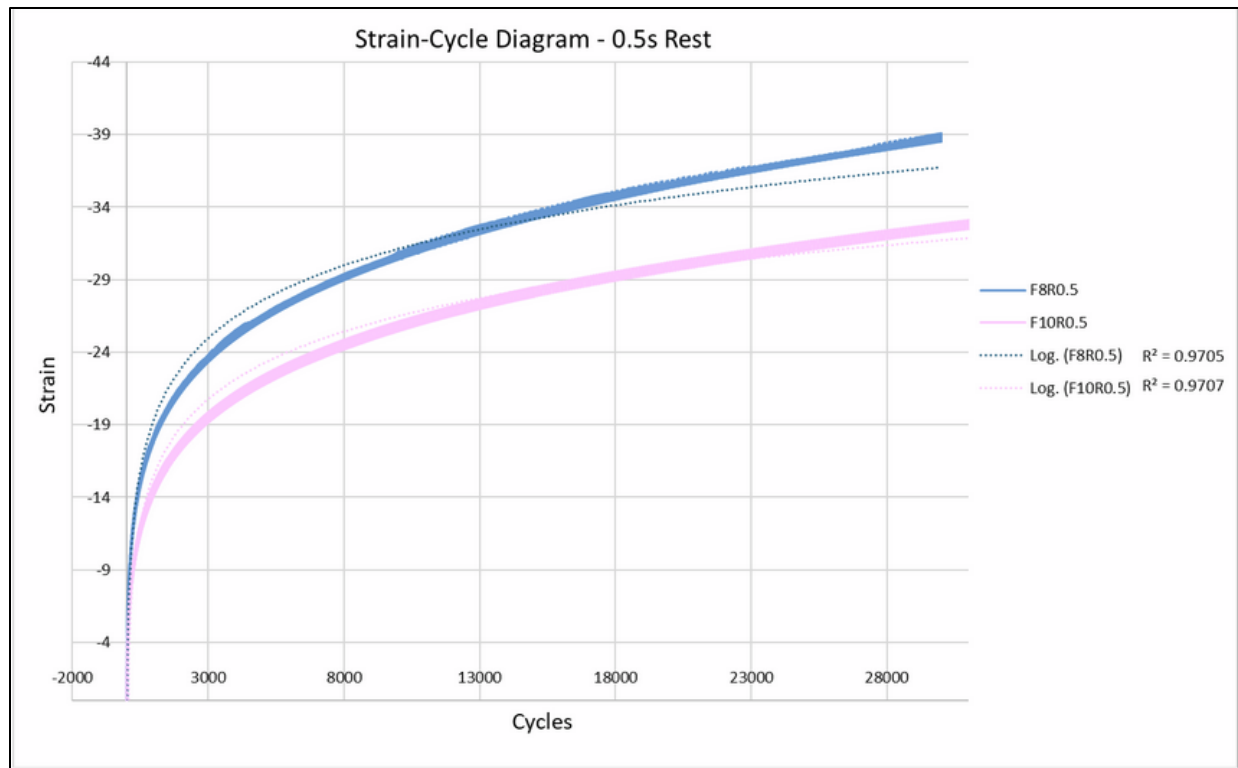


Figure 43: Strain-Cycle Diagram with 0.5 s Rest

The asphalt test results indicate that the least amount of accumulated permanent strain occurs with the higher frequency, and therefore least loading period, and shortest rest period combination, in this case the 10 Hz frequency and 0.5 second rest period. When tested, the 10 Hz frequency resulted in lower accumulated strain than the samples tested with 8 Hz frequency, and the 0.5 second rest time resulted in the same when compared to the other rest periods. Moreover, the percent recovered was the greatest for the 1.5 second rest period combinations, and specifically the greatest percent recovered strain was found with 10Hz frequency. In addition to having a greater amount of accumulated strain, the rate of strain accumulation is also higher at a lower frequency. Figure 44 shows an example of platooning trucks with the optimum configuration, with 10 Hz representing a speed of 100 km/hr.

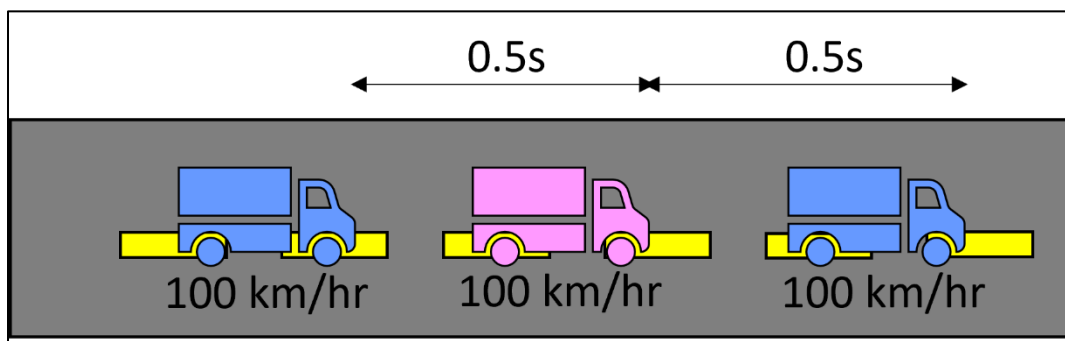


Figure 44: Truck Platooning

5.2 VISSIM Simulation Results

VISSIM simulation was carried out to determine the effect of ATs platoons of different penetration rates of ATs paired with different traffic regimes on traffic operations and emissions in Quebec. A total of ten runs were performed for each of the 13 scenarios, and the average and standard deviation for parameter of each scenario was obtained from the software. Table 25 shows a summary of the results for the average network delay, the average vehicle speeds and travel times as well as the average total emissions for CO₂, NO_x, PM₁₀ and PM_{2.5}. Figure 45 through 51 present the average results of 10 simulated runs for each of the parameters tested. The base scenario with current traffic is shown and compared to the scenarios with varying ATs penetration rates and traffic volumes. The full results for all scenarios can be found in Table 53 through Table 65 in Appendix H. The traffic volumes for the results are represented in terms of proportion of the current peak hour traffic.

Table 25: Simulation Results Summary

Scenario	Delay (s)	Speed (km/h)	Travel Time (s)	CO ₂ (kg)	NO _x (kg)	PM ₁₀ (kg)	PM _{2.5} (kg)
Base	30.34	77.93	139.83	53.97	143.93	413.79	467.76
1.0, 25% ATs	21.95	83.34	130.74	50.93	135.82	390.49	441.43
1.0, 50% ATs	18.43	85.79	126.79	49.52	132.06	379.68	429.20
1.0, 75% ATs	8.00	88.87	119.66	47.04	125.44	360.64	407.68
1.0, 100% ATs	11.16	91.19	118.13	46.45	123.87	356.13	402.58
0.75, 25% ATs	3.26	97.22	106.15	33.74	89.98	258.69	292.43
0.75, 50% ATs	3.09	97.41	105.16	33.43	89.16	256.33	289.77
0.75, 75% ATs	1.38	93.80	106.23	33.78	90.07	258.97	292.74
0.75, 100% ATs	2.72	97.84	103.45	32.92	87.79	252.39	285.31
1.25, 25% ATs	35.76	74.61	143.90	59.65	159.07	457.34	516.99
1.25, 50% ATs	34.98	75.05	142.89	59.64	159.04	457.24	516.88
1.25, 75% ATs	30.34	74.05	141.35	59.34	158.24	454.94	514.28
1.25, 100% ATs	32.52	76.44	139.63	59.03	157.41	452.56	511.58

Figure 45 demonstrates the average delay of the network for all tested scenarios. As expected, the delay is increased with a greater traffic volume, and reduced with a lower traffic volume. When ATs are introduced into the network, the delay for the current traffic volumes is reduced compared to the base scenario, and for all traffic demands a similar pattern is observed, with decreased delay up to 75% penetration of ATs, followed by a slight increase at 100%. This observed change in delay is the most significant under the normal traffic volume and has the least impact when the traffic volumes are reduced. Under the current traffic volumes, the delay can be decreased by up to 74% with 75% penetration rate of platooning ATs, and even only a 25% penetration rate in ATs would result in a reduction in delay of 28%. An increase in penetration rate of ATs greatly reduced the delay under the normal traffic volumes, and although the 1.25-times traffic volumes was not simulated without any ATs, it can be assumed that the delay is increased due to the increase in traffic volumes, and therefore the presence of ATs are able to mitigate the effect of the increased

traffic on the delay. The increase in penetration rate at the 1.25 traffic flow results in a 15% decrease in delay when comparing 25% ATs and 75%. At 75% - 100% the impact of the increased traffic volumes on the delay is mitigated, almost to the levels of the previous traffic volumes. As traffic volumes is ever increasing, the introduction of platooning ATs, especially at higher penetration rates, will be able to reduce the negative effects of an increased demand on delay.

The travel time per vehicle is shown in Figure 46. Similarly, to the delay, the vehicle travel times are expectedly greater in increased traffic volumes and are reduced when traffic volume is lower. Overall, for all the traffic volumes, as the penetration rate of platooning ATs is increased the travel time is decreased. An exception to this trend is observed in for the 75% ATs penetration rate for the reduced traffic volumes, although the difference is very slight. This trend is most prevalent under the normal traffic conditions. Compared to the base scenario, ATs result in reduced travel time under current traffic volumes, while for the increased traffic volume the presence of ATs, especially at greater penetrations rates, can mitigate the effect of greater traffic flow on the travel time.

Figure 47 demonstrates the average vehicle speed for the different traffic volume and ATs penetration rate combinations. The average speed is expectedly increased for a reduced traffic volume, and is reduced during greater traffic volumes compared to current peak levels. Under the normal traffic volume, the average vehicle speed is increased as the ATs penetration is increased, however, this trend does not extend to the 0.75- or 1.25-times traffic volumes, which show that for 25% and 50% penetration rates, the speeds are similar, followed by a decrease in speed at 75% ATs and the greatest speed at 100% ATs. For the base traffic volume, an increase in penetration rate of ATs from 0 to 100% would result in a speed increase of up to 14.5%, however this speed difference is less significant at different traffic volumes. The introduction of ATs, especially at greater penetration rates will be able to almost offset the decrease in speed due to an increase in traffic volume, as shown by the 1.25 traffic volume average speeds compared to the baseline value.

The CO₂, NO_x, PM₁₀ and PM_{2.5} emissions results for the simulated scenarios are shown in figures Figure 48 through Figure 51. For all of the emissions, the same trends are observed; greater traffic volumes result in greater emissions and with increasing ATs penetration rates there is a decrease in emissions. It should be noted that there is an exception to this trend for the 75% ATs penetration rate and 0.75 traffic volume, where there is a small increase in emissions. For the base traffic volume, increasing the penetration rate from 0 to 100% ATs trucks could potentially result in a decrease of 16.2% in emissions. As mentioned previously, this work only considers trucks and HVs as conventional internal combustion vehicles,, and therefore these values are likely to be even more significant with increasing EVs penetration rate, especially for HVs. While ATs will likely be EAVs, this analysis quantifies the benefits of improved aerodynamic flow associated with truck platooning.

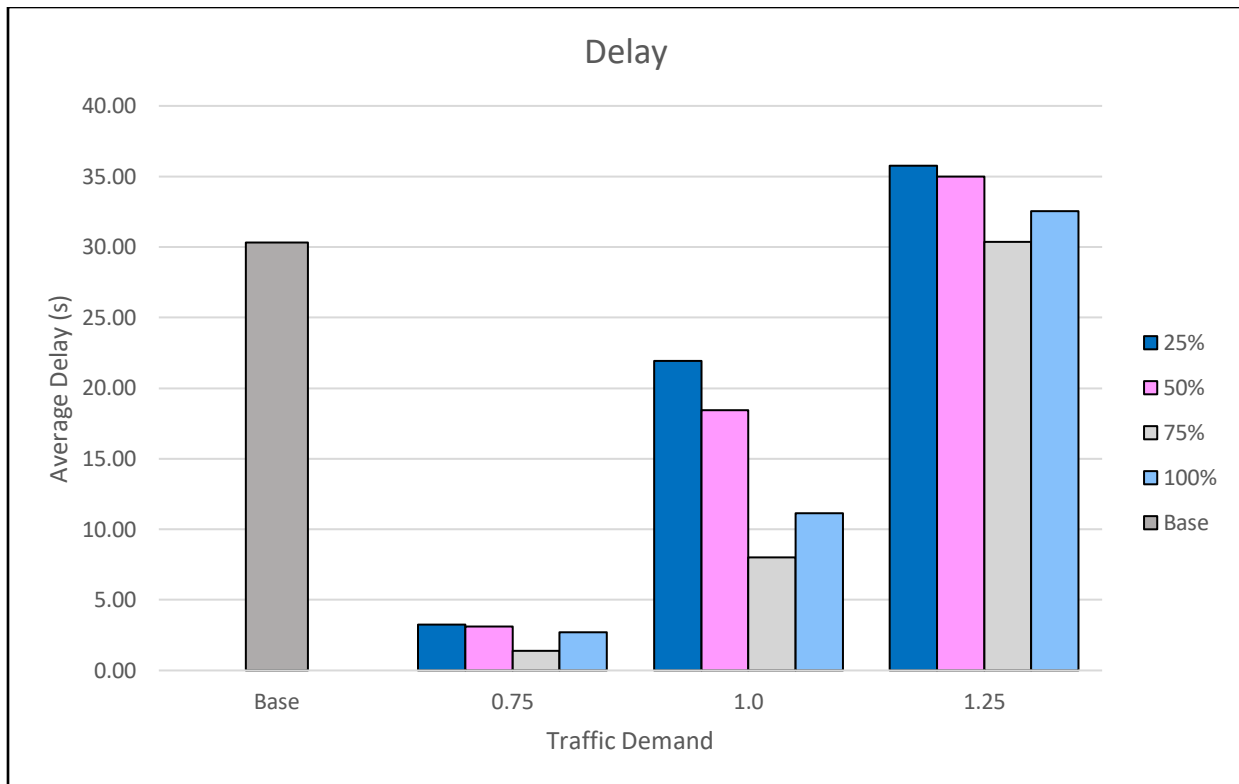


Figure 45: Average Network Delay

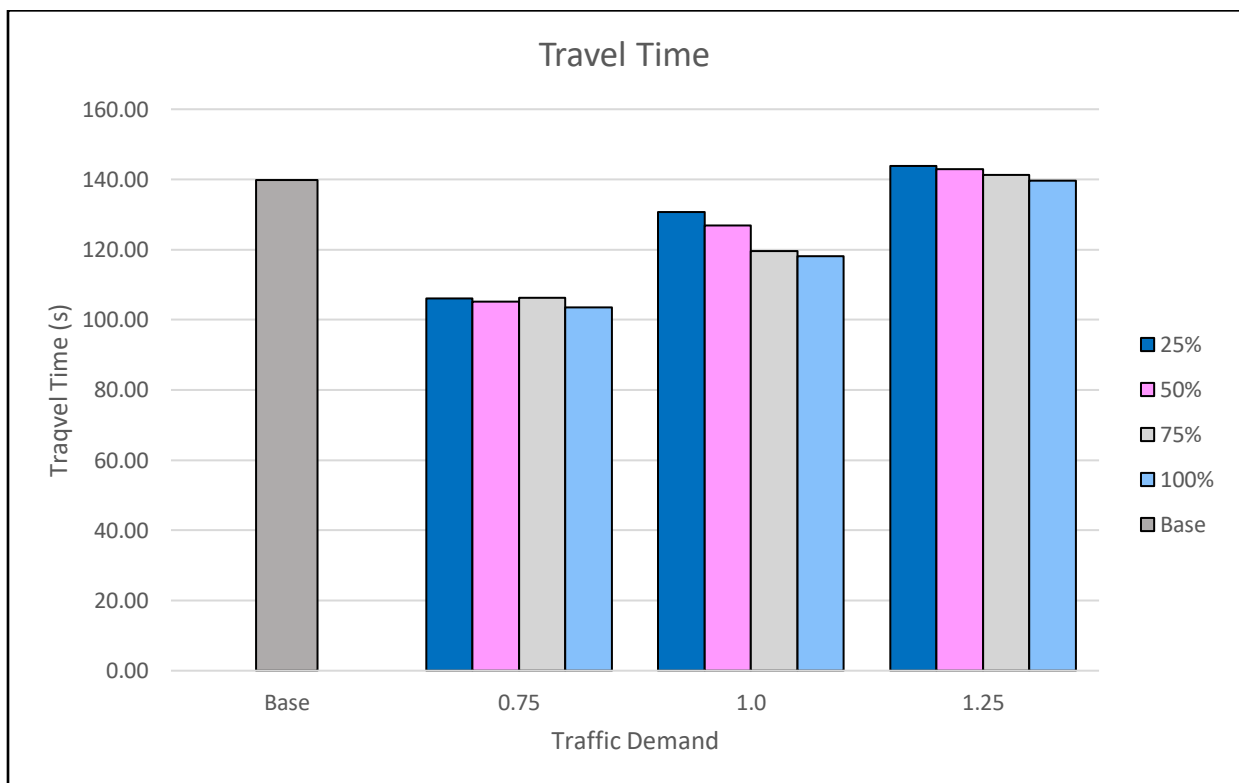


Figure 46: Average Vehicle Travel Times

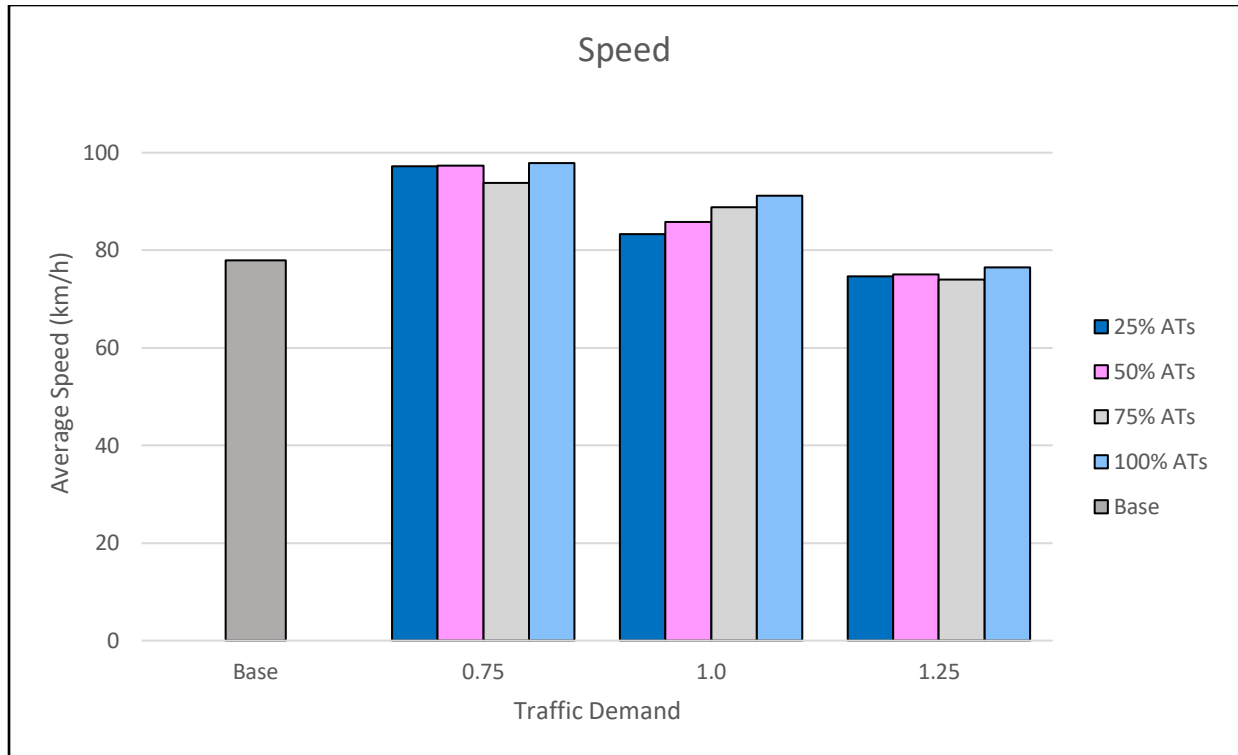


Figure 47: Average Vehicle Speed

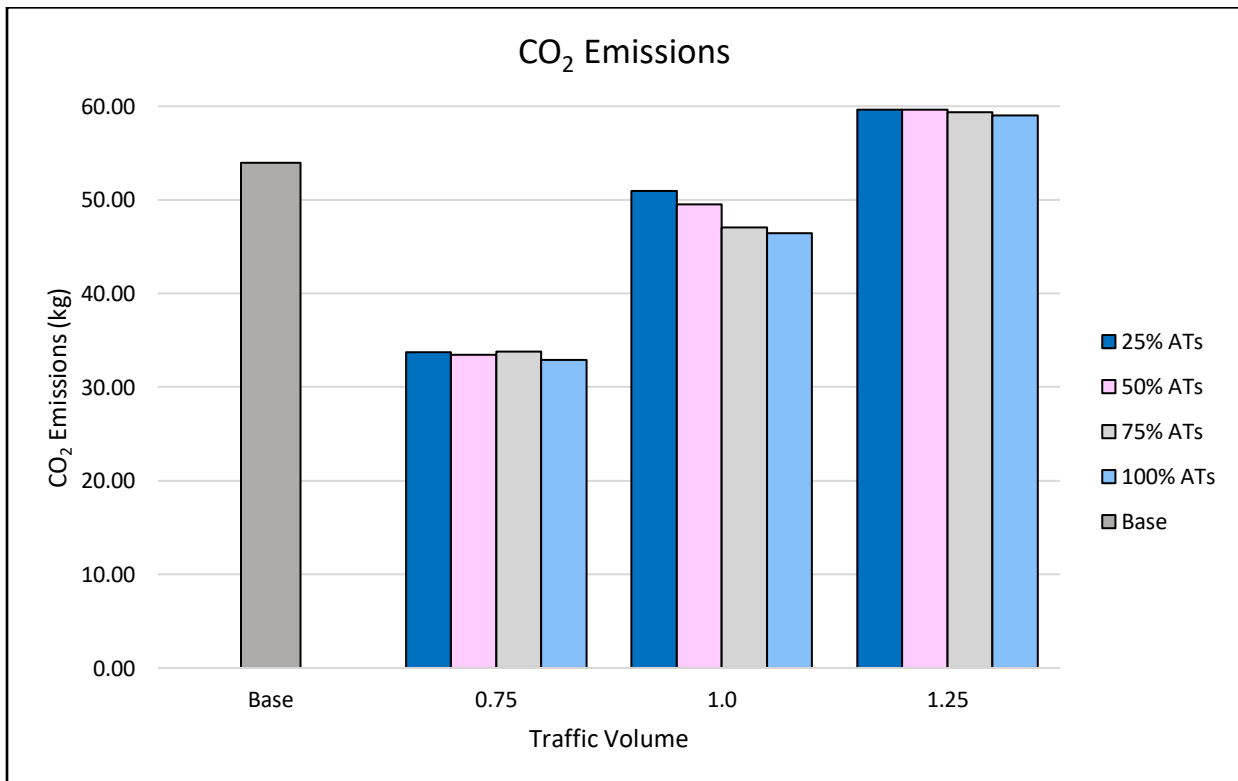


Figure 48: Vehicle CO₂ Emissions

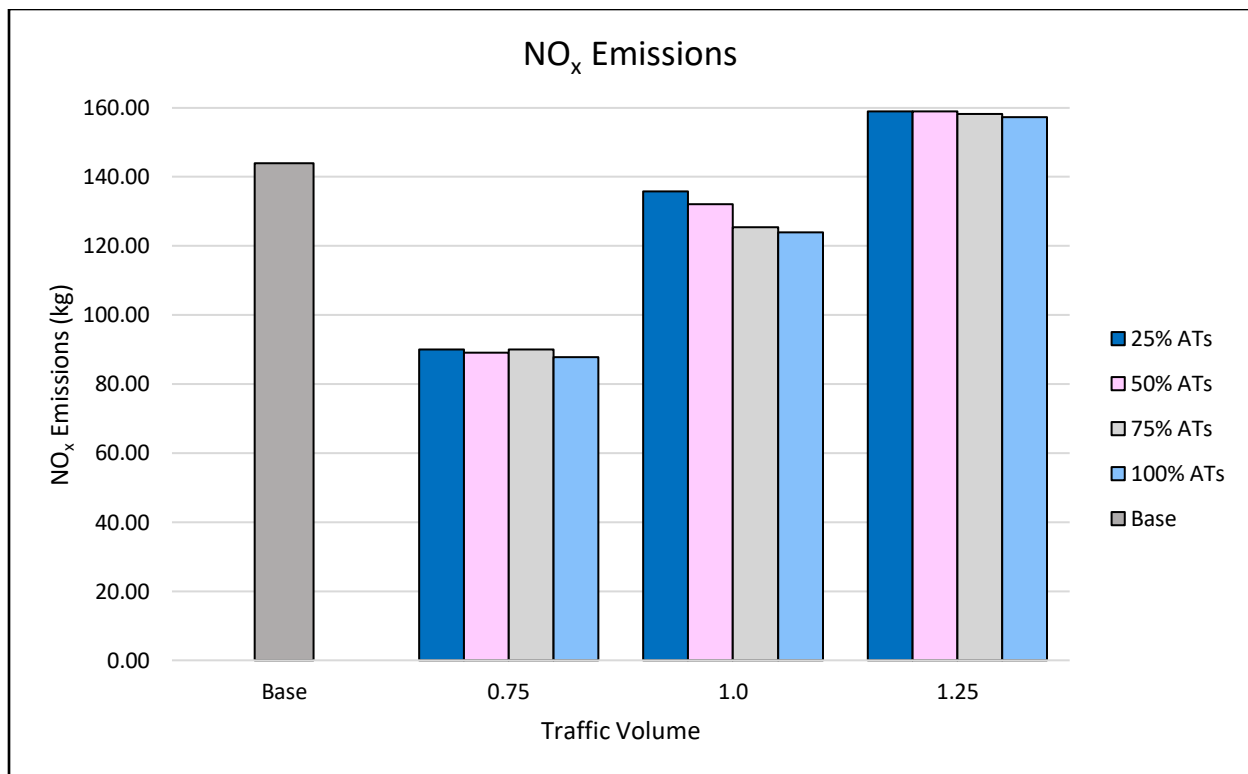


Figure 49: Vehicle NO_x Emissions

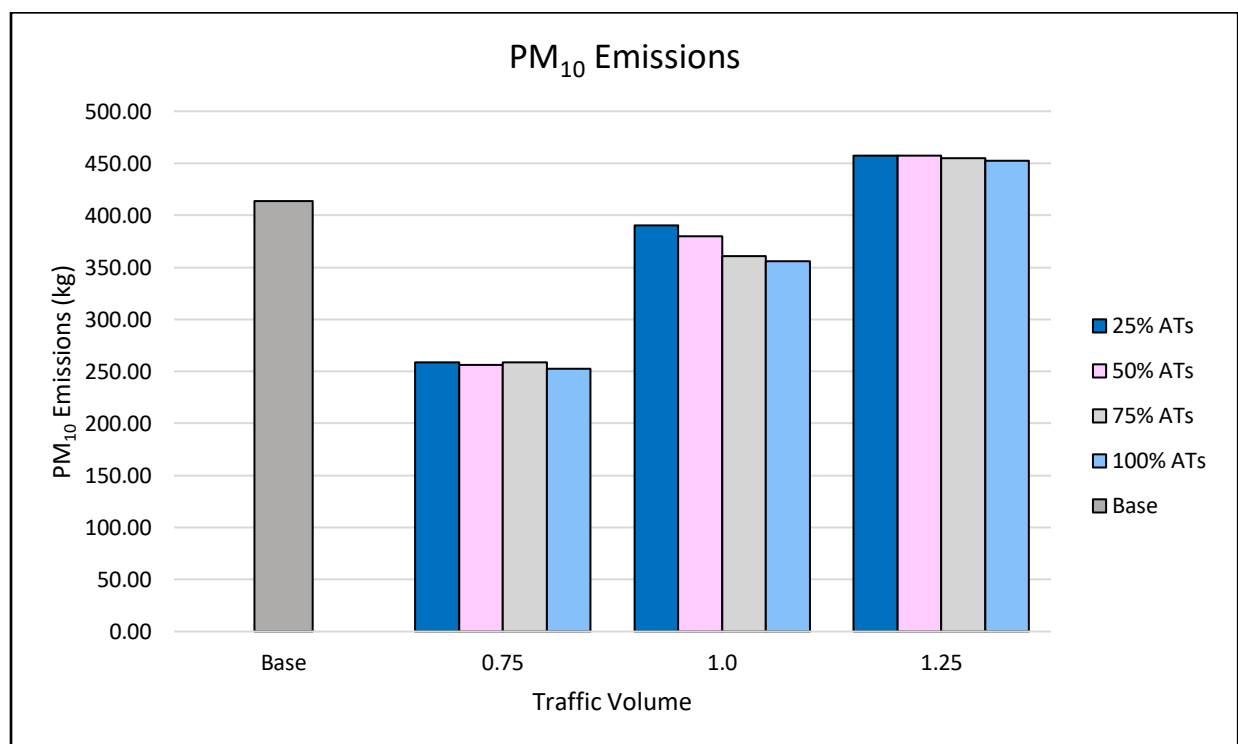


Figure 50: Vehicle PM₁₀ Emissions

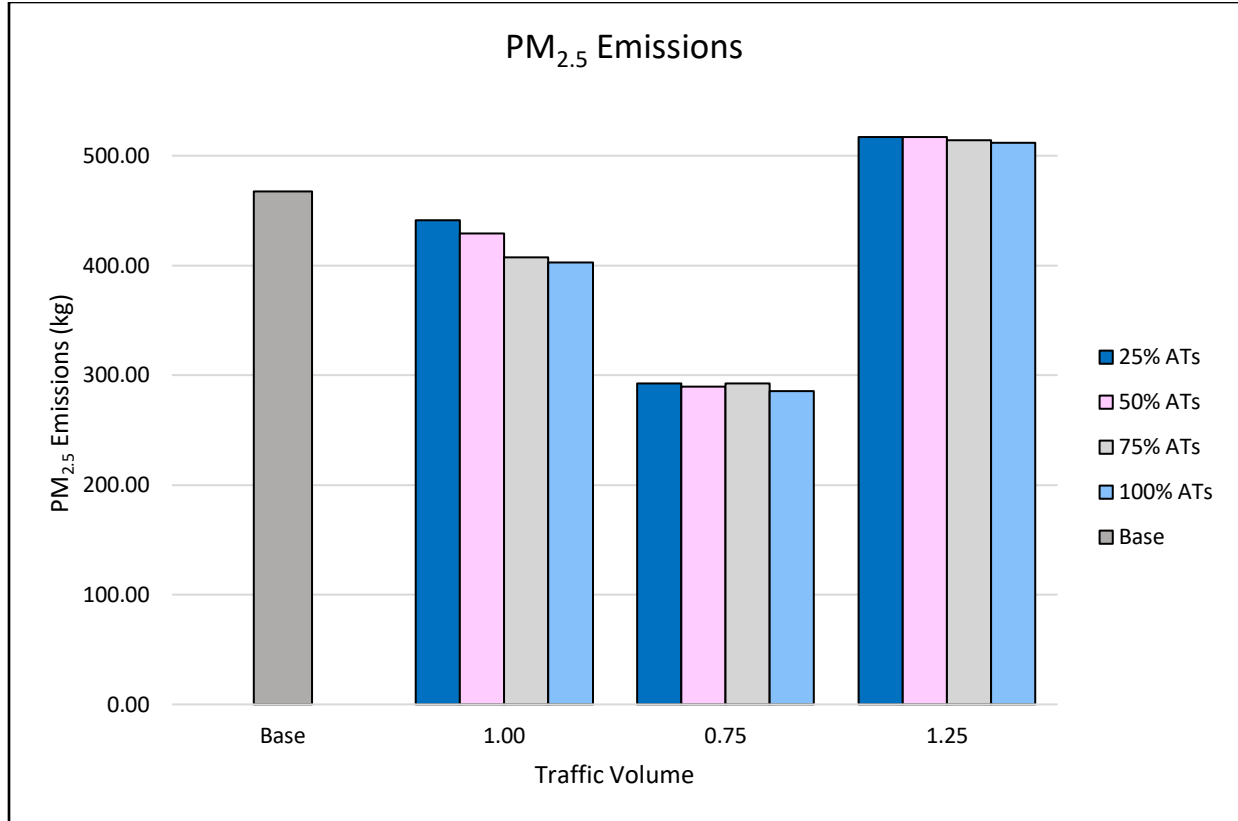


Figure 51: Vehicle PM_{2.5} Emissions

To determine the significance of the parameter's tested, a two-tailed t-test was carried out with a significance level of 0.05. The average and standard deviation values from the VISSIM simulation results were used. For each traffic volume, the significance of increasing ATs penetration rates was determined, and for each penetration rate the significance of the greater or lowered traffic volumes was found as well. For the t-test, the variance between the two groups is found using Equation 12 where σ is the standard deviations and N is number of trial runs (in this case, 10 for all scenarios); the degrees of freedom (DoF) found through Equation 13 and the t-score determined from Equation 14, where μ represents the average. The t-score is then compared with the value from a two-tailed t-test chart given from the significance level and the DoF, in this case 2.101, and if the t-score is greater, than the parameters are significant.

$$S_d = \sqrt{\frac{\sigma_1^2}{N_1} + \frac{\sigma_2^2}{N_2}} \quad \text{Equation 12}$$

$$DoF = N_1 + N_2 - 2 = 18 \quad \text{Equation 13}$$

$$t = \frac{1}{S_d} * (\mu_1 - \mu_2) \quad \text{Equation 14}$$

A sample of the t-test carried out for the baseline traffic volume with penetration rates from 0 – 100% is shown Table 26, with all values compared to the baseline (0% ATs) scenario. The average and standard deviation values, along with the variance, t-score and whether or not the parameter is significant is shown in the table for the average delay, average stops per vehicle, average vehicle speed, as well as for CO₂, NO_x, PM₁₀ and PM_{2.5} emissions. Similar tables for the significance of ATs penetration rate and traffic volume are shown in Table 66 through Table 71 in Appendix I, along with a summary table of the significance of parameters.

For any ATs penetration rate, both an increase and a reduction in traffic volume levels compared to the normal, current conditions are significant in terms of all the parameters measured. When traffic volume is reduced, for the 0.75 demand, the average delay and the average travel time are reduced, while the average vehicle speed is increased, and all emissions are decreased as well. For an increase in traffic volume, up to 1.25 times, the reverse is true; the average speed is decreased, and all other parameters are increased compared to the normal traffic volume.

Under the normal traffic volume, the presence of ATs at any penetration rate is significant for all measured parameters. Based on the t-tests performed, a penetration rate of 75% results in the most significant decrease in delay, and for all other parameters, 100% ATs platooning were found to be the greatest case.

For a reduced demand of 0.75-times the normal, the average delay and average vehicle speed are not impacted with an increase in ATs penetration rate until 75% compared with the baseline of 25%. A penetration rate of 75% has the most significant impact on average delay and average vehicle speed, however, it is less significant for other parameters, and specifically insignificant on travel time, and therefore, it is recommended that for periods of reduced demand, the penetration rate of platooning trucks be 100%, for the greatest results in delay, travel time, speed and emissions.

Finally, for an increased traffic volume of 1.25-times the normal, only with 100% ATs penetration is there a significant difference in all measured parameters compared to the 25% penetration rate. At 50% relative to 25% ATs, there is no significant difference between the average delays, average speeds, average travel times and CO₂ emissions, only for the other emissions, NO_x, PM₁₀ and PM_{2.5}. With an increase in ATs up to 75%, the average speed remains unimpacted significantly, whereas all other parameters are impacted significantly, however, not to the extent of further increased penetration rates up to 100%.

The results of the t-test significance tests show that greater penetration rates of ATs will be more beneficial in terms of traffic operations and emissions. Under normal traffic volume, the presence of any ATs will have a positive impact, with the most significant impact observed at penetration rates of 75%-100%. During lower traffic volume periods, positive impacts on traffic and emissions occurs between 50%-75% ATs penetration, with the most significant effects with 100% ATs. When traffic volume is increased, there is little difference at lower penetration rates on traffic operations, but greater penetration rates yield significantly positive impacts on both traffic operations and emissions. Therefore, for future increased traffic demand, deployment of ATs should be considered when the penetration rate of these vehicles can be in the range of 75% to 100% for optimal impacts.

Table 26: 1.0 Traffic volume t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
Base	μ	30.34	77.93	139.84	53972.01	143925.37	413785.44	467757.45
	σ	3.56	2.07	3.60	1311.82	3498.18	10057.27	11369.09
1.0, 25% ATs	μ	21.95	83.34	130.77	50933.93	135823.82	390493.48	441427.41
	σ	5.71	3.99	5.91	2056.79	5484.77	15768.70	17825.49
	S _d	0.96	0.78	0.98	18.35	29.97	50.82	54.03
	t	8.71	6.95	9.29	165.53	270.31	458.33	487.30
	Significant	yes	yes	yes	yes	yes	yes	yes
1.0, 50% ATs	μ	18.43	85.79	126.79	49522.83	132060.89	379675.05	429197.88
	σ	5.86	4.22	5.88	2299.78	6132.74	17631.63	19931.41
	S _d	0.97	0.79	0.97	19.00	31.03	52.62	55.95
	t	12.27	9.92	13.40	234.12	382.31	648.24	689.22
	Significant	yes	yes	yes	yes	yes	yes	yes
1.0, 75% ATs	μ	8.00	88.87	119.66	47039.72	125439.24	360637.82	407677.53
	σ	3.22	2.40	3.40	1389.36	3704.97	10651.78	12041.14
	S _d	0.82	0.77	0.84	16.44	26.84	45.51	48.38
	t	27.12	14.16	24.12	421.79	688.79	1167.89	1241.73
	Significant	yes	yes	yes	yes	yes	yes	yes
1.0, 100% ATs	μ	11.16	91.19	118.12	46451.33	123870.22	356126.89	402578.22
	σ	3.88	3.08	3.95	1758.50	4689.32	13481.80	15240.29
	S _d	0.86	0.72	0.87	17.52	28.61	48.52	51.58
	t	22.23	18.49	25.00	429.21	700.89	1188.42	1263.55
	Significant	yes	yes	yes	yes	yes	yes	yes

5.3 Discussion

The optimum ATs platoon configuration, in terms of truck speed and headway with respect to pavement deformation, is determined through uniaxial compressive haversine testing was carried out on cylindrical samples under different loading frequency and rest period combinations. The results of these tests show that the least accumulated permanent deformation is achieved when the loading frequency is the highest, related to a lower loading time and higher speed, as well as with the shortest rest period, for an optimum combination of a 10 Hz frequency, and 0.5s rest period. However, it was also observed that the greatest percentage of recovered strain occurred with the greatest rest period of 1.5 s combined with the larger frequency of 10 Hz. The viscoelastic and rheological properties of asphalt explain the phenomena observed in the results. Short rest periods between loading applications keeps the material in a more elastic state, preventing the relaxation of the material to its viscous state. When in its elastic state, the material has greater stiffness, and therefore, a greater resistance to rutting, resulting in reduced permanent strain at the end of the testing. On the other hand, the longer rest period allows for a greater unloading period of the material and therefore, more percentage recovery of strain is possible. However, the longer rest period also increases the total testing time of the material, and therefore, the total accumulated permanent strain is greater due to the material creep over the longer time.

The results above indicating that a greater frequency, correlated to a higher speed, lead to less accumulated permanent strain is in agreement with the results of previous related research based on frequency and loading time [179-181, 183, 187, 189, 191] as well as speed [147, 179, 182]. Moreover, in terms of vehicle headway or rest period, similar results were found compared to those that also found a positive correlation between a shorter rest period and reduced rest period [151, 191, 192], with one exception from Qi and Witzcak [179], who had observed the opposite result. Additionally, the results found from the pavement testing also compliment the results found by Ramakrishnan et al. and Alrajhi et al. [166, 192] whose results showed that an increase in rest period increased accumulated deformation, but that with a greater rest period, great strain recovery per cycle was possible. Finally, through finite element analysis, Al-Qadi, et al. [155] and Ramakrishnan, et al. [166] concluded that the rest period is ineffective for headways of five feet and ten feet, respectively. This research found that the rest period is significant between 1.5 s (about 42 m or ~137 ft at 100 km/hr) and 0.5 s (about 14m or ~46 ft at 100 km/hr), and therefore, it is not able to verify the claims made by the previous researchers. Further research and experimental validation is required to verify these limits at various speeds.

Once the platoon configuration, with the optimal speed and headway obtained based on asphalt testing and the number of trucks in a platoon determined from previous research, the impact of varying penetration rates of platooning ATs under different traffic volumes was modeled using VISSIM microsimulation to determine the impacts of the optimal configuration on traffic and emissions operations. With an increasing penetration rate of platooning ATs, the average vehicle speed was found to increase, along with a reduced travel time, with the best scenario occurring with 100% ATs. The introduction of ATs will have a significant positive impact on the delay, even at low penetration rates, however, best results for reduced delay are obtained when the penetration rate is 75%.

Elsahly and Abdelfatah [130] found that increased penetration rates resulted in greater vehicle speeds and reduced travel times in unsaturated traffic conditions and Babic, et al. [136] came to similar conclusions for peak hour conditions. Park, et al. [131] found improved conditions for travel time, delay and speed with increased penetration rate, while Rahim [134] observed decreased delay with increasing penetration rate, similar to the results found here, except for the optimal penetration rate for delay. Finally Hassan, et al. [145] observed improved traffic flow in terms of speed, travel time and delay, as well as reduced emissions with increasing penetration rates during off-peak hours, and reduced emissions during peak-hour conditions, while the results of this research show that with benefits to traffic operations are possible during peak hour, with high penetration rates of ATs. The results of this research contradict the conclusions of Sharifillierdy and Calvert et al. [126, 127] who found flow deterioration with increasing penetration rates, especially significant during peak hours, however, Calvert et al. did observe improvements to traffic conditions at 100% penetration rate of AVs. It should be noted that while all the above did consider penetration rates, the previous research has been focused on the penetration rate of AVs, including passenger cars, however the current research only considers ATs and conventional passenger vehicles. As such, it can be concluded that the presence of ATs, compared to general AVs, may have a greater positive impact on traffic parameters compared to considering all vehicles as autonomous.

The emissions are all reduced according to the same trend, such that, overall, with increasing ATs penetration, there is a reduction in emissions, with 100% platooning ATs resulting in the least emissions. Emission reduction strategies, such as vehicle platooning, help the effort towards the targets of lowered GHGs set by governments and agencies. The Government of Quebec set reduction targets for 2030 of 37.5% in GHG emissions relative to the 1990 levels. For the lowest cost alternative to meet this goal, the transportation sector should have its emissions reduced by

50%, and ATs platooning may provide a significant contribution to this goal [218]. Additionally, even though only 1% of $PM_{2.5}$ emissions nationally are caused by the transportation industry [219], reduction of these particular emissions is significant, since the smaller particle size (less than $2.5\ \mu m$) poses a greater threat to health, such as increased cardiovascular and respiratory issues [220], and specifically in the Montreal Region, these particular emissions are found to be higher than WHO recommended levels [221]. The research presented provides emissions reductions strategies through connected platooning trucks, however, does consider EHV, only conventional internal combustion engine trucks. As such, the emissions are likely to be even further decreased than what is presented in this research with the electrification of HVs, further aiding towards emission reduction goals.

Based on the asphalt permanent deformation testing as well as the VISSIM simulation, it is recommended to have autonomous or connected ATs travelling in a platoon during off-peak and current peak-hour conditions, with three trucks per platoon at a headway of 0.5 seconds from one another (or at the least possible spacing), and a desired speed of 100 km/hr. A headway of 0.5 s is significantly below the human reaction time (2.5 s). This configuration implies a reliance on high-reliability V2V communication and automated braking, as it leaves zero margin for human error, and should therefore be applied with high levels of autonomy and greater levels of penetration. During current peak and off-peak hours, any penetration rate of ATs will be beneficial for traffic and emissions, but greater penetration rates will have more significant impacts on these parameters. In the future when the traffic volume is increased, ATs platoons should be carefully considered and should be utilized when the penetration rates of ATs is greater. Previous research found that with increasing penetration of AVs, considering all vehicles as autonomous, that there is deterioration of traffic flow until very high, if not full, penetration of AVs, however, this work demonstrated that if just the trucks are considered for autonomy, the benefits in non-over-saturated conditions are present with lower penetration rates of ATs.

6. CONCLUSION

6.1 Summary of Research

The transportation of freight and cargo is a massive global industry, and with the rapid rise of e-commerce, the demand for trucking is projected to grow exponentially. In Canada alone, trucks move approximately 90% of all freight, accounting for \$65 billion worth of shipments annually. However, the scale of this industry comes with significant drawbacks. Heavy vehicles (HVs) are responsible for 20% of fatal collisions and 6% of all national Greenhouse Gas (GHG) emissions. Furthermore, the sheer weight and volume of these vehicles significantly deteriorate traffic flow and are the primary cause of pavement structural failure. This is particularly acute in Southern Quebec, where high trucking density converges with harsh environmental factors, such as extreme temperature fluctuations and freeze-thaw cycles, to accelerate pavement degradation.

This research explored the viability of Autonomous Truck (AT) platooning as a solution to these multifaceted challenges. By leveraging Vehicle-to-Vehicle (V2V) connectivity, ATs can travel in platoons to reduce aerodynamic drag and improve road capacity. While the benefits to fuel economy and traffic flow are theoretically sound, the impact of these channelized loading patterns on asphalt pavement, specifically regarding rutting, remains a critical concern for infrastructure owners. The primary objective of this thesis was to identify an optimal ATs platoon configuration that minimizes pavement damage while maximizing traffic efficiency and environmental sustainability. This was achieved through a two-phase methodology: (1) Laboratory testing of Quebec-standard asphalt mixtures (ESG-10) to determine the rheological response to simulated platoon loading, and (2) Microscopic traffic simulation (VISSIM) of Autoroute 15 to quantify operational benefits.

6.2 Key Findings

6.2.1 Pavement Performance and Viscoelastic Response

The experimental phase utilized uniaxial compressive haversine loading to simulate the passing of truck axles at different speeds (frequencies) and headways (rest periods). The investigation yielded a counter-intuitive but critical finding regarding the viscoelastic nature of asphalt binder (PG 64E-28).

- **The Frequency Effect:** Results indicated that a loading frequency of 10 Hz (simulating 100 km/h) resulted in significantly lower accumulated permanent strain compared to 8 Hz (80 km/h). This confirms that higher vehicle speeds are beneficial for rutting resistance because they reduce the duration of load application, limiting the time available for the viscous component of the binder to flow and permanently deform.
- **The Rest Period Trade-off:** While literature often suggests that longer rest periods, or headways, are beneficial for material "healing," this study found that for rutting specifically, a **0.5 s rest period** resulted in the least total deformation. Although longer rest periods (1.5 s) allowed for a higher percentage of recovery per cycle, the total test duration and creep effects led to higher overall deformation.
- **Optimal Configuration:** Consequently, the optimal platoon configuration for minimizing rutting on Quebec highways was identified as a **high-speed (100 km/h), short-headway (0.5 s)** formation.

6.2.2 Traffic Operations and Environmental Impact

Using the parameters derived from the lab results (100 km/h, 0.5 s gap), the VISSIM simulation of Autoroute 15 demonstrated that ATs platoons can drastically improve network performance, though the magnitude of benefits depends heavily on the penetration rate.

- **Delay and Speed:** Under current peak-hour conditions, increasing ATs penetration consistently reduced average delay and travel time. At 100% ATs penetration, network delay decreased by over 60%, and average vehicle speeds increased by 14.5%.
- **Thresholds for Benefit:** During off-peak conditions, significant improvements in delay were not observed until ATs penetration exceeded 75%. However, under future demand scenarios (1.25x traffic volume), ATs proved vital in mitigating congestion; a 75% to 100% penetration rate effectively neutralized the negative impacts of the increased traffic volume.
- **Emissions:** The smoothing of traffic flow provided by AT platoons resulted in a linear reduction of emissions. At full penetration, the optimal platoon configuration yielded reductions in CO₂, NO_x, and Particulate Matter (PM₁₀/PM_{2.5}) of approximately 16%.

6.3 Recommendations for Deployment

Based on the triangulation of pavement durability data and traffic flow simulations, the following recommendations are proposed for the strategic deployment of ATs on Quebec's highway network:

1. **Standard Platoon Configuration:** Regulatory bodies and fleet operators should aim for a standard **three-truck platoon** configuration maintaining a constant **0.5 s headway** and a cruising speed of **100 km/h**. This configuration offers the best balance of minimizing asphalt rutting (due to short loading times) and maximizing road capacity.
2. **Peak-Hour Strategy:** During peak congestion hours, ATs platoons should be permitted and encouraged at any penetration rate. Even low adoption rates provide immediate benefits to traffic flow stability and delay reduction.
3. **Off-Peak and Future Demand Strategy:** For off-peak hours, or in scenarios of significantly increased future traffic volume, the benefits of platooning are most pronounced at high penetration rates (>75%). Policy incentives to accelerate fleet renewal and ATs adoption will be necessary to realize these gains.
4. **Infrastructure Adaptation:** Road owners (MTMD) should note that while higher speeds reduce rutting, the channelization of ATs requires asphalt mixes with high stiffness and polymer modification (like the PG 64E-28 tested) to resist the intensified loading cycles.

6.4 Limitations of the Research

While this research provides a robust framework for ATs deployment, several limitations must be acknowledged to contextualize the findings:

- **Material Scope:** The laboratory testing was limited to a single asphalt mix type (ESG-10) and binder grade (PG 64E-28). While this is the standard surface mix for high-volume roads in Quebec, the results may not be directly applicable to binder courses (ESG-14) or base courses (GB-20), nor to rural roads utilizing softer binders.
- **Loading Simulation:** The use of unconfined compressive haversine loading on cylindrical samples serves as a proxy for rutting but does not fully replicate the complex stress state

of a wheel rolling over pavement (which includes shear stresses). Furthermore, the testing protocol simulated continuous platoon passing without accounting for the random, larger gaps that would occur between distinct platoons in mixed traffic.

- **Distress Mechanisms:** The study focused exclusively on rutting (permanent deformation). It did not experimentally evaluate fatigue cracking. There is a potential conflict here: while short rest periods minimize rutting, literature suggests longer rest periods are required to heal micro-cracks and prevent fatigue failure. Therefore, the optimal configuration identified here is optimal for rutting only.
- **Emissions Modeling:** The emissions analysis in VISSIM was calibrated based on conventional internal combustion engine vehicles. Given that the deployment of ATs will likely coincide with fleet electrification, the reported emissions reductions (based on flow smoothing) likely underestimate the true environmental benefits of electric ATs platoons. Conversely, the simulation did not explicitly model the aerodynamic drag reduction of platooning, which would further improve fuel economy for internal combustion engine vehicles.

6.5 Recommendations for Future Work

To build upon this foundational work and support the full integration of ATs into the Canadian transportation network, the following avenues for future research are recommended:

1. **Fatigue vs. Rutting Optimization:** Future studies should conduct fatigue cracking tests, such as 4-point bending beam, using the same frequency/rest period parameters. A multi-objective optimization analysis is needed to find a global optimum configuration that balances the conflicting requirements of rutting resistance (short gaps) and fatigue healing (long gaps).
2. **Lateral Wander Analysis:** This study assumed a fixed lateral position. Future research should incorporate lateral wheel wander into the laboratory testing or Finite Element (FE) modeling to determine if programmed artificial wander for ATs can further mitigate pavement damage.
3. **Electrification Integration:** Traffic simulations should be updated with emission models specific to electric HVs. This would provide a more accurate picture of the GHGs reduction potential of the "Electric + Autonomous" trucking future.
4. **Field Validation:** The findings of this laboratory and simulation study should be validated through controlled field trials, potentially at the Motor Vehicle Test Center (MVTC) in Blainville, Qc. Instrumented pavement sections could provide real-world strain data to verify the laboratory frequency-speed correlations.
5. **Safety and Human Factors:** While the 0.5 s headway is optimal for efficiency, it is well below the human reaction time. Future research must evaluate the safety implications of such tight platooning in mixed traffic, focusing on the interactions between human drivers and automated platoons at merge/diverge points to ensure the theoretical capacity gains do not come at the cost of safety.

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APPENDIX

Appendix A. Trucking

Table 27: Quebec Vehicle Registration Data for 2000 and 2022 [67]

Type d'utilisation	Type de véhicule	n ₂₀₀₀	n ₂₀₂₂
<i>Promenade</i>	<i>Automobile et camion léger</i>	3,446,903	4,993,645
	- <i>Automobile</i>	2,629,106	2,609,579
	- <i>Camion léger</i>	816,997	2,383,835
	- <i>Non précisé</i>	800	231
	<i>Motocyclette</i>	85,447	213,825
	<i>Cyclomoteur</i>	16,046	36,551
	<i>Habitation motorisée</i>	6,295	11,942
<i>Utilisation institutionnelle, professionnelle ou commerciale</i>	<i>Automobile et camion léger</i>	388,086	511,840
	- <i>Automobile</i>	130,493	99,736
	- <i>Camion léger</i>	257,094	412,020
	- <i>Non précisé</i>	499	84
	<i>Motocyclette</i>	1,143	2,399
	<i>Cyclomoteur</i>	160	425
	<i>Habitation motorisée</i>	523	644
	<i>Taxi</i>	8,049	0
	<i>Autobus</i>	6,971**	9,073**
	<i>Autobus scolaire</i>	9,392**	11,411**
	<i>Camion ou tracteur routier</i>	107,678*	175,190*
	<i>Véhicule-outil</i>	27,552	61,681
	<i>Autre véhicule</i>	16,452	24,161
<i>Circulation restreinte</i>	<i>Automobile et camion léger</i>	1,186	1,845
	- <i>Automobile</i>	894	1,112
	- <i>Camion léger</i>	177	674
	- <i>Non précisé</i>	115	59
	<i>Motocyclette</i>	756	2,848
	<i>Cyclomoteur</i>	0	445
	<i>Camion ou tracteur routier</i>	36*	60*
	<i>Motoneige</i>	190	228
	<i>Autre véhicule</i>	107	37

Table 27: Quebec Vehicle Registration Data for 2000 and 2022 [67] Continued

Type d'utilisation	Type de véhicule	n ₂₀₀₀	n ₂₀₂₂
Hors réseau	Automobile et camion léger	6,213	4,820
	- Automobile	1,997	465
	- Camion léger	3,263	4,186
	- Non précisé	953	169
	Cyclomoteur	174	90
	Autobus	152**	167**
	Camion ou tracteur routier	4,466*	5,011*
	Motoneige	148 308	236,973
	Véhicule tout-terrain	230,065	452,913
	Véhicule-outil	147,630	286,476
	Autre véhicule	967	13,905
Total	Total (Véhicule)	4,660,947	7,058,605

*Note: *camion ou tracteur routier* indicates vehicle counts for trucks and road tractors, bolded values were used for vehicle counts displayed in Table 1.

** Note: *Autobus* and *Autobus scolaire* indicate vehicle counts for buses and school buses, respectively; bolded values were used for vehicle counts displayed in Table 1.

Table 28: Vehicle Registration by Fuel Type in Canada in 2023 [74]

Vehicle Type	All	Gas	Diesel	EV	Hybrid	PHEV	Other*
Total, road motor vehicle registrations	25,688,408	23,055,019	1,708,789	329,402	444,630	145,018	5,550
Total, vehicles weighing 4,536 kilograms or less; 5, 6	23,551,759	21,982,395	651,248	327,732	444,564	145,010	810
Passenger cars, weighing 4,536 kilograms or less	8,589,013	8,058,883	104,370	171,742	185,437	68,299	282
Multi-purpose vehicles, weighing 4,536 kilograms or less; 7	9,408,526	8,873,121	94,993	148,935	221,910	69,546	21
Pickup trucks, weighing 4,536 kilograms or less; 8	4,008,583	3,582,985	397,984	5,172	22,370	3	69
Vans, weighing 4,536 kilograms or less; 9	1,541,771	1,464,635	52,857	1,874	14,847	7,162	396
Other vehicles, weighing 4,536 kilograms or less; 10	3,866	2,771	1,044	9	0	0	42
Total, vehicles weighing 4,536 to 11,793 kilograms; 5, 11	681,523	229,158	452,219	84	0	2	60
Total, vehicles weighing 11,794 kilograms or more; 5, 12	576,254	20,022	553,905	..	..	..	..
Buses; 13	65,149	..	..	..	..	..	..
Motorcycles and mopeds	813,723	..	..	..	..	..	..

*Other fuel types include those not mentioned otherwise, such as natural gas, propane, hydrogen, etc.

Table 29: Modified HV-by-Fuel-Type Registration Data [74]

Statistics	All	Gas	Diesel	EV	Hybrid	PHEV	Other*
Total, road motor vehicle registrations	25,688,408	23,055,019	1,708,789	329,402	444,630	145,018	5,550
Total, vehicles weighing 4,536 to 11,793 kilograms	681,523	229,158	452,219	84	0	2	60
Total, vehicles weighing 11,794 kilograms or more	576,254	20,022	553,905	..	..	..	..
Buses	65,149	..	..	..	..	..	..
total HVs (vehicles > 4,536 kg, buses)	1,322,926	249,180	1,006,124	84	0	2	60
%Total Registered	5.15	1.08	58.88	0.026	0	0.001	1.08

0.026% of all vehicles in 2023 are ZEHV, 0.0065% (86/1,322,926) of all HVs are ZEHV.

Taux d'émission de GES et de polluants atmosphériques et consommation de carburant pour un camion lourd¹³ moyen de la grande région de Montréal circulant sur une autoroute urbaine à 7h du matin un jour moyen d'octobre 2019 (correspondant à environ 7°C et 85% d'humidité relative à Montréal)

Polluant	GES ¹⁴ (g/km)	CO (g/km)	COV (g/km)	NO _x (g/km)	SO ₂ (g/km)	PM _{2.5} ¹⁵ (g/km)	PM ₁₀ ¹⁵ (g/km)	Carburant (L/km)
Vitesse (km/h)								
5	4 291	2,568	0,924	8,804	0,014	0,405	0,440	1,618
10	2 450	1,459	0,520	4,801	0,008	0,214	0,232	0,924
15	1 848	1,029	0,360	3,509	0,006	0,152	0,166	0,697
20	1 633	0,831	0,283	2,967	0,005	0,128	0,139	0,616
25	1 524	0,710	0,233	2,652	0,005	0,115	0,125	0,575
30	1 409	0,644	0,203	2,418	0,005	0,107	0,116	0,531
35	1 326	0,593	0,181	2,235	0,004	0,099	0,108	0,500
40	1 270	0,553	0,167	2,092	0,004	0,093	0,101	0,479
45	1 254	0,521	0,156	2,032	0,004	0,089	0,097	0,473
50	1 201	0,488	0,146	1,936	0,004	0,082	0,090	0,453
55	1 075	0,456	0,134	1,764	0,004	0,071	0,077	0,405
60	1 028	0,434	0,127	1,685	0,003	0,065	0,071	0,388
65	1 011	0,416	0,122	1,641	0,003	0,062	0,067	0,381
70	997	0,401	0,117	1,606	0,003	0,059	0,064	0,376
75	975	0,388	0,113	1,567	0,003	0,055	0,060	0,368
80	946	0,375	0,108	1,525	0,003	0,051	0,055	0,357
85	916	0,363	0,104	1,484	0,003	0,046	0,050	0,345
90	897	0,352	0,101	1,455	0,003	0,042	0,045	0,338
95	904	0,341	0,099	1,453	0,003	0,039	0,042	0,341
100	933	0,330	0,098	1,485	0,003	0,038	0,042	0,352
105	971	0,318	0,098	1,534	0,003	0,039	0,042	0,366

Figure 52: GHG Emissions for HVs at Various Speeds in Montreal (October 2019) [79]

Appendix B. Pavement Design- Asphalt Binder

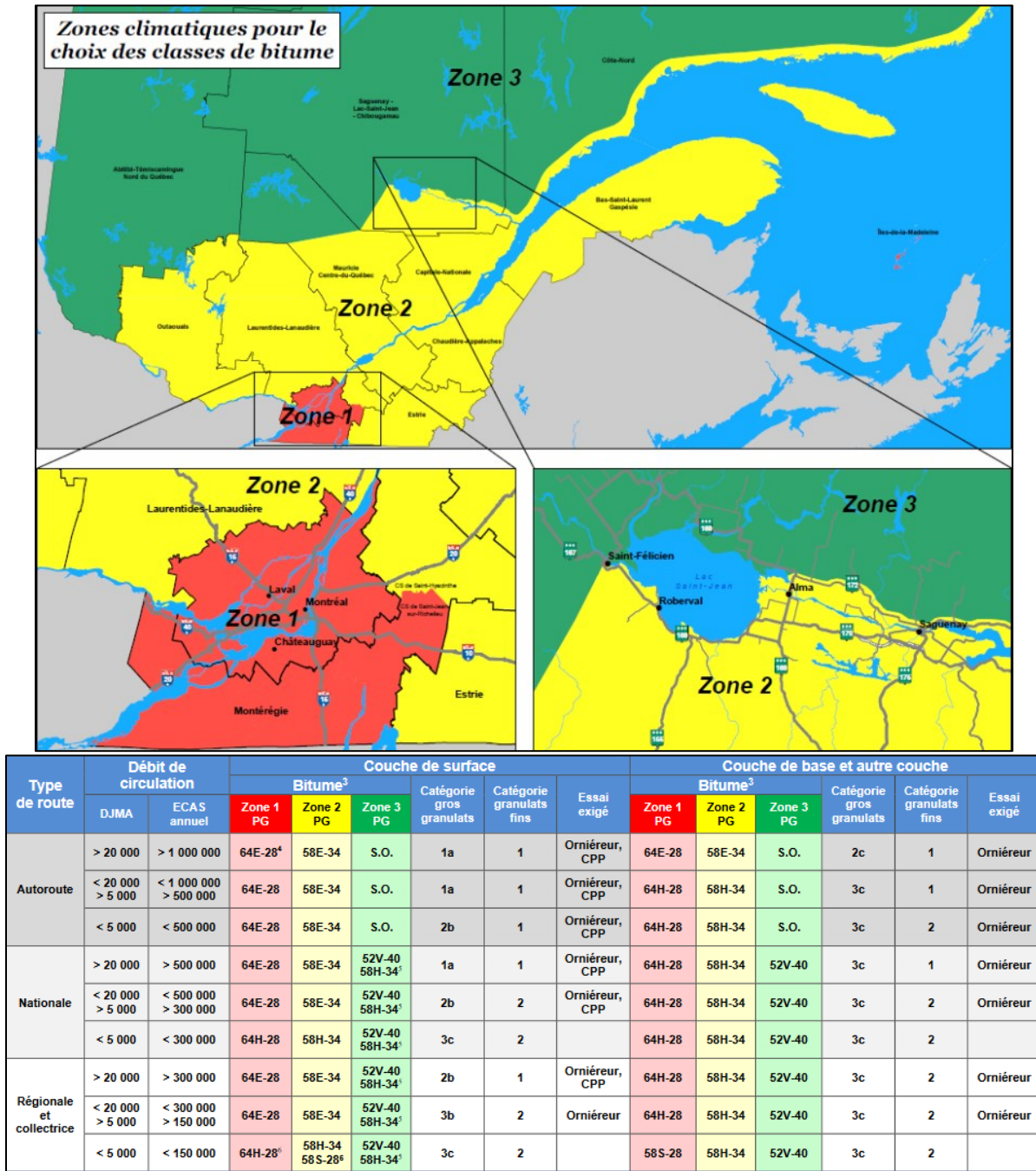


Figure 53: Quebec Climatic Zones for Asphalt Binder Class [222]

Appendix C. Pavement Design-Aggregate

Table 30: Coarse Aggregate Sieving Initial and Final Masses

	Sample 1	Sample 2
Initial Mass (g)	1515.3	1049.4
Mass after Sieving (g)	1510.9	1046.3
% Difference	0.29	0.30

Table 31: Fine Aggregate Sieving Initial and Final Masses

	Sample 1	Sample 2
Initial Mass (g)	361.1	480.3
Mass after Washing (g)	310.2	410.6
Mass Washed Filler (g)	50.9	69.7
Mass Filler Retained (g)	5.3	7.4
Mass Filler (g)	56.2	77.1

Table 32: Coarse Aggregate Sieving Results

	Sample 1			Sample 2			Average
Sieve Size	Mass Retained, Cumulative (g)	% Retained, Cumulative	% Passing	Mass Retained, Cumulative (g)	% Retained, Cumulative	% Passing	% Passing
10	39.7	2.63	97.37	30.8	2.94	97.06	97.21
5	1384.8	91.76	8.24	957.5	91.51	8.49	8.36
2.5	1473.4	97.63	2.37	1020.4	97.52	2.48	2.42
1.25	1484.2	98.34	1.66	1027.0	98.16	1.84	1.75
0.63	1488.8	98.65	1.35	1029.8	98.42	1.58	1.46
0.315	1490.9	98.79	1.21	1031.0	98.54	1.46	1.34
0.16	1491.7	98.84	1.16	1032.0	98.63	1.37	1.26
0.08	1494.5	99.03	0.97	1033.9	98.81	1.19	1.08
<0.08	1509.2	100.00	0.00	1046.3	100.00	0.00	0.00

Table 33: Fine Aggregate Sieving Results

	Sample 1			Sample 2			Average
	Mass Retained, Cumulative (g)	% Retained, Cumulative	% Passing	Mass Retained, Cumulative (g)	% Retained, Cumulative	% Passing	% Passing
10	0	0.00	100.00	0.0	0.00	100.00	100.00
5	10	2.77	97.23	15.9	3.32	96.68	96.96
2.5	122.2	33.87	66.13	162.5	33.88	66.12	66.12
1.25	199.1	55.18	44.82	263.7	54.98	45.02	44.92
0.63	245.1	67.93	32.07	323.1	67.37	32.63	32.35
0.315	272.7	75.58	24.42	360.4	75.15	24.85	24.64
0.16	291.2	80.71	19.29	384.2	80.11	19.89	19.59
0.08	304.6	84.42	15.58	402.5	83.92	16.08	15.83
<0.08	360.8	100.00	0.00	479.6	100.00	0.00	0.00

Table 34: ESG-10 Gradation Specification Limits

Gradation Specifications								
Sieve Size	14	10	5	0.08	0.08	5	10	14
%Passing	100	100	65	10	4	52	92	100

Table 35: ESG-10 Gradation Restricted Zone

RESTRICTED ZONE								
Sieve Size	2.5	1.25	0.613	0.315	0.32	0.613	1.25	2.5
%Passing	46.1	36.70	26.80	18.10	18.10	22.80	30.70	46.10

Appendix D. Pavement Design- Volumetric Design Steps and Calculations

1. Determine combined aggregate bulk specific gravity, 50/50 proportion

$$G_{sb\ coarse} = 2.716$$

$$G_{sb\ fine} = 2.677$$

$$G_{sb} = \frac{\sum P_i}{\sum \frac{P_i}{G_{sb_i}}}$$

$$G_{sb} = \frac{100}{\frac{50}{2.716} + \frac{50}{2.677}} = 2.696$$

2. Estimate initial binder content

$$V_{be} = 12.2\%$$

$$G_b = 1.02$$

$$G_{sb} = 2.696$$

$$P_{ba} \approx \frac{1}{2} \sum P_i * P_w$$

$$P_{ba} \approx \frac{1}{2} * (0.5 * 1.21 + 0.5 * 0.69) = 0.48\%$$

$$P_{bi} = \frac{V_{be} * G_b + P_{ba} * G_{sb}(100 - V_{be})/100}{G_{sb}(100 - V_{be}) + V_{be} * G_b + P_{ba} * G_{sb}(100 - V_{be})/100} * 100\%$$

$$P_{bi} = \frac{12.2 * 1.02 + 0.48 * 2.696(100 - 12.2)/100}{2.696(100 - 12.2) + 12.2 * 1.02 + 2.696(100 - 12.2)/100} * 100\% = 5.42\%$$

3. Perform two G_{mm} tests with a target binder content of 5.42% and 5.42%+0.3% = 5.72%

Sample calculation shown for G_{mm} test with 5.39% binder content

$$G_w = 999.85308 + 6.32693 \times 10^{-2} * T - 8.523829 \times 10^{-3} * T^2 + 6.943624810^{-5} * T^3 - 3.82121610^{-7} * T^4$$

$$G_w = 999.85308 + 6.32693 \times 10^{-2} * 25.7 - 8.523829 \times 10^{-3} * 25.7^2 + 6.943624810^{-5} * 25.7^3 - 3.82121610^{-7} * 25.7^4 = 996.861$$

$$G_{mm} = \frac{M_{Dry}}{M_{Dry} + M_{equip} - M_{Total}} * \frac{G_w}{0.997044}$$

$$G_{mm} = \frac{1089.1}{1089.1 + 5142.4 - 5798.5} * \frac{996.861}{0.997044} = 2.515$$

Table 36: G_{mm} Test for P_b at 5.39%

P_b (%)	5.39	P_b (%)	5.39
Dry Mass A	1089.1	Dry mass B	1054.6
Mass A equip	5142.4	Mass B equip	5142.4
Mass A +water +equip	5798.5	Mass B +water +equip	5777.6
T (°C)	25.7	T (°C)	27.1
G_w	996.861	G_w	996.483
G_{mm}	2.515	G_{mm}	2.513
G_{mm} (average)		2.514	

Table 37: G_{mm} Test for P_b at 5.72%

P_b (%)	5.72	P_b (%)	5.72
Dry Mass A	1108.1	Dry mass B	1073.1
Mass A equip	5156.5	Mass B equip	5168.7
Mass A +water +equip	5821.8	Mass B +water +equip	5813.8
T (°C)	24.7	T (°C)	24.7
G_w	997.120	G_w	997.120
G_{mm}	2.503	G_{mm}	2.507
G_{mm} (average)		2.505	

4. Determine effective aggregate specific gravity

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm} - \frac{P_b}{G_b}}}$$

$$G_{se} = \frac{100 - 5.39}{\frac{100}{2.514 - \frac{5.39}{1.02}}} = 2.75$$

5. Determine actual binder absorbed

$$P_{ba} = \frac{G_{se} - G_{sb}}{G_{se} * G_{sb}} \times 100\%$$

$$P_{ba} = \frac{2.751 - 2.696}{2.751 * 2.696} \times 100\% = 0.749\%$$

6. Determine mass of effective binder

$$M_{be} = \left[\frac{100}{G_{mm}} - \frac{P_s}{G_{sb}} \right] * G_b$$

$$M_{be} = \left[\frac{100}{2.514} - \frac{(100 - 5.39)}{2.696} \right] * 1.02 = 4.68$$

7. Determine effective binder content

$$V_{be} = \left[\frac{100}{G_{mm}} - \frac{P_s}{G_{sb}} \right] * G_{mm}$$

$$V_{be} = \left[\frac{100}{2.514} - \frac{(100 - 5.39)}{2.696} \right] * 2.514 = 11.567\%$$

8. Using interpolation between the two samples, find optimum binder content for 12.2% V_{be}
Equation of trendline for binder content versus V_{be} based on trial binder contents of 5.39% and 5.72%:

$$V_{be} = 2.5486 * P_b - 2.1705$$

$$12.2 = 2.5486 * P_b - 2.1705$$

The interpolated binder content for $V_{be}=12.2\%$ is $P_b = 5.64\%$

9. Repeat G_{mm} test with new P_b to check if V_{be} of 12.2% is reached

Table 38: G_{mm} Test Results and Calculations

P_b (%)	G_{sb}	G_{mm}	G_b	G_{se}	P_{ba} (%)	M_{be}	V_{be} (%)
5.39	2.696	2.520	1.02	2.751	0.749	4.681	11.567
5.64	2.696	2.508	1.02	2.748	0.708	4.972	12.226
5.72	2.696	2.505	1.02	2.748	0.708	5.052	12.408

10. Determine theoretical maximum specific gravity

Sample calculations shown for $P_b=5.64\%$ and corresponding volumetric parameters

$$G_{mm_{max}} = \frac{100}{\frac{P_s}{G_{se}} + \frac{P_{bi}}{G_b}}$$

$$G_{mm_{max}} = \frac{100}{\frac{(100 - 5.64)}{2.571} + \frac{5.64}{1.02}} = 2.508$$

Appendix E. Pavement Design- SGC Compaction

$$V_a = \frac{H_{Ni} - H_{Nmax}}{H_{N80}} \times 100\%$$

$$V_a = \frac{122.8 - 115}{122.8} \times 100\% = 6.35$$

$$VMA = V_{be} * \frac{100 - V_a}{100} + V_a$$

$$VMA = 12.2 * \frac{100 - 6.35}{100} + 6.35 = 17.78$$

$$VFA = \frac{VMA - V_a}{VMA} \times 100\%$$

$$VFA = \frac{17.78 - 6.35}{17.78} \times 100\% = 64.27$$

Table 39: SGC Compaction Results and Calculations

	#Gyrations	Height (mm)	V _a (%)	VMA	VFA
Sample 1	N ₁₀	137.4	16.30	26.51	38.51
	N ₈₀	122.8	6.35	17.78	64.27
	N ₂₀₀	118.4	2.87	14.72	80.49
	N _{max}	115	0.00	12.20	100.00
Sample 2	N ₁₀	155.2	25.90	34.94	25.87
	N ₈₀	138.5	16.97	27.10	37.38
	N ₂₀₀	133.3	13.73	24.25	43.40
	N _{max}	115	0.00	12.20	100.00
Sample 3	N ₁₀	137	16.06	26.30	38.94
	N ₈₀	122.3	5.97	17.44	65.78
	N ₂₀₀	118.5	2.95	14.79	80.03
	N _{max}	115	0.00	12.20	100.00
Sample 4	N ₁₀	137.7	16.49	26.67	38.20
	N ₈₀	123.2	6.66	18.04	63.11
	N ₂₀₀	118.9	3.28	15.08	78.25
	N _{max}	115	0.00	12.20	100.00

Table 40: Limiting Void Values

V _a (N=10)	> 11.0 %
V _a (N=80)	4.0 %-7.0%
V _a (N=200)	>2.0%

Appendix F. Pavement Testing- Sample Preparation

Table 41: Slab 1 Mix Properties

	Batch 1	Batch 2
Mass Coarse (kg)	17.659	17.812
Mass fine (kg)	17.685	17.858
Mass Agg	35.344	35.67
Mass Total	37.457	37.802
Mass Binder	2.113	2.132
Actual Mass Binder	2.122	2.154
Batch Mass	37.466	37.824
Total Mass	75.290	
P _b (%)	5.679	
% CA	49.95	
% FA	50.05	
P _s (%)	94.321	
G _{sb}	2.696	
G _b	1.020	
G _{mm}	2.508	
G _{se}	2.750	
P _{ba} (%)	0.732	
V _{be} (%)	12.267	
P _{bi}	0.057	

Table 42: Slab 1 Cylinder Properties

Cylinder	d ₁ (mm)	d ₂ (mm)	d ₃ (mm)	d _{ave} (mm)	h ₁ (mm)	h ₂ (mm)	h ₃ (mm)	h _{ave} (mm)	V (mm ³)	Density (kg/m ³)
1	76.31	76.49	76.45	76.42	151.09	151.22	151.16	151.16	693256	2.389
2	76.27	76.19	76.09	76.18	152.13	152.18	152.28	152.20	693769	2.411
3	76.63	76.29	76.30	76.41	151.14	151.18	151.25	151.19	693227	2.415
4	76.44	76.19	76.19	76.27	140.2	140.01	140.23	140.15	640351	2.427
5	75.98	76.44	75.65	76.02	141.84	141.92	142.07	141.94	644316	2.452
6	76.15	76.25	76.35	76.25	141.67	141.75	141.53	141.65	646824	2.428
7	76.23	76.34	76.37	76.31	127.57	128.34	128.73	128.21	586440	2.415
8	76.39	76.3	76.39	76.36	127.05	127.62	127.45	127.37	583311	2.419
9	76.6	76.59	76.40	76.53	127.02	126.97	128.18	127.39	585988	2.419

Table 43: Slab 1 Bulk Specific Gravity Test Results

Cylinder	M _{dry} (g)	M _{sub} (g)	M _{SSD} (g)	T (°C)	G _{mb}	V _a (%)	VMA (%)	VFA (%)
1	1656.3	923.4	1657.9	25.10	2.25	10.09	21.12	52
2	1672.6	959.5	1675.2	24.90	2.34	6.82	18.25	63
3	1674.1	987.2	1676.3	24.80	2.43	3.13	15.01	79
4	1554	917.8	1557.1	24.80	2.43	3.07	14.96	79
5	1580	934.4	1581.7	24.80	2.44	2.67	14.61	82
6	1570.7	926.3	1572.5	24.70	2.43	3.08	14.97	79
7	1416.3	834.5	1418.5	24.70	2.43	3.30	15.16	78
8	1411.1	833	1412.9	24.50	2.43	2.96	14.87	80
9	1417.5	837.2	1419.3	24.50	2.44	2.89	14.80	80

Table 44: Slab 2 Mix Properties

	Batch 1	Batch 2
Mass Coarse (kg)	19.095	18.077
Mass fine (kg)	17.113	17.586
Mass Agg	36.208	35.663
Mass Total	38.372	37.795
Mass Binder	2.164	2.132
Actual Mass Binder	2.167	2.140
Batch Mass	38.375	37.803
Total Mass	76.178	
%Binder	5.654	
% CA	51.72	
% FA	48.28	
P _s (%)	94.346	
G _{sb}	2.696	
G _b	1.020	
G _{mm}	2.508	
G _{se}	2.748	
P _{ba} (%)	0.732	

Table 45: Slab 2 Cylinder Properties

Cylinder	d ₁ (mm)	d ₂ (mm)	d ₃ (mm)	d _{ave} (mm)	h ₁ (mm)	h ₂ (mm)	h ₃ (mm)	h _{ave} (mm)	V (mm ³)	Density (kg/m ³)
1	76.7	76.34	76.51	76.52	158.44	158.68	158.38	158.50	728838	2.384
2	76.41	76.35	76.45	76.40	158.19	158.13	157.78	158.03	724541	2.414
3	76.57	76.31	76.32	76.40	160.34	160.05	160.78	160.39	735282	2.419
4	76.42	76.31	77.18	76.64	167.64	168.2	167.42	167.75	773810	2.393
5	76.37	76.4	76.22	76.33	152.36	151.95	152.23	152.18	696367	2.436
6	76.4	76.21	76.72	76.44	150.99	150.9	151.07	150.99	692959	2.431
7	76.22	76.99	76.33	76.51	156.04	156.27	156.09	156.13	717893	2.433
8	76.58	76.36	76.75	76.56	151.37	151.04	150.58	151.00	695183	2.426
9	76.31	76.85	76.62	76.59	151.63	151.1	151.09	151.27	697002	2.428
10	76.3	76.48	76.54	76.44	144.32	144.59	144.51	144.47	663008	2.410
11	75.99	76.36	76.28	76.21	145.65	146.1	146.36	146.04	666156	2.417
12	76.26	76.76	76.34	76.45	145.38	145.42	145.2	145.33	667188	2.429

Table 46: Slab 2 Bulk Specific Gravity Test Results

Cylinder	M _{dry} (g)	M _{sub} (g)	M _{SSD} (g)	T (°C)	G _{mb}	V _a (%)	VMA (%)	VFA (%)
1	1737.9	1017.4	1738.6	25.60	2.41	3.93	15.69	75
2	1749	1029.9	1749.7	25.40	2.43	3.13	14.99	79
3	1778.9	1048.7	1779.4	25.20	2.43	2.94	14.82	80
4	1851.7	1087.8	1852.3	25.20	2.42	3.43	15.25	78
5	1696.5	1003.3	1697.0	25.10	2.45	2.49	14.43	83
6	1684.4	997	1685.1	25.10	2.45	2.40	14.35	83
7	1746.7	1033.9	1747.6	25.20	2.45	2.42	14.37	83
8	1686.2	998.1	1686.7	25.30	2.45	2.37	14.32	83
9	1692.1	1002.5	1692.7	25.30	2.45	2.26	14.22	84
10	1597.8	940.8	1599.2	25.20	2.43	3.24	15.09	79
11	1609.8	946.4	1610.9	25.10	2.42	3.41	15.23	78
12	1620.3	958.2	1620.9	25.00	2.44	2.51	14.45	83

Table 47: Slab 3 Mix Properties

	Batch 1	Batch 2
Mass Coarse (kg)	14.006	22.122
Mass fine (kg)	16.38	20.181
Mass Agg	30.386	42.303
Mass Total	32.202	44.831
Mass Binder	1.816	2.528
Actual Mass Binder	1.816	2.530
Batch Mass	32.202	44.833
Total Mass	77.035	
%Binder	5.642	
% CA	49.70	
% FA	50.30	
P _s (%)	94.358	
G _{sb}	2.696	
G _b	1.020	
G _{mm}	2.508	
G _{se}	2.748	
P _{ba} (%)	0.732	

Table 48: Slab 3 Cylinder Properties

Cylinder	d ₁ (mm)	d ₂ (mm)	d ₃ (mm)	d _{ave} (mm)	h ₁ (mm)	h ₂ (mm)	h ₃ (mm)	h _{ave} (mm)	V (mm ³)	Density (kg/m ³)
1	75.53	75.42	75.92	75.62	153.26	153.1	123.39	143.25	643422	2.664
2	75.48	75.58	75.57	75.54	152.47	152.29	151.99	152.25	682401	2.474
3	76.15	75.97	75.77	75.96	153.33	153.35	154.02	153.57	695977	2.474
4	76.45	75.95	75.80	76.07	148.95	149.29	149.35	149.20	678013	2.461
5	76.25	75.65	76.07	75.99	153.73	154.3	153.6	153.88	697872	2.468
6	75.6	76.04	76.00	75.88	149.77	149.89	150.25	149.97	678186	2.465
7	76.02	76.12	75.93	76.02	156.64	157.11	157.24	157.00	712646	2.438
8	75.93	76.02	75.91	75.95	149.59	150.03	149.65	149.76	678531	2.467
9	76.1	75.77	75.91	75.93	155.08	155.12	155.27	155.16	702504	2.459
10	76.23	75.86	76.02	76.04	151.67	152.36	152.34	152.12	690767	2.444
11	75.33	75.99	75.86	75.73	156.54	156.7	155.67	156.30	703973	2.475
12	75.6	76.12	75.57	75.76	157.64	158.08	158.34	158.02	712394	2.456

Table 49: Slab 3 Bulk Specific Gravity Results

Cylinder	M _{dry} (g)	M _{sub} (g)	M _{SSD} (g)	T (°C)	G _{mb}	V _a (%)	VMA (%)	VFA (%)
1	1713.9	1014.7	1714.2	24.90	2.45	2.30	14.29	84
2	1688.5	994.2	1688.9	24.90	2.43	3.09	14.97	79
3	1721.6	1021.2	1721.8	24.90	2.46	2.02	14.04	86
4	1668.6	987.2	1668.6	24.90	2.45	2.36	14.34	84
5	1722	1019.8	1722.3	24.90	2.45	2.26	14.25	84
6	1671.4	987.2	1671.5	24.90	2.44	2.61	14.56	82
7	1737.6	1022.3	1738.1	24.90	2.43	3.21	15.08	79
8	1673.9	991	1674.2	24.90	2.45	2.31	14.29	84
9	1727.5	1020.8	1727.8	24.90	2.44	2.57	14.52	82
10	1688.4	995.7	1688.6	25.00	2.44	2.84	14.76	81
11	1742	1027.5	1742.0	25.00	2.44	2.79	14.71	81
12	1749.6	1028.4	1749.6	24.90	2.43	3.27	15.13	78

Table 50: Slab 4 Mix Properties

	Batch 1	Batch 2
Mass Coarse (kg)	17.299	17.652
Mass fine (kg)	17.595	17.424
Mass Agg	34.894	35.076
Mass Total	36.980	37.173
Mass Binder	2.086	2.097
Actual Mass Binder	1.950	2.155
Batch Mass	36.844	37.231
Total Mass	74.075	
%Binder	5.542	
% CA	49.95	
% FA	50.05	
P _s (%)	94.458	
G _{sb}	2.696	
G _b	1.020	
G _{mm}	2.508	
G _{se}	2.743	
P _{ba} (%)	0.732	

Table 51: Slab 4 Cylinder Properties

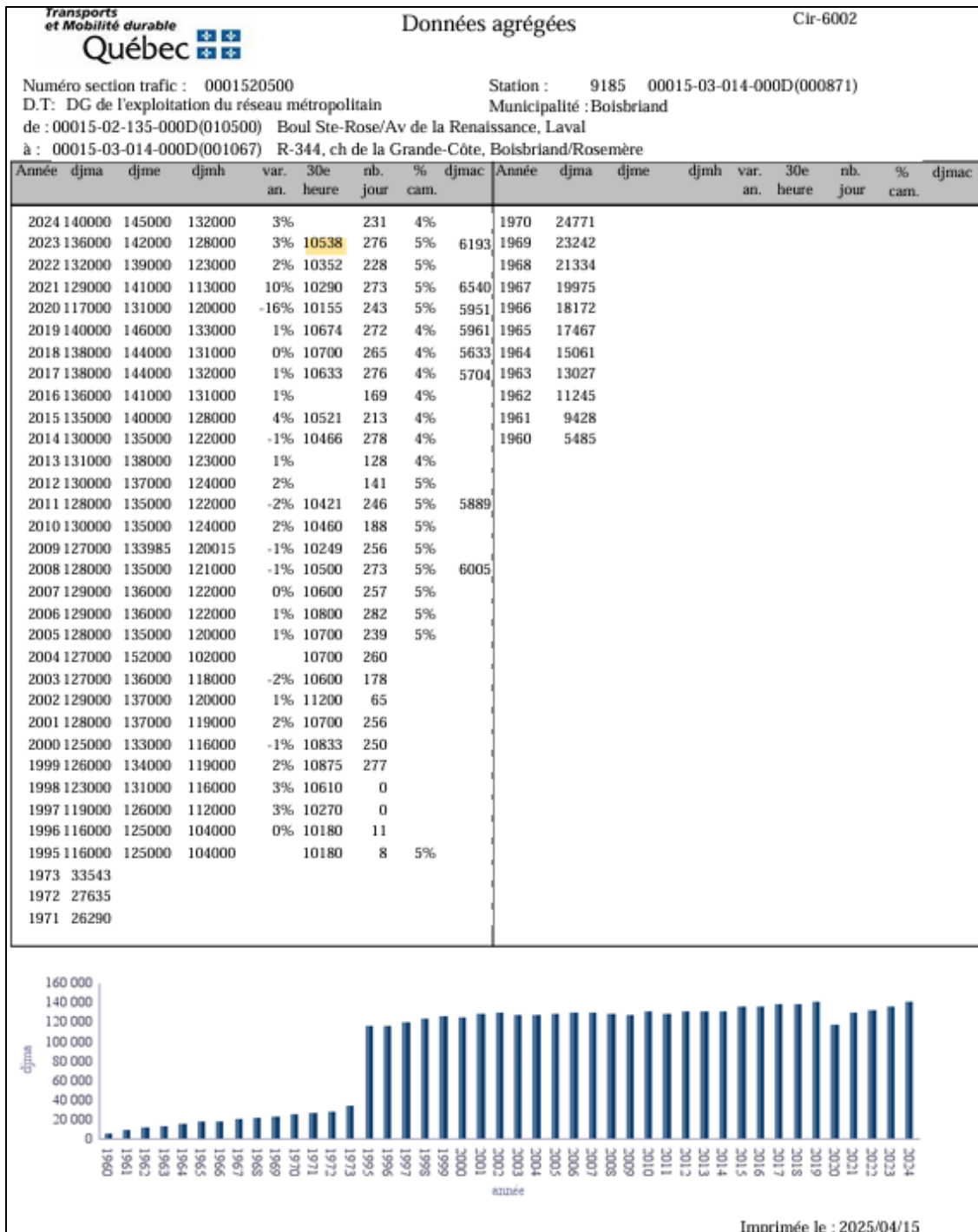
Cylinder	d ₁ (mm)	d ₂ (mm)	d ₃ (mm)	d _{ave} (mm)	h ₁ (mm)	h ₂ (mm)	h ₃ (mm)	h _{ave} (mm)	V (mm ³)	Density (kg/m ³)
1	76.33	76.74	76.41	76.49	150.61	150.11	150.48	150.40	691170	2.398
2	76.24	76.54	76.37	76.38	151.81	151.73	151.67	151.74	695309	2.409
3	76.71	76.69	76.62	76.67	153.78	153.81	154.03	153.87	710464	2.379
4	76.63	76.6	76.32	76.52	152.54	152.54	152.82	152.63	701861	2.356
5	76.36	76.39	76.11	76.29	153.4	153.96	153.62	153.66	702341	2.437
6	76.4	76.26	75.84	76.17	151.96	151.97	152.02	151.98	692494	2.456
7	76.44	76.48	76.24	76.39	152.17	151.61	151.2	151.66	695018	2.423
8	76.47	76.45	76.53	76.48	151.22	151.04	151.06	151.11	694236	2.429
9	76.8	76.62	77.00	76.81	137.98	138.66	138.28	138.31	640812	2.399
10	76.28	76.37	76.24	76.30	134.89	134.82	134.5	134.74	616009	2.421
11	76.24	76.15	76.23	76.21	139.91	140.04	139.46	139.80	637666	2.408
12	76.4	76.12	76.24	76.25	140.01	139.56	139.81	139.79	638402	2.431

Table 52: Slab 4 Bulk Specific Gravity Results

Cylinder	M _{dry} (g)	M _{sub} (g)	M _{SSD} (g)	T (°C)	G _{mb}	V _a (%)	VMA (%)	VFA (%)
1	1657.3	974	1657.2	24.90	2.43	3.28	15.14	78
2	1674.8	983	1674.9	24.90	2.42	3.48	15.32	77
3	1689.9	989.6	1690.3	24.90	2.41	3.84	15.63	75
4	1653.7	963.9	1657.1	24.90	2.39	4.88	16.55	71
5	1711.3	1010.6	1711.3	24.90	2.44	2.62	14.56	82
6	1701	1007.2	1701.0	24.90	2.45	2.24	14.23	84
7	1684.2	993.8	1684.2	24.90	2.44	2.73	14.66	81
8	1686.4	997	1686.6	24.90	2.45	2.49	14.45	83
9	1537.6	906.7	1537.6	24.90	2.44	2.82	14.74	81
10	1491.2	877.7	1491.4	24.90	2.43	3.11	15.00	79
11	1535.2	898.7	1535.4	24.90	2.41	3.86	15.65	75
12	1552.2	914.8	1552.3	24.90	2.43	2.92	14.82	80

Appendix G. Traffic Data

Figure 54 through Figure 61 show the traffic data used in the VISSIM simulation, obtained from open-source data from Transport Quebec. All data is from the interactive map, provided by *Infrastructure Géomatique Ouverte* (IGO), or the Open GIS Infrastructure developed by the *Gouvernement du Québec*. The interactive map provides traffic data, including AADT (denoted as *DJMA* below), top 30th hourly traffic (denoted as *30e heure* below), and truck percentage (denoted as *%cam* below) among others. The highlighted values in the figures below indicate the traffic counts used in the simulation of this research. Data was taken from 2023 as much as possible, with interpolation or assumptions used for missing data. The 30th greatest hourly traffic counts for each year show the amounts for both directions, therefore, a directional split of 0.5 was used to determine the peak hour traffic counts in the North-bound direction.



☰ 🔍 < 2024 DJMA:140000 / DJME:145000 / DJMH:132000 / %c... > ↗ ✕	
Section de trafic	0001520500
Année en cours	2024 DJMA:140000 / DJME:145000 / DJMH:132000 / %cam:4.41 / 30e h:
Année 2	2023 DJMA:136000 / DJME:142000 / DJMH:128000 / %cam:4.55
Année 3	2022 DJMA:132000 / DJME:139000 / DJMH:123000 / %cam:4.71
Année 4	2021 DJMA:129000 / DJME:141000 / DJMH:113000 / %cam:5.08
Année 5	2020 DJMA:117000 / DJME:131000 / DJMH:120000 / %cam:5.09
Année 6	2019 DJMA:140000 / DJME:146000 / DJMH:133000 / %cam:4.27
Année 7	2018 DJMA:138000 / DJME:144000 / DJMH:131000 / %cam:4.09
Année 8	2017 DJMA:138000 / DJME:144000 / DJMH:132000 / %cam:4.13
Année 9	2016 DJMA:136000 / DJME:141000 / DJMH:131000 / %cam:3.86
Année 10	2015 DJMA:135000 / DJME:140000 / DJMH:128000 / %cam:4.31
Description début	Boul Ste-Rose/Av de la Renaissance, Laval
Description fin	R-344, ch de la Grande-Côte, Boisbriand/Rosemère
RTSS début (chaînage)	00015-02-135-000D (10500)

Numéro section trafic : 0001521300

Station : 4615 00015-03-014-000D(004872)

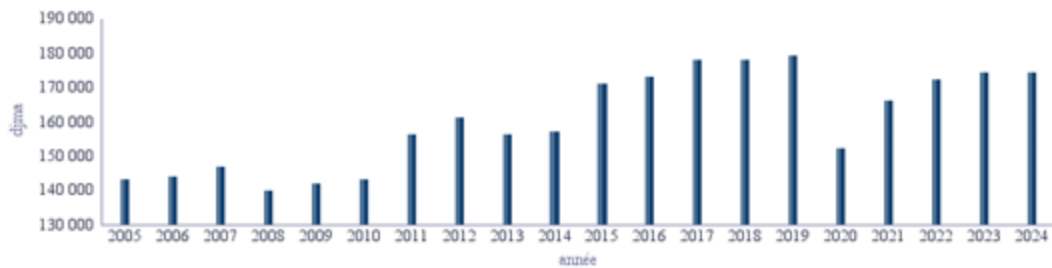
D.T: DG de l'exploitation du réseau métropolitain

Municipalité : Boisbriand

de : 00015-03-014-000D(003928) Bretelles A-640 Ouest

à : 00015-03-014-000D(005192) Bretelles au Sud du ch de la Côte Nord/Rue St-Charles, Sortie 23

Année	djma	djme	djmh	var. an.	30e heure	nb. jour	% cam.	djmac
2024	174000	188000	156000	0%	14100	5	8%	
2023	174000	188000	156000	1%	14100	6		
2022	172000	186000	154000	4%	13900	0		
2021	166000			9%		0		
2020	152000	180000	144000	-15%		18	8%	
2019	179000	193000	161000	1%	14500	0		
2018	178000	192000	160000	0%	14400	0		
2017	178000	192000	160000	3%	14400	18	7%	
2016	173000	187000	155000	1%	14000	0		
2015	171000	185000	153000	9%	13900	18	7%	
2014	157000	170000	141000	1%	12700	14	6%	
2013	156000	169000	140000	-3%	12600	3	7%	
2012	161000	174000	144000	3%	13000	18	7%	
2011	156000	169000	140000	9%	12600	13	6%	
2010	143000	155000	128000	1%	11600	0		
2009	142000	154000	127000	1%	11500	0		
2008	140000	151000	126000	-5%	11400	12	7%	
2007	147000	159000	132000	2%	11900	9		
2006	144000	156000	129000	1%	11700	16	6%	
2005	143000	155000	128000		11600	19	6%	



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Figure 56: Section 2 Traffic Data [224]

☰ 🔍 2024 DJMA:174000 / DJME:188000 / DJMH:156000 / %c... ↗ ✕	
Section de trafic	0001521300
Année en cours	2024 DJMA:174000 / DJME:188000 / DJMH:156000 / %cam:7.7 / 30e h:14100
Année 2	2023 DJMA:174000 / DJME:188000 / DJMH:156000 / %cam:
Année 3	2022 DJMA:172000 / DJME:186000 / DJMH:154000 / %cam:
Année 4	2021 DJMA:166000 / DJME: / DJMH: / %cam:
Année 5	2020 DJMA:152000 / DJME:180000 / DJMH:144000 / %cam:7.7
Année 6	2019 DJMA:179000 / DJME:193000 / DJMH:161000 / %cam:
Année 7	2018 DJMA:178000 / DJME:192000 / DJMH:160000 / %cam:
Année 8	2017 DJMA:178000 / DJME:192000 / DJMH:160000 / %cam:6.6
Année 9	2016 DJMA:173000 / DJME:187000 / DJMH:155000 / %cam:
Année 10	2015 DJMA:171000 / DJME:185000 / DJMH:153000 / %cam:6.5
Description début	Bretelles A-640 Ouest
Description fin	Bretelles au Sud du ch de la Côte Nord/Rue St-Charles, Sortie 23
RTSS début (chainage)	00015-03-014-000D (3928)

Figure 57: Section 2 Open-Source Traffic Data [209]

Numéro section trafic : 0001522000

Station : 9260 00015-03-030-000D(000800)

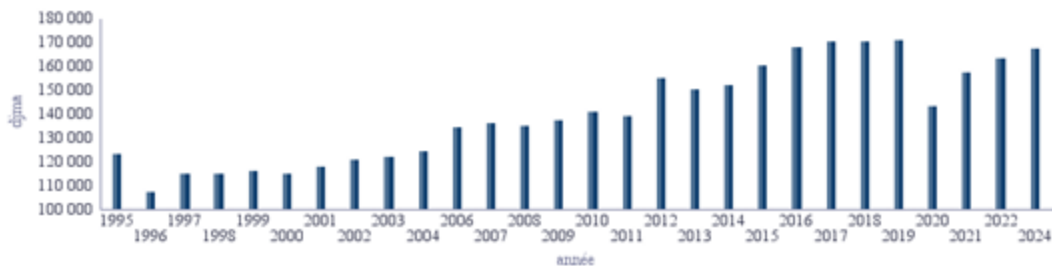
D.T: DG de l'exploitation du réseau métropolitain

Municipalité : Mirabel

de : 00015-03-030-000D(000763) Bretelles au Nord du ch de la Côte Nord/Rue St-Charles, Sortie 23

à : 00015-03-030-000D(001691) Boul de la Seigneurie, sortie 25 (Blainville)

Année	djma	djme	djmh	var. an.	30e heure	nb. jour	% cam.	djmac	Année	djma	djme	djmh	var. an.	30e heure	nb. jour	% cam.	djmac
2024	167000	181000	150000		13500	6	8%										
2022	163000	176000	146000	4%	13200	0											
2021	157000			10%		0											
2020	143000			-16%		0											
2019	171000	185000	153000	1%	13900	19	7%										
2018	170000	184000	152000	0%	13800	0											
2017	170000	184000	152000	1%	13800	0											
2016	168000	182000	151000	5%	13600	17	7%										
2015	160000	173000	144000	5%	13000	3	7%										
2014	152000	164000	136000	1%	12300	0											
2013	150000	162000	135000	-3%	12200	4	7%										
2012	155000	168000	139000	12%	12600	16	7%										
2011	139000	150000	125000	-1%	11400	0											
2010	141000	152000	126000	3%	11400	0											
2009	137000	148000	123000	1%	11200	0											
2008	135000	146000	121000	-1%	11100	12	7%										
2007	136000	147000	122000	1%	11200	6											
2006	134000	145000	120000		11100	14	7%										
2004	124000	134000	111000		10600	0											
2003	122000	132000	109000	1%	10500	0											
2002	121000	131000	109000	3%	10400	0											
2001	118000	128000	107000	3%	10300	4											
2000	115000	124000	103000	-1%	10100	8											
1999	116000	125000	104000	1%	10200	17											
1998	115000	124000	103000	0%	10100	11											
1997	115000	124000	103000	7%	10100	2											
1996	107000	116000	96000	-13%	9720	7	6%										
1995	123000	171000	84000		10540	12	5%										



Imprimée le : 2025/04/15

Figure 58: Section 3 Traffic Data [225]

1 < 2024 DJMA:167000 / DJME:181000 / DJMH:150000 / %c... >	
Section de trafic	0001522000
Année en cours	2024 DJMA:167000 / DJME:181000 / DJMH:150000 / %cam:7.7 / 30e h:13500
Année 2	2023 DJMA: / DJME: / DJMH: / %cam:
Année 3	2022 DJMA:163000 / DJME:176000 / DJMH:146000 / %cam:
Année 4	2021 DJMA:157000 / DJME: / DJMH: / %cam:
Année 5	2020 DJMA:143000 / DJME: / DJMH: / %cam:
Année 6	2019 DJMA:171000 / DJME:185000 / DJMH:153000 / %cam:7.3
Année 7	2018 DJMA:170000 / DJME:184000 / DJMH:152000 / %cam:
Année 8	2017 DJMA:170000 / DJME:184000 / DJMH:152000 / %cam:
Année 9	2016 DJMA:168000 / DJME:182000 / DJMH:151000 / %cam:6.8
Année 10	2015 DJMA:160000 / DJME:173000 / DJMH:144000 / %cam:6.5
Description début	Bretelles au Nord du ch de la Côte Nord/Rue St-Charles, Sortie 23
Description fin	Boul de la Seigneurie, sortie 25 (Blainville)
RTSS début (chainage)	00015-03-030-000D (763)

Figure 59: Section 3 Open-Source Traffic Data [209]

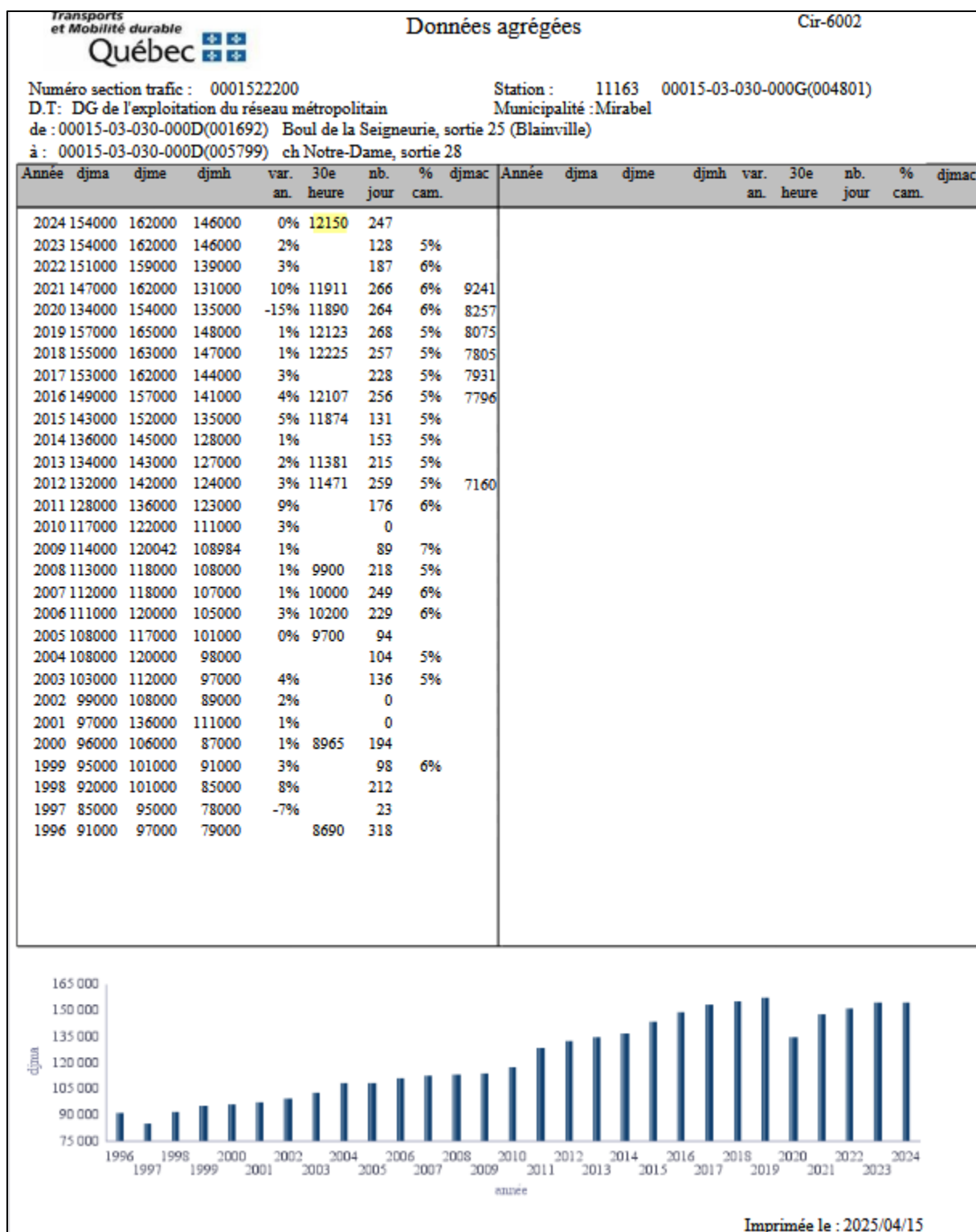


Figure 60: Section 4 Traffic Data [226]

1 < 2024 DJMA:154000 / DJME:162000 / DJMH:146000 / %c... >	
Section de trafic	0001522200
Année en cours	2024 DJMA:154000 / DJME:162000 / DJMH:146000 / %cam: / 30e h:12150
Année 2	2023 DJMA:154000 / DJME:162000 / DJMH:146000 / %cam:5.29
Année 3	2022 DJMA:151000 / DJME:159000 / DJMH:139000 / %cam:6.07
Année 4	2021 DJMA:147000 / DJME:162000 / DJMH:131000 / %cam:6.28
Année 5	2020 DJMA:134000 / DJME:154000 / DJMH:135000 / %cam:6.18
Année 6	2019 DJMA:157000 / DJME:165000 / DJMH:148000 / %cam:5.16
Année 7	2018 DJMA:155000 / DJME:163000 / DJMH:147000 / %cam:5.05
Année 8	2017 DJMA:153000 / DJME:162000 / DJMH:144000 / %cam:5.19
Année 9	2016 DJMA:149000 / DJME:157000 / DJMH:141000 / %cam:5.23
Année 10	2015 DJMA:143000 / DJME:152000 / DJMH:135000 / %cam:5.2
Description début	Boul de la Seigneurie, sortie 25 (Blainville)
Description fin	ch Notre-Dame, sortie 28
RTSS début (chaînage)	00015-03-030-000D (1692)

Figure 61: Section 4 Open-Source Traffic Data [209]

Appendix H. VISSIM Simulation Results

Tables 53 through 65 present the VISSIM simulation results for the baseline scenario and all combinations of traffic volume and ATs penetration rate. The average delay, average vehicle speed, average vehicle travel time, as well as CO₂, NO_x, PM₁₀ and PM_{2.5} emissions are shown.

Table 53: Base Scenario Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	27.61	79.33	1762116	12902	136.58	52863.48	140969.29	405286.7	458150.2
2	28.07	79.6	1785356	12835	139.1	53560.69	142828.51	410631.97	464192.7
3	31.48	77.11	1829112	13015	140.54	54873.35	146328.93	420695.67	475569
4	28.43	78.83	1778419	12976	137.05	53352.58	142273.54	409036.44	462389
5	29.29	78.7	1804912	12926	139.63	54147.35	144392.93	415129.67	469277
6	33.06	76.52	1826890	12804	142.68	54806.71	146151.22	420184.77	474991.5
7	30.78	77.38	1780162	12741	139.72	53404.87	142412.98	409437.33	462842.2
8	25.81	80.72	1734952	12800	135.54	52048.56	138796.15	399038.94	451087.5
9	38.55	73.29	1894096	12779	148.22	56822.86	151527.64	435641.96	492464.8
10	30.3	77.8	1794656	12883	139.3	53839.69	143572.5	412770.95	466610.6
μ	30.34	77.93	1799067	12866	139.84	53972.01	143925.37	413785.44	467757.5
σ	3.56	2.07	43727	89	3.60	1311.82	3498.18	10057.27	11369.09
Min	25.81	73.29	1734952	12741	135.54	52048.56	138796.15	399038.94	451087.5
Max	38.55	80.72	1894096	13015	148.22	56822.86	151527.64	435641.96	492464.8

Table 54: 1.0 Traffic, 25% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	29.25	78.36	1773437	12898	137.50	53203.11	141874.97	407890.53	461093.65
2	21.81	83.46	1689624	12805	131.95	50688.71	135169.90	388613.45	439302.16
3	17.56	86.14	1649095	13125	125.65	49472.84	131927.57	379291.76	428764.60
4	20.26	84.23	1696229	13182	128.68	50886.86	135698.30	390132.62	441019.49
5	24.42	81.67	1738724	12988	133.87	52161.72	139097.92	399906.52	452068.24
6	11.04	91.41	1558363	13035	119.55	46750.88	124669.02	358423.42	405174.30
7	24.25	81.55	1725449	12939	133.35	51763.47	138035.93	396853.29	448616.77
8	23.10	82.40	1687124	12796	131.85	50613.73	134969.94	388038.59	438652.32
9	30.18	77.93	1800945	12931	139.27	54028.35	144075.60	414217.35	468245.70
10	17.60	86.23	1658988	13158	126.08	49769.64	132719.04	381567.24	431336.88
μ	21.95	83.34	1697798	12986	130.77	50933.93	135823.82	390493.48	441427.41
σ	5.71	3.99	68560	138	5.91	2056.79	5484.77	15768.70	17825.49
Min	11.04	77.93	1558363	12796	119.55	46750.88	124669.02	358423.42	405174.30
Max	30.18	91.41	1800945	13182	139.27	54028.35	144075.60	414217.35	468245.70

Table 55: 1.0 Traffic, 50% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	26.08	80.27	1735864	12954	134.00	52075.90	138869.08	399248.60	451324.51
2	9.84	92.33	1548550	12971	119.39	46456.51	123884.02	356166.57	402623.08
3	15.48	87.68	1614946	13103	123.25	48448.39	129195.70	371437.63	419886.01
4	17.98	85.87	1668251	13229	126.11	50047.54	133460.10	383697.78	433745.31
5	18.30	85.79	1653416	13004	127.15	49602.46	132273.24	380285.56	429888.03
6	26.76	80.35	1756150	12932	135.80	52684.50	140492.01	403914.52	456599.03
7	21.01	83.81	1671257	12929	129.26	50137.72	133700.59	384389.20	434526.92
8	9.13	92.60	1506709	12893	116.86	45201.26	120536.70	346543.02	391744.29
9	20.50	84.11	1680039	13007	129.16	50401.16	134403.10	386408.90	436810.06
10	19.21	85.10	1672429	13178	126.91	50172.87	133794.32	384658.67	434831.54
μ	18.43	85.79	1650761	13020	126.79	49522.83	132060.89	379675.05	429197.88
σ	5.86	4.22	76659	113	5.88	2299.78	6132.74	17631.63	19931.41
Min	9.13	80.27	1506709	12893	116.86	45201.26	120536.70	346543.02	391744.29
Max	26.76	92.60	1756150	13229	135.80	52684.50	140492.01	403914.52	456599.03

Table 56: 1.0 Traffic, 75% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	7.46	89.06	1543287	13097	117.84	46298.61	123462.95	354955.99	401254.59
2	5.38	90.96	1534292	12986	118.15	46028.77	122743.38	352887.23	398916.00
3	7.13	89.45	1562212	13198	118.37	46866.37	124976.98	359308.83	406175.20
4	11.19	86.44	1637409	13266	123.43	49122.27	130992.71	376604.05	425726.31
5	6.44	90.06	1558413	13153	118.48	46752.39	124673.05	358435.01	405187.41
6	7.35	89.43	1555903	13031	119.40	46677.10	124472.26	357857.74	404534.83
7	15.75	83.19	1663686	13032	127.66	49910.58	133094.87	382647.76	432558.33
8	6.09	90.28	1515291	12927	117.22	45458.71	121223.24	348516.81	393975.53
9	8.17	88.63	1564179	13063	119.74	46925.36	125134.30	359761.12	406686.49
10	5.06	91.16	1545233	13283	116.33	46357.00	123618.66	355403.64	401760.63
μ	8.00	88.87	1567991	13104	119.66	47039.72	125439.24	360637.82	407677.53
σ	3.22	2.40	46312	119	3.40	1389.36	3704.97	10651.78	12041.14
Min	5.06	83.19	1515291	12927	116.33	45458.71	121223.24	348516.81	393975.53
Max	15.75	91.16	1663686	13283	127.66	49910.58	133094.87	382647.76	432558.33

Table 57: 1.0 Traffic, 100% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	10.04	91.81	1517710	13050	116.30	45531.29	121416.77	349073.21	394604.50
2	6.32	95.19	1485430	12988	114.37	44562.89	118834.37	341648.81	386211.70
3	10.40	91.61	1542982	13182	117.05	46289.46	123438.56	354885.86	401175.32
4	17.14	86.54	1652319	13294	124.29	49569.56	132185.50	380033.30	429602.86
5	6.46	95.01	1492230	13144	113.53	44766.90	119378.39	343212.88	387979.77
6	12.37	90.34	1553274	13030	119.21	46598.22	124261.91	357253.00	403851.21
7	15.21	87.98	1603768	13091	122.51	48113.04	128301.45	368866.66	416979.71
8	7.22	94.31	1474746	12953	113.85	44242.39	117979.70	339191.65	383434.04
9	15.52	87.75	1607462	13078	122.91	48223.86	128596.96	369716.26	417940.12
10	10.88	91.38	1553858	13257	117.21	46615.73	124308.62	357387.27	404003.00
μ	11.16	91.19	1548378	13107	118.12	46451.33	123870.22	356126.89	402578.22
σ	3.88	3.08	58617	112	3.95	1758.50	4689.32	13481.80	15240.29
Min	6.32	86.54	1474746	12953	113.53	44242.39	117979.70	339191.65	383434.04
Max	17.14	95.19	1652319	13294	124.29	49569.56	132185.50	380033.30	429602.86

Table 58: 0.75 Traffic, 25% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	3.12	97.21	1110454	10566	105.10	33313.63	88836.34	255404.49	288718.12
2	3.43	97.22	1129752	10477	107.83	33892.55	90380.13	259842.87	293735.42
3	3.06	97.31	1114542	10598	105.17	33436.27	89163.38	256344.71	289780.97
4	3.40	97.03	1145969	10760	106.50	34379.06	91677.50	263572.80	297951.86
5	3.49	97.06	1144314	10635	107.60	34329.41	91545.09	263192.13	297521.54
6	3.13	97.56	1122642	10570	106.21	33679.25	89811.34	258207.59	291886.84
7	3.57	96.80	1131740	10606	106.71	33952.21	90539.22	260300.25	294252.45
8	2.92	97.39	1099042	10453	105.14	32971.27	87923.39	252779.75	285751.02
9	3.16	97.28	1115487	10553	105.70	33464.61	89238.97	256562.03	290026.65
10	3.28	97.28	1133239	10740	105.52	33997.18	90659.14	260645.02	294642.19
μ	3.26	97.22	1124718	10596	106.15	33741.54	89977.45	258685.16	292426.71
σ	0.21	0.21	15050	98	1.00	451.51	1204.02	3461.56	3913.07
Min	2.92	96.80	1099042	10453	105.10	32971.27	87923.39	252779.75	285751.02
Max	3.57	97.56	1145969	10760	107.83	34379.06	91677.50	263572.80	297951.86

Table 59: 0.75 Traffic, 50% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	3.26	97.14	1101121	10572	104.15	33033.62	88089.66	253257.78	286291.41
2	3.22	97.43	1119101	10479	106.79	33573.04	89528.10	257393.28	290966.31
3	2.92	97.50	1104555	10598	104.22	33136.64	88364.38	254047.58	287184.22
4	3.53	96.89	1137238	10753	105.76	34117.15	90979.06	261564.81	295681.96
5	3.03	97.53	1130911	10641	106.28	33927.33	90472.89	260109.55	294036.89
6	2.82	97.94	1108762	10569	104.91	33262.86	88700.95	255015.24	288278.09
7	3.39	96.94	1119091	10607	105.50	33572.75	89527.32	257391.04	290963.79
8	2.73	97.66	1094102	10457	104.63	32823.08	87528.20	251643.57	284466.65
9	2.95	97.55	1106313	10564	104.72	33189.40	88505.07	254452.08	287641.48
10	3.08	97.52	1123798	10735	104.69	33713.96	89903.88	258473.65	292187.61
μ	3.09	97.41	1114499	10598	105.17	33434.98	89159.95	256334.86	289769.84
σ	0.26	0.33	13780	95	0.89	413.40	1102.40	3169.39	3582.79
Min	2.73	96.89	1094102	10457	104.15	32823.08	87528.20	251643.57	284466.65
Max	3.53	97.94	1137238	10753	106.79	34117.15	90979.06	261564.81	295681.96

Table 60: 0.75 Traffic, 75% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	1.35	93.71	1105244	10575	104.51	33157.33	88419.54	254206.17	287363.49
2	1.57	93.70	1126653	10474	107.57	33799.58	90132.22	259130.14	292929.73
3	1.55	93.57	1122411	10594	105.95	33672.33	89792.88	258154.53	291826.86
4	1.44	93.76	1151605	10759	107.04	34548.15	92128.40	264869.15	299417.30
5	1.30	93.91	1138836	10643	107.00	34165.08	91106.88	261932.28	296097.36
6	1.32	93.95	1127262	10578	106.57	33817.87	90180.98	259270.33	293088.20
7	1.56	93.65	1133863	10608	106.89	34015.90	90709.06	260788.56	294804.46
8	1.24	93.92	1103402	10450	105.59	33102.06	88272.17	253782.48	286884.55
9	1.18	93.90	1109286	10567	104.98	33278.57	88742.86	255135.73	288414.31
10	1.29	93.94	1140794	10745	106.17	34223.83	91263.54	262382.67	296606.49
μ	1.38	93.80	1125936	10599	106.23	33778.07	90074.85	258965.20	292743.27
σ	0.14	0.14	16113	99	0.98	483.40	1289.07	3706.08	4189.48
Min	1.18	93.57	1103402	10450	104.51	33102.06	88272.17	253782.48	286884.55
Max	1.57	93.95	1151605	10759	107.57	34548.15	92128.40	264869.15	299417.30

Table 61: 0.75 Traffic, 100% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	2.82	97.63	1082101	10574	102.34	32463.04	86568.10	248883.28	281346.31
2	2.78	97.89	1097875	10479	104.77	32936.25	87829.99	252511.23	285447.47
3	2.59	97.91	1092871	10614	102.97	32786.14	87429.71	251360.42	284146.56
4	2.87	97.61	1117488	10778	103.68	33524.63	89399.02	257022.17	290546.80
5	2.75	97.82	1111937	10649	104.42	33358.12	88954.99	255745.60	289103.72
6	2.71	98.14	1096684	10581	103.65	32900.52	87734.73	252237.34	285137.87
7	2.77	97.62	1101151	10609	103.79	33034.54	88092.10	253264.78	286299.31
8	2.41	98.00	1072878	10459	102.58	32186.36	85830.28	246762.05	278948.41
9	2.74	97.76	1091121	10567	103.26	32733.64	87289.70	250957.88	283691.51
10	2.79	97.96	1109192	10759	103.09	33275.78	88735.40	255114.27	288390.05
μ	2.72	97.84	1097330	10607	103.45	32919.90	87786.40	252385.90	285305.80
σ	0.13	0.18	13597	103	0.77	407.92	1087.78	3127.36	3535.28
Min	2.41	97.61	1072878	10459	102.34	32186.36	85830.28	246762.05	278948.41
Max	2.87	98.14	1117488	10778	104.77	33524.63	89399.02	257022.17	290546.80

Table 62: 1.25 Traffic, 25% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	36.70	73.96	1985662	13752	144.39	59569.87	158852.98	456702.31	516272.17
2	33.87	75.95	1973496	13765	143.37	59204.89	157879.70	453904.15	513109.04
3	34.66	75.08	1979372	13959	141.80	59381.16	158349.75	455255.54	514636.69
4	35.97	74.35	1992899	13859	143.80	59786.97	159431.93	458366.79	518153.77
5	35.52	74.75	1987156	13831	143.67	59614.66	158972.44	457045.76	516660.43
6	34.17	75.80	1986120	13871	143.19	59583.60	158889.59	456807.58	516391.17
7	38.67	72.92	2009209	13687	146.80	60276.26	160736.69	462117.98	522394.24
8	34.94	75.04	1981465	13844	143.13	59443.94	158517.18	455736.88	515180.82
9	38.05	73.31	1998184	13669	146.18	59945.51	159854.69	459582.23	519527.74
10	35.00	74.98	1990707	13941	142.80	59721.19	159256.52	457862.49	517583.69
μ	35.75	74.61	1988427	13818	143.91	59652.80	159074.15	457338.17	516990.98
σ	1.61	0.99	10112	98	1.52	303.35	808.95	2325.72	2629.07
Min	33.87	72.92	1973496	13669	141.80	59204.89	157879.70	453904.15	513109.04
Max	38.67	75.95	2009209	13959	146.80	60276.26	160736.69	462117.98	522394.24

Table 63: 1.25 Traffic, 50% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	34.47	75.02	1959902	13848	141.53	58797.07	156792.18	450777.53	509574.60
2	33.87	75.93	1972895	13821	142.75	59186.86	157831.62	453765.92	512952.78
3	33.21	76.07	1989455	14126	140.84	59683.65	159156.41	457574.67	517258.33
4	34.63	75.22	1997731	14014	142.55	59931.94	159818.50	459478.20	519410.14
5	33.91	75.63	1987827	14006	141.93	59634.81	159026.17	457200.23	516835.05
6	35.11	75.18	1988439	13866	143.40	59653.15	159075.08	457340.85	516994.01
7	38.21	73.18	2009328	13739	146.25	60279.84	160746.25	462145.46	522425.31
8	35.07	75.05	1992679	13922	143.13	59780.38	159414.35	458316.26	518096.64
9	37.70	73.51	2005005	13771	145.60	60150.16	160400.42	461151.22	521301.38
10	33.66	75.73	1976880	14017	141.03	59306.40	158150.41	454682.42	513988.83
μ	34.98	75.05	1988014	13913	142.90	59640.43	159041.14	457243.28	516883.71
σ	1.68	0.97	14923	125	1.81	447.69	1193.85	3432.31	3880.00
Min	33.21	73.18	1959902	13739	140.84	58797.07	156792.18	450777.53	509574.60
Max	38.21	76.07	2009328	14126	146.25	60279.84	160746.25	462145.46	522425.31

Table 64: 1.25 Traffic, 75% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	30.09	73.94	1958424	13977	140.12	58752.71	156673.90	450437.47	509190.19
2	30.75	74.03	1979481	13843	143.00	59384.41	158358.44	455280.51	514664.93
3	29.86	74.28	1992974	14180	140.55	59789.21	159437.89	458383.93	518173.14
4	32.42	72.81	1999691	13955	143.30	59990.72	159975.26	459928.86	519919.58
5	26.24	76.48	1957575	14167	138.18	58727.24	156605.98	450242.20	508969.45
6	29.28	74.75	1973514	14021	140.75	59205.41	157881.10	453908.17	513113.59
7	34.89	71.40	1992861	13753	144.90	59785.83	159428.87	458358.01	518143.83
8	28.52	75.14	1967709	14094	139.61	59031.26	157416.69	452572.98	511604.24
9	32.61	72.69	1981991	13816	143.46	59459.71	158559.24	455857.81	515317.53
10	28.78	74.98	1975700	14137	139.75	59270.98	158055.96	454410.88	513681.87
μ	30.34	74.05	1977992	13994	141.36	59339.75	158239.33	454938.08	514277.83
σ	2.46	1.45	14397	153	2.15	431.90	1151.74	3311.26	3743.16
Min	26.24	71.40	1957575	13753	138.18	58727.24	156605.98	450242.20	508969.45
Max	34.89	76.48	1999691	14180	144.90	59990.72	159975.26	459928.86	519919.58

Table 65: 1.25 Traffic, 100% ATs Simulation Results

Run	Average Delay (s)	Average Speed (km/h)	Travel time (s)	Vehicles (arrived)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1	32.13	76.42	1945870	14039	138.60	58376.09	155669.58	447550.03	505926.12
2	32.18	76.86	1966140	13975	140.69	58984.21	157291.23	452212.29	511196.50
3	30.95	77.32	1960018	14234	137.70	58800.54	156801.43	450804.12	509604.65
4	32.84	76.19	1984445	14179	139.96	59533.35	158755.61	456422.37	515955.73
5	32.88	76.26	1980607	14117	140.30	59418.20	158448.53	455539.52	514957.72
6	30.02	78.21	1952115	14167	137.79	58563.45	156169.21	448986.47	507549.93
7	33.90	75.52	1966019	13986	140.57	58980.58	157281.54	452184.42	511164.99
8	30.33	77.82	1961286	14218	137.94	58838.58	156902.89	451095.80	509934.39
9	35.18	74.83	1976919	13901	142.21	59307.56	158153.50	454691.32	513998.89
10	34.80	74.95	1982888	14107	140.56	59486.65	158631.07	456064.33	515550.98
μ	32.52	76.44	1967631	14092	139.63	59028.92	157410.46	452555.07	511583.99
σ	1.77	1.14	13281	113	1.53	398.43	1062.48	3054.63	3453.06
Min	30.02	74.83	1945870	13901	137.70	58376.09	155669.58	447550.03	505926.12
Max	35.18	78.21	1984445	14234	142.21	59533.35	158755.61	456422.37	515955.73

Appendix I. t-Test Tables

Table 66: 0.75 Traffic volume t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
0.75, 25% ATs	μ	3.26	97.22	106.15	33741.54	89977.45	258685.16	292426.71
	σ	0.21	0.21	1.00	451.51	1204.02	3461.56	3913.07
0.75, 50% ATs	μ	3.09	97.41	105.17	33434.98	89159.95	256334.86	289769.84
	σ	0.26	0.33	0.89	413.40	1102.40	3169.39	3582.79
	S _d	0.22	0.23	0.43	9.30	15.19	25.75	27.38
	t	0.75	0.85	2.26	32.96	53.83	91.27	97.04
	Significant	no	no	yes	yes	yes	yes	yes
0.75, 75% ATs	μ	1.38	93.80	106.23	33778.07	90074.85	258965.20	292743.27
	σ	0.14	0.14	0.98	483.40	1289.07	3706.08	4189.48
	S _d	0.19	0.19	0.45	9.67	15.79	26.77	28.46
	t	9.98	18.32	0.18	3.78	6.17	10.46	11.12
	Significant	yes	yes	no	yes	yes	yes	yes
0.75, 100% ATs	μ	2.72	97.84	103.45	32919.90	87786.40	252385.90	285305.80
	σ	0.13	0.18	0.77	407.92	1087.78	3127.36	3535.28
	S _d	0.19	0.20	0.42	9.27	15.14	25.67	27.29
	t	2.88	3.15	6.40	88.63	144.73	245.40	260.92
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 67: 1.25 Traffic volume t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.25, 25% ATs	μ	35.75	74.61	143.91	59652.80	159074.15	457338.17	516990.98
	σ	1.61	0.99	1.52	303.35	808.95	2325.72	2629.07
1.25, 50% ATs	μ	34.98	75.05	142.90	59640.43	159041.14	457243.28	516883.71
	σ	1.68	0.97	1.81	447.69	1193.85	3432.31	3880.00
	S _d	0.57	0.44	0.58	8.67	14.15	24.00	25.51
	t	1.34	0.99	1.75	1.43	2.33	3.95	4.20
	Significant	no	no	no	no	yes	yes	yes
1.25, 75% ATs	μ	30.34	74.05	141.36	59339.75	158239.33	454938.08	514277.83
	σ	2.46	1.45	2.15	431.90	1151.74	3311.26	3743.16
	S _d	0.64	0.49	0.61	8.57	14.00	23.74	25.24
	t	8.49	1.14	4.21	36.51	59.62	101.09	107.48
	Significant	yes	no	yes	yes	yes	yes	yes
1.25, 100% ATs	μ	32.52	76.44	139.63	59028.92	157410.46	452555.07	511583.99
	σ	1.77	1.14	1.53	398.43	1062.48	3054.63	3453.06
	S _d	0.58	0.46	0.55	8.38	13.68	23.20	24.66
	t	5.57	3.95	7.74	74.47	121.61	206.21	219.24
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 68: 0.25% ATs t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.0, 25% ATs	μ	21.95	83.34	130.77	50933.93	135823.82	390493.48	441427.41
	σ	5.71	3.99	5.91	2056.79	5484.77	15768.70	17825.49
0.75, 25% ATs	μ	3.26	97.22	106.15	33741.54	89977.45	258685.16	292426.71
	σ	0.21	0.21	1.00	451.51	1204.02	3461.56	3913.07
	S _d	0.77	0.65	0.83	15.84	25.86	43.85	46.62
	t	24.28	21.42	29.61	1085.54	1772.68	3005.73	3195.75
	Significant	yes	yes	yes	yes	yes	yes	yes
1.25, 25% ATs	μ	35.75	74.61	143.91	59652.80	159074.15	457338.17	516990.98
	σ	1.61	0.99	1.52	303.35	808.95	2325.72	2629.07
	S _d	0.86	0.71	0.86	15.36	25.09	42.54	45.23
	t	16.14	12.37	15.23	567.53	926.78	1571.43	1670.77
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 69: 50% ATs t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.0, 50% ATs	μ	18.43	85.79	126.79	49522.83	132060.89	379675.05	429197.88
	σ	5.86	4.22	5.88	2299.78	6132.74	17631.63	19931.41
0.75, 50% ATs	μ	3.09	97.41	105.17	33434.98	89159.95	256334.86	289769.84
	σ	0.26	0.33	0.89	413.40	1102.40	3169.39	3582.79
	S _d	0.78	0.67	0.82	16.47	26.90	45.61	48.49
	t	19.61	17.24	26.28	976.69	1594.94	2704.35	2875.31
	Significant	yes	yes	yes	yes	yes	yes	yes
1.25, 50% ATs	μ	34.98	75.05	142.90	59640.43	159041.14	457243.28	516883.71
	σ	1.68	0.97	1.81	447.69	1193.85	3432.31	3880.00
	S _d	0.87	0.72	0.88	16.58	27.07	45.90	48.80
	t	19.07	14.91	18.37	610.39	996.77	1690.11	1796.95
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 70: 75% ATs t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.0, 75% ATs	μ	8.00	88.87	119.66	47039.72	125439.24	360637.82	407677.53
	σ	3.22	2.40	5.88	1389.36	3704.97	10651.78	12041.14
0.75, 75% ATs	μ	1.38	93.80	105.17	33778.07	90074.85	258965.20	292743.27
	σ	0.14	0.14	0.89	483.40	1289.07	3706.08	4189.48
	S _d	0.58	0.50	0.82	13.68	22.35	37.89	40.29
	t	11.42	9.79	17.62	969.07	1582.49	2683.24	2852.87
	Significant	yes	yes	yes	yes	yes	yes	yes
1.25, 75% ATs	μ	30.34	74.05	142.90	59339.75	158239.33	454938.08	514277.83
	σ	2.46	1.45	1.81	431.90	1151.74	3311.26	3743.16
	S _d	0.75	0.62	0.88	13.50	22.04	37.37	39.73
	t	29.65	23.87	26.49	911.42	1488.35	2523.61	2683.15
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 71: 100% ATs t-Test Values

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.0, 75% ATs	μ	11.16	91.19	118.12	46451.33	123870.22	356126.89	402578.22
	σ	3.88	3.08	5.88	1758.50	4689.32	13481.80	15240.29
0.75, 75% ATs	μ	2.72	97.84	105.17	32919.90	87786.40	252385.90	285305.80
	σ	0.13	0.18	0.89	407.92	1087.78	3127.36	3535.28
	S _d	0.63	0.57	0.82	14.72	24.04	40.75	43.33
	t	13.31	11.63	15.75	919.33	1501.27	2545.52	2706.45
	Significant	yes	yes	yes	yes	yes	yes	yes
1.25, 75% ATs	μ	32.52	76.44	142.90	59028.92	157410.46	452555.07	511583.99
	σ	1.77	1.14	1.81	398.43	1062.48	3054.63	3453.06
	S _d	0.75	0.65	0.88	14.69	23.98	40.67	43.24
	t	28.42	22.70	28.25	856.41	1398.51	2371.28	2521.19
	Significant	yes	yes	yes	yes	yes	yes	yes

Table 72: Summary of t-Test Significance

		Average Delay (s)	Average Speed (km/h)	Average Travel Time (s)	CO ₂ Emissions (g)	NO _x Emissions (g)	PM ₁₀ Emissions (g)	PM _{2.5} Emissions (g)
1.0, 0% (Base)	1.0, 25%	yes	yes	yes	yes	yes	yes	yes
	1.0, 50%	yes	yes	yes	yes	yes	yes	yes
	1.0, 75%	yes	yes	yes	yes	yes	yes	yes
	1.0, 100%	yes	yes	yes	yes	yes	yes	yes
0.75, 25%	0.75, 50%	no	no	yes	yes	yes	yes	yes
	0.75, 75%	yes	yes	no	yes	yes	yes	yes
	0.75, 100%	yes	yes	yes	yes	yes	yes	yes
1.25, 25%	1.25, 50%	no	no	no	no	yes	yes	yes
	1.25, 75%	yes	no	yes	yes	yes	yes	yes
	1.25, 100%	yes	yes	yes	yes	yes	yes	yes
25%. 1.0	25%, 0.75	yes	yes	yes	yes	yes	yes	yes
	25%, 1.25	yes	yes	yes	yes	yes	yes	yes
50%. 1.0	50%, 0.75	yes	yes	yes	yes	yes	yes	yes
	50%, 1.25	yes	yes	yes	yes	yes	yes	yes
75%. 1.0	75%, 0.75	yes	yes	yes	yes	yes	yes	yes
	75%, 1.25	yes	yes	yes	yes	yes	yes	yes
100%. 1.0	100%, 0.75	yes	yes	yes	yes	yes	yes	yes
	100%, 1.25	yes	yes	yes	yes	yes	yes	yes