A Model for Calculating Negative Externalities Experienced as a Result of Marginal Increases In Public Transit and Ridesharing Services on Arterial and Collector Road Segments

by

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Abstract

Increased transit service is widely regarded as an effective solution to traffic congestion, particularly because transit vehicles have larger capacity. Surface transit, however, operates in a different service paradigm, and may preclude the use of space by passenger cars or impose delay on traffic.

By adapting existing delay and queuing models, a model was developed to quantify negative externalities imposed on the network for various conditions found or implied on Fisher Avenue in Ottawa and Taunton Road in Oshawa, and evaluated for realism using VISSIM and AIMSUN. This model may support existing methods for evaluating marginal changes in transit service to maximize traveller throughput. The model extends to autonomous vehicles, ride-hailing services, and performance models.

The model shows that as the proportion of transit vehicles increases, the imposed delay may reduce arterial throughput, reducing road performance and traveller benefit, which can indicate conditions better served by grade-separated transit.

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Nomenclature

Symbol	Description	Units
B	Transit vehicle arrivals per hour	Buses/hour
b_{eff}	Bus Space Efficiency Ratio	--
C_{eff}	Car Space Efficiency Ratio	--
c	Traffic Signal Cycle Time	Seconds
D	Total Transit Vehicle Delay	Seconds
D_{avg}	Average Delay	Seconds
D_{total}	Total Delay on all vehicles	Seconds
D_{nr}	Total Non-Recoverable Delay	Seconds
d_a	Acceleration Delay	Seconds
d_b	Boarding Lost Time	Seconds
d_d	Deceleration Time	Seconds
d_b	Dwell Time	Seconds
d_f	Vehicle Stop Failure	Seconds
d_r	Re-Entry Delay	Seconds
d_{ts}	Traffic Signal Delay	Seconds
Δ_{eff}	Space Efficiency Ratio	--
F_{bus}	Saturation flow adjustment factor	--
F_{transit}	Near-side stop Transit Adjustment factor	--
g_c	Effective green signal	Seconds

h_a	Vehicle Headway at Arrival Flow Rate	Metres
h_d	Vehicle Headway at Departure Flow Rate	Metres
i	Position in platoon or queue	--
K	Coefficient for the effect of transit vehicle loading during green intervals	--
k_a	Vehicle density – arrival flow	Vehicles/km
k_j	Jam Density [1] [2] [3]	Vehicles/km
L	Number of lanes – one direction	Lanes
L_a	Length of available storage space	Metres
L_{pcu}	Space occupied by one passenger car	Metres
N	Number of Through Lanes [4] [2] [3]	Lanes
N_{max}	Maximum number of vehicles in Queue	Vehicles
N_q	Number of Vehicles in a Queue at time t	Vehicles
n	Position in platoon or queue	--
n_{max}	Maximum number of vehicles in queue	--
n_t	Position in platoon or queue – Transit Vehicle	--
o_{pc}	Passenger Car Occupancy	Persons
o_{tr}	Transit Vehicle Occupancy	Persons
q	Flow Rate	Cars/hour
q_a	Arrival Flow Rate	Cars/hour
q_d	Departure Flow Rate	Cars/hour
q_r	Maximum Flow Rate	Cars/hour
R	Delay Recovery	Seconds

R_{total}	Total Delay Recovery	Seconds
T	Average transit dwell time	Seconds
t	Time	Seconds
t_d	Total bus dwell time per hour	Seconds / hour
t_f	Time at Departure	Seconds
t_o	Time at Origin	Seconds
t_{rs}	Time to fill curb lane behind a bus	Seconds
Δt_p	Stopped time in Queue	Seconds
Δt_e	Time to End of Road Segment	Seconds
Δt_q	Time to Arrive at Queue	Seconds
u	Speed	km/hr or m/s
u_f	Free Flow Speed	km/hr or m/s
u_o	Speed at Origin	km/hr or m/s
$\Delta x, x$	Length of road section [1] or distance from origin	Metres
Δx_e	Distance to End of Road Segment	Metres
Δx_{qo}	Distance from Origin to Queue	Metres
Δx_s	Distance from Origin to Stop	Metres

1 Introduction

1.1 Background

In urban areas, the demand for transportation infrastructure is often great enough to exceed the ability of the transportation network to provide high levels of service for the users of the infrastructure. This phenomenon, usually manifested as vehicle traffic congestion, can be observed in nearly every single urban area around the world. This condition occurs because as demand for space and time on the roads increase, interactions between vehicles increase, resulting in reduced overall traffic speeds and increased travel time. Congested conditions are prevalent in most urban areas because transportation planners are generally unable to plan infrastructure to sufficiently address demand across long time horizons, considering changes in demand, technology, and user modal choice. Planning for, or reserving space to provide, necessary infrastructure to support projected demand over an infinitely long time horizon has two significant roadblocks: First, the precision of a forecast declines with increasing projection horizons [5] [6], and secondly, it is an inefficient use of resources [7]. For example, it is unreasonable to build an 8-lane expressway in a currently rural area, and similarly to reserve a 200 metre wide corridor around an existing roadway to protect against increased urban density or traffic increase on some long-term horizon. Further, this approach would increase the geographic footprint of transportation infrastructure and may serve to reduce the propensity of an area to urbanize and densify, among other drawbacks.

Considering it unreasonable to expect planners to pinpoint demand across a long time horizon or provide space for infrastructure to support a worst-case future scenario, congestion in some form is an inevitability in both urban and sub-urban environments. While undesirable at the individual level, congestion serves to balance the level of supply and demand in a transportation system [8]. As congestion increases, the opportunity cost to travel also increases, and drivers choose to optimize their travel choices by trading off time costs, expected accident costs, fuel costs, or even the decision whether to travel at all [9]. This trade off reduces demand for space on the road, and balances the level of congestion with the economic benefit realized through travel action.

Allowing congestion to be managed through social and economic forces has an undesired effect, however. In the presence of congestion, the price at which supply and demand are balanced increases, thus eliminating travel activities with lower utility. This utility, measured as the difference between the monetized benefit of travel and the monetized cost of travel, becomes negative for lower economic benefit activities as the cost to travel increases. Specifically relating to equality, increased congestion and the corresponding increase in travel costs disproportionately penalizes those with lower incomes, and may reduce the number of trips taken for social and recreational activities. Travellers who receive the highest benefit from space on the infrastructure thus become the ones who consume the majority of the space. As an example, when congestion is increased network-wide, a traveler who commutes a long distance for a large salary may continue to commute, while a traveler who commutes the same distance for a modest salary may self-deselect from their employment or choose to relocate if travel costs are sufficiently high. Only the activities that result in a net positive economic gain for an

individual will remain on the transport network, while those with a net negative economic impact will modify their behaviours accordingly.

Some proposed solutions to traffic congestion involve increasing the overall cost to travel, through such mechanisms as road tolls (such as on Highway 407 in Toronto), or congestion fees (such as in London, England). These solutions further reduce accessibility for travellers by increasing the overall cost to travel. Other solutions include increasing capacity by adding lanes, using traffic control devices, or implementing transit service. These solutions can be effective at increasing capacity, and also reduce the overall cost to travel, promoting economic activity and equality if implemented correctly, though there are two drawbacks. Firstly, these solutions rely on the availability of sufficient space and infrastructure to implement, and secondly, by reducing the overall cost to travel, more trips are generated, limiting the local effectiveness of the solution.

Proponents of transit as a solution to traffic congestion argue that because transit vehicles are able to carry more passengers and take less space on the road, they are an effective solution to traffic congestion. Likewise, proponents of ride-sharing and autonomous cars argue that by having vehicles work in a more coordinated and effective way, they are an effective solution to traffic congestion. Since these types of vehicles operate with a different set of behaviours (different paradigm), there are interactions between these vehicles and automotive transit that result in reduced speeds and throughputs on urban roads. Historically, however, the number of transit vehicles and ride-sharing vehicles have been insignificant enough when compared to automotive traffic flow, that transit and traffic have been analysed separately or indexed using

weighting factors. As these types of vehicles become more pervasive, and represent a greater proportion of the overall traffic flow, especially in dense urban areas, the interactions between them and automotive traffic become more pronounced and could actually reduce overall traveller throughput.

1.2 Problem Definition

The prevailing method for addressing congestion in the absence of available space for widening roadways is primarily concerned with the re-allocation of demand from automobiles to transit and/or ridesharing programs. Justification for projects often project a demand shift to transit and ridesharing, and calculate a corresponding reduction in automobile trips in the study area, resulting in a reduction in total vehicle space demand on a network. Current best practices in transportation engineering with respect to the implementation of transit or ridesharing consider changes at the macroscopic level, projecting the effects of proposed changes by corridor or zone, driven by politically-generated projects. These practices extrapolate future traffic conditions in various scenarios against extrapolated future traffic conditions over some lengthy period of time to consider a reduction in vehicle kilometers travelled or a reduction in generalized travel times. Given the complexity of our transportation networks and the available computing power, studies should consider the impact of a marginal change in transit, ridesharing, and other services with respect to the marginal person throughput on a link, marginal decrease in travel time on a link, or the marginal user benefit. Further, by considering the impact of these changes at the microscopic level, combined with an increase in the pervasiveness of devices that can measure and report traffic in real time, we should be able to implement directed improvements to

infrastructure and transit in a directed manner to achieve specific goals with regards to reducing congestion, promoting equality, and getting best value-for-money.

1.3 Objectives and Scope

The primary objective of this thesis is to develop, from first principles of transportation engineering and economics, an analytical microscopic model that can be used to validate the efficiency losses, if any, experienced by the network due to the implementation or augmentation of transit service or ridesharing services in various forms, in response to the Victoria Transport Policy Institute's assertion that 'under typical conditions buses impose congestion costs equivalent to 1.5 cars on highway and 4.5 cars on surface streets' [24] This research has the following specific objectives:

1. Propose a methodology for determining efficiency losses due to the presence of transit service in various forms along arterial roads;
2. Develop analytical model(s), from first principles, for quantifying the network efficiency losses related to the provision of or increase in transit service in various forms;
3. Compare the results from the developed analytical model to the assertions of the Victoria Transport Policy Institute; and
4. Determine boundary conditions for situations where we would expect the marginal change in transit service to have a positive, negative, and neutral effect on both congestion and user benefits, using this methodology, and discuss the implications of these results.

The scope of this research is limited to the following:

1. Reduction in network efficiencies are being modeled for mixed-use arterials, without an existing dedicated transit right-of-way;
2. The analytical model(s) will only consider the interaction between transit vehicles and traffic. Additional considerations, including truck, cycling, and pedestrian traffic, will be left for future research. Validation of real-world data must consider some imprecision due to these considerations.
3. Only basic traffic control strategies are considered. Any intelligent transportation systems including Transit Signal Priority and Dynamic Signal Timing are excluded from the analysis.
4. The analytical model aims to calculate efficiency losses along each segment of an arterial road individually, and not for the entire network. Evaluating overall network performance using this method is reserved for further research and professional applications.

1.4 Methodological Framework

This research is concerned with developing a micro-simulation approach to evaluating the efficiency losses, if any, associated with the implementation of an increase in transit or ridesharing services on individual arterial links in urban areas. The performance measures predicted include traveller throughput, travel time, and cost to travel.

As part of the literature review process, information was gathered about transportation economics and reactions to supply and demand, current practices in transportation planning, previous and related research on the impacts of transit-traffic interactions, as well as related research, including work-zone traffic impacts.

The model has been developed using first principles from both the Highway Capacity Manual and the Transit Capacity and Quality of Service Manual, and is applied to one-lane and two-lane arterials in urban areas with curb-side stops. Sample traffic counts are utilized in order to generate predictive results, and any other quantitative inputs to the model are derived from the literature review process. Data for the base case models was obtained from a combination of published reports and publicly available data. After the models were developed and the results available, they were analyzed using two approaches:

1. Comparison of the results of this predictive model to publicly available impacts, including those from the Victoria Transport Institute and the Highway Capacity Model;
2. Evaluating the results to determine the circumstances under which we would expect a marginal change in transit service to result in a net decrease in cost (and increase in traveller throughput), and where we expect it to be counterproductive.

Graphically, the methodological framework is as follows:

Figure 1: Methodological Framework

1.5 Thesis Contribution

This research is unique when compared with previous studies and current best practices in transportation engineering. First and foremost, minimal research has been performed previously on the effects of interaction between transit/ridesharing and automotive traffic with respect to travel times and user benefits. This model may provide an initial step into more comprehensive analysis that could include other modes of transport as well.

Further, where macroscopic models consider the impacts on a region using aggregate data, and microscopic models currently consider the impacts on a small region, this model will permit analysis at the link-node level. As data collection techniques and the Internet of Things (IoT) allows us to do calculations using real-world data in near real-

time, this type of model could be used to generate maps showing particularly congested links, and using business intelligence, be able to make decisions about where best to spend money on infrastructure improvements. Additionally, when data can be collected directly, instead of generalized, we can eliminate negative externalities that remain when averages or statistical distributions are utilized.

1.6 Thesis Organization

Chapter 2: Literature Review – In this section, published material concerning traffic engineering, transportation economics, and other short-term traffic disruptions are reviewed and presented. Any challenges and issues facing transportation engineers when it comes to analyzing traffic performance and recommending particular project options is presented. Previous studies done to quantify the interaction between transit/ridesharing and transit are also discussed.

Chapter 3: Model Development – This chapter builds, from first principles a micro-simulation model that will be used to try to quantify these marginal impacts. Each component of the model is discussed in terms of its inputs and outputs and sources of information are given. Since sufficient data is not yet available to be able to execute the model, calibration parameters and process are described, but left for future research, as each calibration parameter will be location-specific.

Chapter 4: Results – The results from each scenario modelled are summarized and presented in this section, with trends, differences, and key observations from each result highlighted.

Chapter 5: Discussion – The results are discussed further, particularly in comparison to other methodologies in this section. Furthermore, a detailed comparison of this methodology compared to other methods of quantifying this information are presented

Chapter 6: Conclusions – Based on the results and previous discussion, specific conclusions about expected results are presented, as well as future research opportunities and objectives, including a section on how to utilize Internet of Things technology to implement this model.

2 Literature Review

2.1 Transportation Economics

2.1.1 Supply and Demand

In economics, a demand curve illustrates the impact of price on the quantity of a good demanded by the public. When the price is low, typically consumers demand more of a good than when the price is high [10]. Figure 2 illustrates a general demand curve for a single good or service.

Figure 2: General Demand Curve for a Single Good or Service

The same principles apply to transportation economics. On the demand side, when the cost of travel increases, either due to an increase in automobile acquisition costs, operating costs, or travel time, the quantity, or number of trips taken by travellers is decreased.

Similarly, on the supply side, there is an opposing curve called the supply curve. The supply curve illustrates the relationship between the price of a good and the quantity supplied by manufacturers of a good or service [10]. Figure 3 illustrates both a supply and demand curve. The point at which the lines cross represents the equilibrium price and quantity for that good. At that point there is no longer a surplus or a shortage of goods – the free market forces have exactly matched the amount of a good that consumers want to purchase with the exact number of goods that manufacturers are willing to sell [10].

Figure 3: General Supply and Demand Curves for a Good or Service

These principles also apply to transportation infrastructure. As the supply of a good is increased, manifested as an increase in the total passenger throughput in the road, the price of that good, manifested as travel times, declines. Thus, we can expect that if travel times are minimized, the demand for time and space on the infrastructure will increase. This phenomenon is known as latent, or induced demand [11].

These supply and demand curves can either represent a particular user buying a quantity of good or service and a particular seller selling a quantity of a good or service, in a one-to-one relations, or it can represent an entire market, where each user's willingness to pay for a product is a point on the curve, and each supplier's willingness to sell is another point on the curve. When viewing this as a representation of the entire market, we can see the parallels to transport planning. If the maximum price for a particular user, as a point on the graph, is higher than the market's willingness to sell, then that user decides not to use that service. Alternately, if the maximum price for a particular user is higher than the market's willingness to sell, then the user uses that service and has an economic surplus. As an example, if a traveller is willing to pay up to \$10 in total travel costs to commute to work, and their travel costs to commute to work are \$8, that user will choose to travel and have \$2 in economic surplus remaining. Likewise, if another traveller is only willing to pay \$6 to make the same trip, then they will decide not to make the trip, and avoid a \$2 additional cost.

Each user of a transportation network is inherently assigning value to a particular trip. This value can be real, as in the case of travel for the purposes of employment; or this value can be assigned arbitrarily, as in the case of travel for social reasons. A particular traveller will generally make the decision whether or not to travel based on the difference of their individually perceived positive values and the direct aggregate cost required to complete the trip. Much like the market for a single good, demand will adjust dynamically for each user as the aggregate price changes.

Figure 4 illustrates an economic concept known as economic surplus. The purple section shown represents the total economic surplus received by both buyers and sellers of a

particular good. In particular, the area below the demand curve and above the price represents the consumer surplus for a good, while the area above the supply curve and below the price represents the producer surplus for a good. The larger this area, the greater the benefit to both buyers and sellers is in a transaction [10].

Figure 4: General Economic Surplus for a Single Good or Service

Now that these concepts are developed, consider that the supply on a road is not variable. The amount of space available on a road is fixed and supply is generally inelastic. This results in a supply curve that is vertical or nearly vertical. The result of this is that there is little to no producer surplus, and indeed, the government should not be in the business of making money from the services it provides. Next, it is a direct objective of the transportation network to maximize consumer surplus – that is, a government aims to have its citizens receive the greatest possible benefit from the provision of road infrastructure, transit services, or any other supply. See Figure 5. This graph represents what occurs when supply is increased. At “supply1”, the consumer

surplus is the area of the upper triangle. When the supply is increased to the level shown as “supply2” an additional consumer surplus is realized and would-be travellers receive more benefit from trips they make, as well as choose to make more trips. This is a desirable outcome.

Figure 5: General Consumer Surplus Change for an Increase in Supply

As a result of these market factors, it can be seen that it is advantageous for a government to attempt to maximize supply on a road network in terms of passenger throughput, for the purposes of fostering economic development and consumer welfare.

2.1.2 Managing Congestion Through Supply and Demand

In a trade equilibrium, supply and demand are balanced using market forces, and the price of a particular good serves as the balancing reagent. With public road networks, supply and demand are currently balanced using market forces, but the combination of price and time serve as the balancing reagent. As with many public services, however, there is political pressure to ensure that all members of society have equal access to the

service. As congestion, and therefore the cost of travel, increases, the public begins to demand that something be done to reduce the price so it is accessible for all. In this scenario, there are two prevailing methods for addressing congestion on the network: reduce demand, or increase supply.

Demand reduction methods can be effective at reducing localized demand on problematic sections of the network through re-distribution of traffic flow. Examples of demand-reduction methods include road tolls and metered on-ramps. Other methods that increase travel time or price for some or all users of the network would be considered demand reduction strategies as well. There are challenges related to using a demand reduction strategy, however. If a demand-reduction strategy can be implemented equally for all road users, either as a flat rate or as a cost-per-use strategy, congestion will be reduced due to an increase in the cost of each trip and a corresponding reduction in the total number of trips. Metered on-ramps serve to reduce the time-cost spent travelling along a highway, however, it increases the time-cost spent waiting to access the highway. This delay encourages those who travel shorter distances on the highway to find alternate routes, since ramp metering provides unequal benefits that favour the long-distance traveller [7]. Tolls can be effective for re-balancing congestion across the network where congestion is typically localized, however, this also does not improve the total price to transportation users. Lastly, most demand reduction methods in practice today are designed to address congestion on limited-access highways, and as a result, options for reducing congestion on arterial roads are limited. In summary, demand reduction methods generally increase the deadweight losses on the economy from an overall transportation benefits view by marginally increasing the overall cost to travel

for all or some users of the network, and since they are most often implemented on limited-access highways, they often increase congestion on arterial roads.

Conversely, methods for increasing supply are typically effective at reducing congestion and lowering the cost to travel, often at a network-level. Supply increases, or an increase in capacity, allow more travellers to use that path, rebalancing the supply and demand equilibrium and reducing cost to travel. Examples of supply increases on arterial roads include lane-widening or addition, signal re-timing, queue management, right/left turn channelization, reversible lanes, and truck-only lanes [12]. Intelligent systems like Variable Message Signs or real-time in-car information can serve to increase the capacity of the network overall or mitigate against demand fluctuations, but have less effect locally. Supply increases, however, tend to increase the total amount of trips taken in a particular study area as a consequence of reducing the cost to travel for all users, while reducing local congestion. Another drawback is that if the supply increase affects passenger cars, the corresponding increase in Vehicle Kilometres Travelled (VKT) can have adverse social and environmental effects, which are difficult to quantify precisely.

Urban planners and transportation engineers are frequently tasked with the challenge of designing infrastructure to reduce, limit or maintain levels of congestion on city streets as cities expand and densify. Historically, the most common solution to a local congestion problem has been road widening, especially in suburban or developing areas where traffic levels increase dramatically as areas develop and space is available to do so. In practice, however, road widening does not have the projected effect. As the capacity of the road is increased, the travel time along the road is reduced, and each user non-cooperatively seeks to minimize his cost of transportation [12]. This observation,

commonly known as Wardrop's first principle, reduces the effectiveness of the local road widening by reducing travel times along this corridor, and users who would use a different path at the previous equilibrium will unilaterally minimize their transportation costs, where possible, by shifting paths to the new, widened road. In short, a localized increase in road capacity as a solution to congestion results in a small improvement in level of service in all corridors on a similar path as the network approaches equilibrium, limiting the benefit realized by the additional capacity. Further to this, at a network level, roads where demand exceeds supply by the largest proportion tend to restrict the ability of the rest of the network to realize the benefit of additional capacity. Clearly, if a network link is beyond capacity, and thus developing a queue, it can be seen that adjacent network links will have their throughput reduced as well.

Considering the network performance effects and equality implications of demand-reducing and supply-increasing strategies, it is clear that supply-increasing strategies provide the most benefit if they can be implemented.

2.2 Traffic Congestion

2.2.1 Transit and Ridesharing as a Solution to Congestion

More recently, the provision of ridesharing / transit service has been touted as a solution to address road congestion issues caused by excessive demand for space on the roadways [13] [14]. This solution has been proposed because of two main assumptions. Firstly, transit vehicles, whether standard bus, articulated bus, light rail, or heavy rail, are all able to transport significantly more commuters, while consuming a smaller quantity of road space [15]. Secondly, it is assumed that the introduction of transit

service will not affect the network equilibrium in any meaningful way; that the number of trips taken per person remains constant throughout the analysis. In fact, economic justification for the provision or increase of transit services is typically based on economic gains from a reduction in traffic, or more specifically, travel times. These gains are calculated as a reduction in travel time for vehicles along certain corridors, multiplied by their corresponding average value of time [16]. If used by the public, the provision of transit services appears to address a number of the shortcomings that prevent local road widening from being effective. Transit users take a fixed route from one place to another, restricting a road user from autonomously selecting a path of higher speed and lower cost. Additionally, transit users have a nearly fixed cost for travel, meaning that the price elasticity of demand for transit for discretionary users is relatively inelastic [17]—travellers using transit do not typically make more trips due to the reduced cost of travel when congestion is reduced.

While it is true that transit vehicles are able to transport significantly more commuters in a smaller space, transit commuters tend to take fewer automobile trips, and the variability in travel trip times are limited, this represents only a portion of the scenario. Even if we consider that the total number of trips made by all road users may be static or near-static, transit vehicles operate on a different travel paradigm than automobile traffic. Unlike passenger cars, transit vehicles make regular stops to allow passengers to board or alight, may travel at a different speed than the rest of traffic, and have a different set of performance characteristics, especially with regards to acceleration, deceleration, and turning radius [2]. This difference in paradigm results in a set of competing priorities between automotive traffic and transit traffic. Since these two types

of road vehicles have competing priorities, yet often share infrastructure, occasionally the priorities of one type will negatively affect the other, if even for a short time. If these interactions are sufficiently destructive to traffic flow and occur at a sufficiently high frequency, we would expect that the performance of one or both will be reduced. The number, type, and frequency of these interactions and the overall effect multi-modal service has on arterial roads is not well understood and nearly impossible to determine analytically; rather, these interactions are typically measured on aggregate, and indirectly, using observations related to the cessations of existing public transit service, or, by using an aggregate transport system model. [18]

Illustration 1 shows an often used graphic (and there are many more that provide similar information) advertising that transit service and associated infrastructure is a good solution to solving the problem of traffic congestion. This particular graphic was used by the City of Charlotte, NC to help gain support for a light rail line, and they indicate on the page that transit alone is not in itself a solution to congestion. This information has been re-used numerous times, however, by many other proponents of transit to make the case for transit exclusively. To provide support for funding of transit initiatives, assumptions are made which are quite optimistic for the business case for transit. The first is that each commuter vehicle is carrying a single passenger, whereas in reality, these vehicles may often carry just a single passenger, but occasionally will carry multiple passengers. Further, it is usually assumed for the purposes of this justification that each transit vehicle is at or nearing crush capacity. The New Flyer Excelsior 40” bus is one of the most commonly-used buses by public transit agencies around North America, and they list a maximum of 40 passengers seated, with another maximum of

43 standing [19]. This bus utilization may occur at peak hour near major alighting points (i.e. Business centres of major cities), but is less likely to occur at the start of a route, the end of a route, or outside of the peak hour. Another concern with this optimistic approach is that the illustration assumes that the vehicles are not moving.

Illustration 1: Transit Vehicles vs. Cars [20]

Since the problem of congestion is related to both the volume of traffic and the speed of travel, this comparison of square footage utilized is missing a dimension to its analysis. More directly, this argument should rely on the amount of time a particular vehicle occupies a given unit of space. As such, the speed at which the vehicles move people through a particular space and the number of passengers per vehicle are both important

to this argument, not the area each vehicle occupies when it is stopped. In fact, we should measure passenger throughput through some distance as a measure of efficiency. For example, cars on an arterial road may have an average speed approaching 60 km/h, while the average speed of a city bus is actually 22.8 km/h [21]. If we assume more conservative estimates, that is, that a passenger vehicle carries 1.6 passengers on average, and that the bus carries 30 on average, and that they are travelling 1km in a single lane, you will find that a bus is approximately 7x better at moving passengers through space than a car, not the 27x that this illustration would suggest. When considering ridesharing services, which carry significantly fewer passengers, this benefit is further reduced.

$$c_{eff} = \left(\frac{1.6 \text{ passengers}}{\frac{1\text{km}}{60\text{km/h}} \times \frac{60\text{min}}{h}} \right) = 1.6 \text{ passengers/min}$$

Equation 1: Car Space Efficiency Sample Calculation

$$b_{eff} = \left(\frac{30 \text{ passengers}}{\frac{1\text{km}}{22.8\text{km/h}} \times \frac{60\text{min}}{h}} \right) = 11.4 \text{ passengers/min}$$

Equation 2: Bus Space Efficiency Sample Calculation

$$\Delta_{eff} = \frac{b_{eff}}{c_{eff}} = \frac{11.4 \text{ passengers/min}}{1.6 \text{ passengers/min}} = 7.1$$

Equation 3: Space Efficiency Ratio Sample Calculation

Further, this efficiency ratio does not consider the size of the vehicles involved. Over a distance of 1km, the size of the respective vehicles becomes insignificant to the measurement – only at sufficiently small marginal distance increments would the vehicle size have an effect on the efficiency of each vehicle.

In practice, transport networks are more complex than these idealized calculations would indicate. Consider that when a transit vehicle or ride share stops to allow passengers to board or alight, it imposes some delay on up-stream traffic. Consider as examples: when a bus stops on the roadside to allow a passenger to board or alight; when a bus pulls in or out of a bus bay to allow passengers to board or alight; or when a bus may wait on the roadside to recover their schedule when running ahead. For some period of time, depending on the particular design of the infrastructure, a lane may be temporarily blocked for passenger traffic, or passenger traffic may have to slow in order to permit the ridesharing or transit vehicle to merge. In some circumstances, and for some vehicles, this delay is recoverable – passenger cars in the flow of traffic are eventually able to recover the delay by travelling at a higher rate of speed after successfully passing the transit vehicle, as the density of traffic is reduced ahead of the transit vehicle once the delay clears. For some vehicles, this delay is not recoverable – the delay imposed by the transit vehicle cannot be fully recovered by the vehicle by travelling at a higher rate of speed once the delay has cleared. For the purpose of urban arterials, consider that this unrecoverable delay occurs when the delay is significant enough that the vehicle observes a red light at the next intersection (when otherwise they would have observed a green light), or when the delay imposed by the transit or

ridesharing vehicle is significant enough that the queue affects the performance of the next transit / ridesharing vehicle.

The amount of delay imposed by the transit / ridesharing vehicle on the traffic is affected by a number of parameters. These include the geometric design of the roadway and the stops, signalized intersection timing, and intelligent systems like transit signal priority and traffic shaping. The largest two factors affecting this delay, however, are the length of the transit delay, and the demand for space on the network. The transit delay is determined by the demand for transit, transit scheduling, and the design of the transit facilities. Ridesharing programs like Uber, Lyft, or independent carpools have similar delay factors where they need to pick up or drop off at the roadside, rather than in a parking lot or driveway. The demand for space on the network is determined by the number of people who choose to travel by automobile, and their route choice. The cumulative effect of these choices is manifested as traffic congestion.

During peak periods, high flow rates create larger queues behind the transit / ridesharing vehicle even for short delays, resulting in larger imposed delays. The result of this imposed delay is two-fold: In the shorter term, the delay can be significant enough to offset some of the benefit of having transit / ridesharing vehicles carry a larger number of passengers; and in the longer-term, this reduction in speed will reduce the demand for and utilization of transit and ridesharing services along a corridor. Further, there are scenarios where the imposed delay from transit vehicles is significant enough to effectively reduce the passenger-carrying capacity of an arterial road. This occurs when the delay imposed by transit causes the up-stream demand for road space to exceed the maximum flow of the roadway (congested flow). Clearly, an approach is needed to

quantify the impacts of transit stops on the traffic flow of a roadway in the context of total roadway efficiency. Delay models for other scenarios have previously been developed, and the methodologies for evaluating the delay due to transit can be inferred from available transportation analysis techniques.

2.3 Current Evaluation and Analysis Methods

2.3.1 Transit Planning

Practitioners of transportation planning often consult two separate works, both published by the Transportation Research Board. The first is the Highway Capacity Manual, of which the latest version was published in 2010. This manual provides a multimodal approach to the analysis and evaluation of urban streets, from the point of view of drivers, transit users, cyclists, and pedestrians. This manual is typically used to design urban streets with respect to traffic capacity. The second is the Transit Capacity and Quality of Service Manual, of which the latest version is the 3rd Edition. This manual is typically used to provide appropriate sizing of transit facilities for projected usage. Unfortunately, the procedures for transit location, sizing, and design is not incorporated with the procedures for highway and street design in an integrated way. For example, buses are treated equally by the Highway Capacity Manual, regardless of whether they are inter-city, express, local, or on a deadhead trip. Further, shared right-of-way rail transit, which has been proposed as a solution in many North American cities and continues to be in service in Toronto, is not considered at all. Since transit facilities often share the same facilities as road users, a more integrated and microscopic approach is necessary for the accurate representation of the effects of transit on traffic conditions. Likewise, the Canadian Capacity Guide for Signalized Intersections provides an

adjustment factor for both near-side and far side stops, which can be useful for geometric design and signal timing [22], but is difficult to implement as a model of frictional delay.

In order to obtain funding for transit infrastructure, specifically rapid transit initiatives, a benefits case must be developed. Currently, it is common practice to present these benefits as a Multiple Account Evaluation, or MAE. This MAE was developed by the Crown Corporations Secretariat in 1993, and adaptations to it have been the standard justification for most Canadian Transport Projects since. The MAE framework is typically organized into five major categories:

Financial Performance: documents both the expected revenue and expected expenditures of the alternatives;

Customer Service: documents the benefit that customers or users get from the alternatives;

Environment: documents the impacts of each option on its surroundings, often valued as pollution impacts, natural resource impacts, etc.

Economic Development: documents the change in income and employment of each option vs. the do-nothing alternative; and

Social: documents the impact of each alternative on the society-at-large, communities, etc. [23]

Typically, the customer service account is used to provide the greatest justification for funding a particular transportation project. For transportation projects, this customer

service account is often referred to as “Transportation User Benefits”. There are two main approaches to evaluating these transportation user benefits, both measuring the reduction in congestion. The first is to model total passenger travel time with and without a transit program, and calculate the travel time and vehicle operating cost savings. The travel time savings, multiplied by the value of time, plus the vehicle cost savings is summed into a total value. The other approach used is to assign a monetary value to reduced automobile travel. Each passenger-mile shifted from the average car to a diesel bus results in external savings of between \$0.13 and \$0.24, for example [24]. The remaining benefit from automobile savings is typically articulated as “safety” and involves a reduction in automotive accidents and property damage as a result of a reduction in vehicle kilometers travelled. An example, from the Sheppard-Finch LRT Business Case, is provided below in Figure 6.

All Values PV \$m	Option 2	Option 2a	Difference
Travel Time Savings	857	877	+20
Automobile Operating Cost Savings	819	904	+85
Safety Benefits	87	96	+9
Total Transportation User Benefits	1,763	1,877	+114 or 6%

Figure 6: Transportation User Benefits Example

As can be seen in the example, Option 2a, which in this case involves extending the planned LRT into Lester J. Pearson International Airport, would account for \$20M more in reduced travel time, \$85M more in operating cost benefits, and \$9M more in safety

benefits over the horizon of the study. These benefits are calculated over a 30-year period. This particular study compares the extrapolated scenarios where an LRT is constructed in the middle lanes of Finch Avenue with the scenario where traffic is allowed to grow unmitigated and the existing service is provided. Despite the LRT removing one lane in each direction from the arterial roads, the study projects significant travel time savings for both automobile and transit users [25]. This travel time savings is due to a 2-way peak hour ridership that is higher than the current capacity of the lanes being removed. The questions this generates, however, and the impetus for this research is “How much, if any, travel time savings are expected due to smaller transit or even ridesharing implementations?”, “At what level should we invest in high-volume rapid transit or grade-separated transit?” and “How can we predict the efficacy of these approaches?”

2.3.2 Impact of Lane Closures in Work Zones

The most frequently researched analogue to the delay imposed by transit or ridesharing services on other traffic is calculating the impact of lane closures in work zones. Typically, queuing theory is used to predict traffic characteristics inside the work zone analytically, based on vehicle arrival rate, vehicle departure rate, and projected traffic speeds through the work area [26]. Traffic delay in work zones is typically calculated as the sum of delays due to vehicle deceleration before entering the work zone, a delay due to reduced speed through the work zone, a delay for resuming speed after exiting the work zone, and a delay due to vehicle queues when arrivals exceed departures [26]. A study of traffic delay reduction from automatic workzone information systems indicated that by providing information to drivers and allowing them to self-select a diversion yielded positive results by reducing the arrival rate at the work zone [27]. This

observation could prove useful as autonomous cars are developed, allowing vehicular traffic to self-organize to avoid scenarios where the road capacity is reduced temporarily and choose a different path, however, with both the frequency and randomness of these vehicular delays, it would be unreasonable to expect human drivers to be able to mimic this behaviour to avoid the smaller delays imposed by traffic and ridesharing services.

Since work zones remain in place for longer periods of time than transit / ridesharing services, and influenced by traditional micro-simulation models, average rates and statistical distributions (Poisson's distribution, for example) are used extensively to predict delay [26] [28].

2.3.3 Transit – Traffic Interaction Modeling

Perhaps the first foray into attempting to quantify the interaction between transit and traffic was undertaken in 1977 by Michael Florian, then the director of the Centre for Research on Transportation at the University of Montreal. He noted that "...the choice of passengers between modes has direct effect on the congestion of the road network, which in turn has a direct effect on the transit vehicle travel times, and hence, upon the modal changes made." [29]. While this study doesn't address the physical interaction directly, it does raise some interesting questions. For example, he identifies that the impedance penalty for buses increases with the number of passengers boarding and alighting. This penalty makes transit less attractive to riders as the line becomes more popular. Still, Michael Florian's model assumed that there was no change in impedance for traffic as a result of increased transit dwell times. Much research since has been devoted to solving the multi-modal travel demand model; that is, how to predict the demand for both road space and transit, given the relative properties of each.

Progressively sophisticated models have been developed to minimize the generalized cost of transit passengers [30], and modeling conditions where the probability of queuing exists among transit passengers at bus stops, and not all passengers will necessarily be permitted to board [31]. While much attention has been dedicated to solving the problem of how many passengers will demand to travel on a network, much less attention has been given to the supply analogue of this problem.

Based on the work of the Victoria Transport Policy Institute, and particularly Todd Litman, buses “impose congestion costs equivalent to 1.5 cars on highway and 4.5 cars on surface streets, so net benefits occur when more than about 3 trips shift from automobile to transit” [32]. While an integer is convenient, given the complexity of transportation networks, fluctuations in traffic volumes, transit dwell times, etc. this seems overly simplistic.

Comparatively, the Canadian Capacity Guide for Signalized Intersections, endorsed by the Transportation Association of Canada, offers the following with regards to calculating the capacity of intersections where transit service is also provided. Saturation flow through an intersection is subject to an adjustment factor that is affected by the location of any transit stop. In particular, for near-side transit stops, the adjustment factor is calculated as follows:

$$F_{transit} = 1.0 - kBT/3600$$

Equation 4: Near-side Stop Transit Adjustment Factor

Where k is the coefficient for the effect of transit vehicle loading during green intervals, calculated as:

$$k = [(\% \text{ loading on green}) * c] / [100 * g_c]$$

Equation 5: Coefficient for the Effect of Transit Vehicle Loading During Green Intervals

Where c is the cycle time of the signal in seconds, g_c is the effective green interval in seconds, B is the number of transit vehicle arrivals per hour, and T is the average transit dwell time [33].

The Canadian Capacity Guide for Signalized intersections also proposes a different methodology for far-side bus stops as well, which involves a more complicated process more akin to the analysis performed in this research. The adjustment factor for far-side bus stops is calculated as illustrated in the following table:

Step	Calculation	Note
1	$t_d = T B$	
2	$t_{rs} = (L_a / L_{pcu})(3600 / S) B$	
3	If $t_d * t_{rs} \leq 0$	This adjustment is not applicable
4	If $t_d * t_{rs} > 0$	
5	$F_{bus} = 1 * [(t_d * t_{rs}) / 3600]$	

Table 1: Calculation of the adjustment factor for far-side bus stops

Where, t_d is the total bus dwell time in an hour (s/h), T is the average dwell time of a transit vehicle at a stop determined as the average boarding and alighting time for the transit vehicle plus the acceleration and deceleration time (s), B is the frequency of service (buses / hour), t_{rs} is the time to fill the curb lane behind a bus at the stop determined as the time recovered due to the available storage (s/h), L_a is the length of the available storage space (m), L_{pcu} is the length of the space occupied by one passenger car unit, usually taken as 6m, S is the saturation flow adjusted by other factors, such as width or grade (pcu/h), and F_{bus} is the saturation flow adjustment factor for a far-side bus stop [33].

This procedure analysis is heavily used by traffic engineers when computing the capacity of an intersection, which can often be a limiting factor in traffic flow, and thus is valid. If we are, however, to take an approach where we evaluate intersection performance on a cycle-by-cycle basis, rather than assuming average costs, this problem should inherently become more complicated, as uneven distributions of arriving passenger cars could push intersections beyond capacity at a time where there would not be sufficient lapses in congestion to recover from intersection failure. Secondly, this analysis does not incorporate ridesharing services like Uber, UberPool, Lyft, Taxis, and the like, where passenger cars may stop on the roadside to allow passengers to embark/disembark, causing similar temporary disruptions in traffic on a more frequent basis. While this analysis may prove sufficient for average calculations considering infrequent transit service, as these roadside stops approach or exceed one stop per light cycle, these analysis tools may no longer become applicable.

2.4 Transit Vehicle Delay

2.4.1 Introduction

While traffic modeling and transit modeling are generally well defined independently through queuing theory and the procedures outlined in the Highway Capacity Manual, the specific interactions between these vehicles on a macroscopic level are generalized and obscured by using generalizations. The objective of this chapter is to articulate the factors that are important with regards to modeling the interaction between transit and traffic.

2.4.2 Calculating Transit Vehicle Delay

When a transit vehicle comes to a stop in order to let passengers board or alight, it experiences a delay. This delay, while imposed on the passengers of the vehicle, is

typically incorporated into the vehicle's schedule. Passengers onboard, having taken the schedule into consideration when choosing to travel, do not perceive this as a delay unless the vehicle is significantly behind schedule, however, their time-cost to travel is still higher than if the vehicle did not stop.

The length of this delay is an important input to the model. From the Transit Capacity and Level of Service Manual, Third Edition, published by the Transportation Research Board, we can take the overall transit vehicle delay as a sum of the applicable individual component delays. These delays are:

Deceleration Time, (d_a): The amount of delay the vehicle experiences during deceleration. This value can vary by the type and model of vehicle, based on its performance. If the deceleration is assumed linear, this is usually taken as half of the product of the traffic flow speed and the time required to reach that speed.

Boarding Lost Time (d_b): The amount of time it takes the passengers to move from their waiting position to the door of the vehicle. This time increases if boarding is permitted at the rear door(s), or if the transit shelter is located some distance from the boarding point. For passengers waiting at a flag stop for front door only boarding, this value is typically taken to be near-zero. For this analysis, this delay only occurs when the vehicle is stopped in a regularly travelled traffic lane.

Dwell Time (d_d): The amount of time the bus is stopped during the boarding and alighting process, or while on a lay-over waiting for its departure time. At high-usage stops, the dwell time is the most significant portion of the delay. For this analysis, this delay only applies when the vehicle stops in a regularly travelled traffic lane.

Traffic Signal Delay (d_{ts}): The amount of time a vehicle has to wait for a green signal after making a stop. For vehicles utilizing transit signal priority, this value can be near-zero. In cases where the signal timings are poor, vehicles are off-schedule, or where transit signal priority is not available, this delay can be as long as the red signal in the alternate direction(s).

Re-Entry Delay (d_r): The amount of delay the vehicle experiences either waiting for a gap to re-enter the traffic flow, or the amount of delay during that re-entry process. For this analysis, it is considered that the vehicle is always able to enter the flow (transit vehicles have the right of way). This delay only applies when the vehicle has its own bay outside of the regular traffic flow.

Acceleration Delay (d_a): The amount of delay the vehicle experiences while accelerating after a stop, up to the regular traffic flow. This value, like deceleration is closely tied to the performance and type of the vehicle. If the acceleration is assumed linear, this is usually taken as half of the product of the traffic flow speed and the time required to reach that speed.

Vehicle Stop Failure (d_f): The delay experienced by a vehicle while waiting to enter a bay that is already occupied by enough vehicles to prevent entry. This delay only applies when the vehicle has its own bay outside of the regular traffic flow [34].

As discussed earlier, the total delay experienced by the transit vehicle can be taken as the sum of all applicable component delays. In practice, any non-applicable delay can be taken as zero. The formula for this transit vehicle delay is thus given by D:

$$D = d_a + d_b + d_D + d_{ts} + d_r + d_a + d_f$$

In practice, this total delay experienced by the transit vehicle typically falls between 0 and 160 seconds, but conditions can vary [34].

2.4.3 Curb-Side Stops

One of the most common paradigms for implementing transit service is to provide curb-side stops. A curb-side stop is the most simple of stops from a design perspective, consisting of only a bus-stop flag or perhaps a shelter. In this case, the road is not outdented – while the vehicle stops to allow passengers to board or alight, the bus will remain in a standard traffic lane. Ad-hoc curb side stops will often be utilized by ride-sharing services in much the same way, unless expressly prohibited by signage or local by-law. Illustration 2 shows a typical curb-side stop on Steeles Avenue East in Markham, Ontario.

Illustration 2: Curb-side Stop in Markham, Ontario [35]

Curb side stops have different values for transit vehicle delay than Bus bay stops. Figure 7 demonstrates the component delays for a curb-side stop. Ridesharing services will experience the same approximate length of delays due to dwell time, however, their acceleration time and deceleration time are usually smaller than transit service, owing to the difference in performance between buses/streetcars and private automobiles.

Component Delay	Typical Value
Deceleration Time, (d_d)	4.5 sec – 9.0 sec
Boarding Lost Time (d_b)	0 sec – 9.0 sec
Dwell Time (d_D)	10 sec – 60 sec
Traffic Signal Delay (d_{ts})	0 sec – 70 sec
Re-Entry Delay (d_r)	0 sec
Acceleration Delay (d_a)	5.5 sec – 11 sec
Vehicle Stop Failure (d_f)	0 sec
TOTAL	20 sec – 159 sec

Figure 7: Component Delays for Curb-Side Stops [34]

2.4.4 Bus-Bay Stops

Less commonly, bus bays are often implemented in situations where there are high-volume bus stops on an arterial road that also experiences high traffic. The implementation of bus bays allows the bus to avoid impeding traffic while it allows passengers to board and alight. Illustration 3 shows a bus bay in Ottawa, Ontario.

Illustration 3: Bus Bay Stop in Ottawa, Ontario [36]

While bus bays eliminate the obstruction of traffic while passengers are boarding and alighting, the trade-off associated with a bus bay stop is that once the vehicle has completed its stop, it must merge with existing traffic, which can take additional time. Figure 8 demonstrates the component delays for a bus bay stop. Please note that dwell time, while it is a delay for the transit vehicle, is not a delay to traffic in a bus bay scenario, as the bus is located outside of a normal operating lane of traffic. While bus-bay stops may be the most effective at reducing the impact of transit-traffic interaction delay, they also require more investment and occupy real-estate. If implemented along the length of a corridor, bus bays could take up a similar amount of space to implementing an entire lane of traffic. This consideration should prompt an evaluation of reduced delays and benefits associated with the implementation of an entire travel lane vs. the introduction of bus bays.

Component Delay	Typical Value
Deceleration Time, (d_d)	4.5 sec – 9.0 sec
Boarding Lost Time (d_b)	0 sec
Dwell Time (d_D)*	10 sec – 60 sec
Traffic Signal Delay (d_{ts})	0 sec – 70 sec
Re-Entry Delay (d_r)	0 sec – 10 sec
Acceleration Delay (d_a)	5.5 sec – 11 sec
Vehicle Stop Failure (d_f)	0 sec
TOTAL	10 sec – 160 sec

Figure 8: Component Delays for Bus-Bay Stops

As discussed in the previous chapter, ridesharing services may use bus bay stops in an ad-hoc fashion to allow passengers to board/alight, unless expressly prohibited by signage or local bylaws. In the case of bus bays, the delay imposed by ridesharing services approaches zero, as the delay is limited to deceleration time, re-entry delay, and acceleration delay, which are all linked to vehicle performance criteria. Because private automobiles typically have higher performance than transit vehicles, these delay factors are reduced (but not eliminated). Research into the delay for these vehicles is ongoing as services of this type become more prevalent.

2.5 Limitations of Current Practices

Traditionally, practitioners consider costs of transportation in terms of average costs, and consequently disregard any negative externalities that each traveller or vehicle imposes on others. This results in an underestimation of congestion costs [37].

The prevailing methodology applies macroscopic generalizations to the study area, extrapolated from current data and calibration over a large time horizon. The objective of this approach is to project future traffic and modal split for each of the options under evaluation. Once each model is executed, the vehicle kilometres travelled and total travel times are compared with the base case and any reduction considered a benefit [25]. One problem with this approach is that both the “base-case” scenario, and the scenario under consideration are both extrapolated from current data, and these projections themselves are subject to variations in accuracy and precision as modal shift and urban development occur, free from the constraints of the models in question.

Microsimulation software packages are increasingly being used and provide a number of tools to develop results such as total vehicle delay across the study area, but these simulation packages require significant modeling, development, and often need to be executed across a massive number of scenarios. These software packages also use average values and statistical distributions. As discussed earlier, this can result in underestimate of congestion costs because the negative externalities imposed by one vehicle on another is disregarded. Further, since microsimulation uses average values, statistical distributions and local calibrations, any analysis can be considerably time consuming and require significant interpretation. Furthermore, microsimulation software packages are not yet designed to provide analysis of individual network links within a

network, provide details on total person throughput by default, rather these indicators need to be distilled from other results. In addition, these microsimulation packages use traffic assignment and demand assignment methods that can further obscure the benefits realized by these marginal changes. Likewise, microsimulation packages are not suited to accepting real-world data, say, from Internet of Things devices to generate an appropriate calibrated option.

Ultimately, the ability to distill marginal effects from given real-world data is currently systematically unavailable to transportation engineers and planners. This ability would indicate whether, and under what conditions a marginal increase in transit services correlates to a marginal increase in person throughput and decrease in travel time, permitting a number of analytical planning decisions to be made and directly influencing design factors. In particular, we would be able to answer the question: If demand for transit service is high in a congested area, does a reduction in headway time for transit address both the demand for transit and the total travel demand on that network link? If not, further analysis should be undertaken to determine if other solutions, including urban transportation management or dedicated right-of-way rapid transit construction should be considered.

2.6 Operational Consequences of Shared-Use Infrastructure

Transportation networks are subject to significant negative network effects. One of the most obvious negative externalities of congestion is that as congestion increases, travel time increases, thereby reducing the value of the trip to the user. This negative externality (congestion) is what prevents the positive network effects from going to infinity over time [38].

When transport networks use a single uniform speed, vehicle, etc. the maximum benefit is achieved. In optimum conditions, for example, a subway rail network is capable of operating many high speed trains on limited headway and carry significant passenger volume because the vehicles all operate in a uniform manner. If this consistent operating paradigm is disrupted for any reason, the trains will begin to gap and bunch, and the overall efficiency of the network is reduced.

Similarly, road transportation networks operate in the most efficient manner when all vehicles operate under the same paradigm. If vehicles were all the same size, had similar operational characteristics, operated at the same travel speed, and followed the same path, traffic networks would carry the greatest amount of passengers and/or goods per unit time and length. John Glen Wardrop, an English transport analyst explores this concept of user- equilibrium and system optimization. The first principle states that at user equilibrium, no individual transport user can unilaterally reduce his or her travel time (or total cost) by switching to a different route [39]. This is a Nash Equilibrium concept, with the notable difference that transportation networks have many players, which make the game theory analysis more complicated. In Wardrop's second principle, it states that the average journey time is minimum when no user can lower the total travel time (or cost) in the system by altering their travel behaviour [40]. This principle is known as system optimization.

In a hypothetical transport network where all vehicles operate under the same paradigm and travel at the same average speed throughout the network, the user-equilibrium condition and the system optimization condition are the same. Since there would be no

mode choice and travel speeds and other costs are uniform, users would always select the shortest route to their destination, and it would be the optimal system load.

In reality, however, there are a number of factors keeping travel from operating in a uniform manner. There are a number of different vehicles (cars, trucks, buses, trams, bicycles) and pedestrians that all share the same infrastructure, there are different speed limits, types of roads, traffic control devices, length of network links, and even geographical restrictions that prevent the uniform behaviour of traffic.

Looking at a single network link, however, it can be presumed that the road characteristics are all uniform, and the only differentiation is modal choice between vehicles. Because the varying types of vehicles make different stops and have different performance characteristics, the user-equilibrium of this link may be of reduced efficiency compared to the system-optimization state on the network link under consideration. Compounding the problem is the different passenger (and goods) capacities of all the vehicles traversing the network link. If system optimization is defined as the maximum passenger throughput on this link, the consequence of multi-modal traffic is that the total passenger throughput at user-equilibrium is possibly below the passenger throughput at system optimization, and is directly affected by the demand on the network link under analysis.

3 Model Development

The intent of this research is to provide a modelling framework for individual links on any network, subject to local conditions such as travel speed, traffic demand, effective green at signalized intersections, etc. and therefore by design, needs to be implementable in every situation. Evidently, each measurement unit, whether it be green cycles, minutes, or otherwise, will be different from the previous if actual data is being used, and for that reason, this research considers the effects of marginal changes. Geographically, the scope of these models refers to only a single link between traffic control devices on a network. By definition, the maximum capacity on these links is governed by the number of entrances permitted by a traffic control device upstream of the link, and cannot have throughput exceeding this number of entrances. As interactions between transit / ridesharing services and automotive traffic increase, logically, some vehicles will be delayed enough to be stopped at red lights where they would previously have passed through the next intersection on a green light. This delay is unrecoverable, as the vehicle has now moved into a later platoon of vehicles passing from intersection to intersection. This may repeat a number of times over a trip as traffic conditions and transit stops occur, causing a series of delays for that automobile. Similarly, in situations where there is sufficient traffic to approach the capacity of a signalized intersection, vehicles that have dropped back into a later platoon may cause the intersection capacity to be exceeded, causing cascading failures at intersections, and potentially delaying the transit or ridesharing vehicle also. Objectively, this research is interested in evaluating the projected effects that each design criteria has on traffic in order to identify suitable management strategies. For example, the effect of increased

transit / ridesharing service will produce different traffic and delay results depending on the distribution of automobiles and their speeds at any given time. By modifying the variables, we can attempt to predict which congestion mitigation efforts are most likely to work.

In general, micro-simulation models are classified into two categories depending on how the model generates vehicles and their paths. Deterministic models, for example, assume that driver-vehicle characteristics do not vary during the simulation – with each vehicle driving the same speed and with the same headway. Stochastic models, on the other hand apply a random process using statistical distributions to determine these characteristics [41].

The model developed in this research is unique in that most driver characteristics can be measured and utilized directly in the model, eliminating the need for deterministic or stochastic approximations. Even so, if only deterministic or stochastic approximations are available for analysis, the model can be suitable for these purposes. Further, it can be expanded to incorporate other multi-modal interactions (though these interactions are saved for further research). To evaluate this model for predictive analytics, however, in the absence of specific vehicle data, some generalizations are required. They are as follows:

Vehicle Generation: The model developed here will support any type of vehicle generation process preferred by the user. Actual arrival times at the entry point to the arterial link may be utilized if available, as well as a stochastic or deterministic arrival pattern. This model, however, is only considering transit, ridesharing, and automotive

services, so the class of vehicle (i.e. truck) is not incorporated into this model. For these purposes, we consider all vehicles to have the same performance characteristics.

Vehicle Movement: This model considers only a link in the network for the purposes of eliminating vehicle routing from the analysis. Vehicles allowed through the previous traffic control device are considered to enter this link, with only one exit. Because of this, we can isolate the performance of a particular link on the network independent of driver choice, which is desirable as it reduces uncertainty. Vehicles are assumed, for the purposes of this research, to behave in a uniform manner, or deterministically. I hypothesize that the differences between a deterministic model, stochastic model, and actual data for the ranges of speeds and behaviours seen on a single network link are minor enough to not affect the results of the analysis, and that so long as the same method is used for the baseline scenario as for the projected scenario, any difference is purely trivial. This model can, however, be adopted to incorporate delays due to varying speeds of vehicles as well as driver aggressiveness.

Infrastructure Design: Evaluating this model across a large number of scenarios involves some assumptions on infrastructure design. These parameters will be discussed later, however, the effect of each parameter on the results is more indicative of the desired outcome from the model.

3.1 General Model for Imposed Delay

This chapter is concerned with how to calculate the delay that an individual vehicle experiences when it approaches a transit or ridesharing vehicle that is stopped on the side of an arterial roadway.

To achieve the objectives of this research, it is required that the delay imposed on a single vehicle as well as the total cumulative delay imposed on all vehicles in the traffic stream imposed by the transit or ridesharing vehicle be calculated. From queuing theory, this model will utilize deterministic queuing analysis to determine the total delay. Deterministic queuing analysis is a simple method that treats the length of the queue as the difference between arrivals and departures at a point [3]. This research is unique in that it aims to utilize deterministic queuing analysis to determine the additional time required for a vehicle to travel along an arterial link in a network, and consider whether the vehicle will still proceed through the intersection on the same green cycle as it would if it were unimpeded.

When the arriving traffic flow rate exceeds the capacity permitted by the road during the transit stop, the number of vehicles arriving will exceed the number of vehicles departing the area containing the stop. The difference between the number of vehicles arriving and departing is the queue formed at the stop, given by N_q . [26]

$$N_q = (q_a - q_d)t$$

Equation 7: Number of Vehicles in a Queue at Time t

Where q_a is the arrival rate of vehicles and q_d is the departure rate of vehicles.

Typically, this queue analysis method is used at a planning level with average flow rates calculated over a large period of time (often 15-30 minutes). Vehicle arrival rates are typically calculated in terms of vehicles per hour (pc/hr). In this case, however, because the delay imposed by the transit vehicle is relatively small per vehicle, and the area under investigation is geographically small and simple, it is more reasonable to express

the arrival rates in terms of vehicles per second (pc/sec). Given an arrival rate, q_a , a departure rate during the transit delay, q_d , and a departure rate after the transit delay has completed, q_f , the queue dissipation time, t can be calculated as follows:

$$t = \frac{D(q_d - q_f)}{(q_a - q_f)}$$

Equation 8: Queue Dissipation Time

If t is longer than the amount of time between transit vehicle arrivals, then the vehicles in the queue behind the next transit vehicle, and the next transit vehicle will experience both delays. A delay will be imposed from both the first transit vehicle and the second transit vehicle. These delays can be summed and should be included in the solution for the impacts of the second vehicle when this occurs. It is likely that in this situation, the delay imposed by the transit vehicle on the traffic will be significant enough to eliminate any efficiency gains as a result of transit implementation due to cascading failures, and another solution to congestion should be investigated, whether by increasing passenger throughput on an adjacent road, or by investigating other transit solutions.

A graph of cumulative arrivals and cumulative departures is shown below in Figure 9. This figure demonstrates the scenario where there is an arrival rate, q_a of 0.5 pc/sec, a delay, D , of 60 seconds, a departure rate during the delay q_d , of 0.333 pc/sec, and a departure rate after the delay, q_f , of 0.666 pc/sec. In this case, the time at which the cumulative number of arrivals and cumulative number of departures are equal is found at 240 seconds, which is the amount of time that elapses before the queue is dissipated. Additionally, the length of the queue in terms of the number of vehicles can be shown as the y distance between each line at point t .

Figure 9: Cumulative Arrivals vs. Time

The length of the queue solely is not sufficient to describe the traffic delay imposed by the transit vehicle on the existing vehicular traffic. Of particular interest is the total (cumulative) delay imposed on the traffic by the transit vehicle. In order to obtain this delay, the area under the arrivals curve must be subtracted from the area under the departures curve as follows:

$$D_{total} = \int_0^t F(q_a) dt - \int_0^t F(q_d) dt$$

Equation 9: Area Between the Arrival and Departure Curves

where $F(q_a)$ and $F(q_d)$ are the functions representing cumulative arrivals and cumulative departures respectively. In this development, the function for departures is not smooth and difficult to integrate using this method. Instead, using the equations and conditions

developed earlier in this chapter as well as integration where appropriate, the resulting formula is as follows:

$$D_{total} = \int_0^t q_a t \, dt - \left[\int_0^D q_a t \, dt + \int_a^b q_f t \, dt + q_a D(t - D) \right]$$

Which solves to the following:

$$D_{total} = \frac{q_a t^2}{2} - \left[\frac{q_a D^2}{2} + \frac{q_f (t - D)^2}{2} + q_a D(t - D) \right]$$

Equation 10: Total Imposed Delay

When solved, D_{total} gives the sum of all individual delays experienced by vehicles in the queue. In some cases, the cumulative delay can be quite significant, even for short delays imposed by transit vehicles, as considered here. Please note that any queuing methodology used by a practitioner will be suitable in this application, as long as there is a defined function for vehicle arrivals and departures in that methodology.

3.1.1 Deterministic Queuing Adjustment

The deterministic queuing approach assumes that vehicles are stacked vertically (or are somehow all positioned at the point of delay), but this is not consistent with real world conditions. In practice, vehicles are queued along the roadway behind the transit vehicle. In addition to waiting behind the blockage, these vehicles must also travel from their position in the queue to the site of the blockage. In order to properly evaluate all of the delay imposed on traffic by the transit vehicle, some correction to the delay time must also be incorporated into the analysis.

The effect of deterministic queuing can be compensated for using a number of ways, most notably by increasing the arrival rate to compensate for vehicles arriving at the delay sooner. This formula can be developed using basic flow theory. The flow rate, q_a , is given by the product of the mean free speed, u_a and the traffic density, k_a . We can also calculate the reverse velocity of the queue by using the product of the previous flow rate and the density in the queue, $q_a k_d$. To get the adjusted flow rate, the sum of the forward and reverse velocities is used to re-calculate the new adjusted flow rate. K_a , the vehicle density, is the inverse of the vehicle headway. Therefore, to determine the increase in arrival rate during the queue, the following formula shall be used in place of q_a :

$$q_{a,adjusted} = (q_a h_a + q_a h_d) k_a$$

Equation 11: Adjusted Arrival Rates for Deterministic Queuing

Where h_v is the vehicle headway at maximum flow rate, the sum of both the average length of the vehicle and the forward spacing between a vehicle and the next vehicle during the delay, L is the number of lanes in one direction on a particular roadway, and u_a is the observed speed of the arriving vehicles.

This deterministic queuing adjustment is incorporated into the next chapter, and in subsequent chapters, $q_{a, adjusted}$ is referred to as simply q_a .

3.1.2 General Model for Delay Recovery

In 1933, Greenshields postulated a linear relationship between speed and traffic density as shown below in Figure 10 [42]. This relation still holds up today, though some more sophisticated research projects have developed more specific relations for specific traffic

conditions. From the original Greenshields relations, however, it could be determined that there is a maximum flow rate where density is zero.

Figure 10: Greenshields Speed-Density Relationship

Since there will be no vehicles within some distance of the lead vehicle, Once an automobile or the transit vehicle has passed the queue point, the traffic will leave the queue point at the maximum flow rate of the roadway, given by:

$$q_{max} = L \frac{k_j u_f}{4}$$

Equation 12: Maximum Flow (Greenshields)

Greenshields' speed-flow relation, shown here in Figure 11, shows two regimes, meaning it's possible to have two different speeds at the same flow – one for congested flow and one for uncongested flow.

Figure 11: Greenshields' Speed-Flow Relation

The maximum flow of this relation indicates that the speed of travel at maximum flow is equal to half of the free-flow speed of the roadway [42]. This can be validated through an analysis of the speed-flow relation, given by:

$$q = k_j \left(u - \frac{u^2}{u_f} \right)$$

Equation 13: Greenshields' Speed-Flow Relation

This relation can be isolated for speed, u , as follows:

$$u = \frac{k_j u_f \pm \sqrt{k_j^2 u_f^2 - (4k_j u_f q)}}{2k_j}$$

Equation 14: Inverted Greenshields' Speed-Flow Relation

The assertion that speed u , at maximum flow, q is equal to half of the free-flow speed can be validated using this relation, and both the additive solution and the subtractive solution will yield the same result. Free-flow speed, u_f , is often taken to be equal to the design speed of the roadway, for analysis purposes.

In order to calculate the deterministic queuing adjustment, we must consider that when vehicles leave their imposed delay during the vehicles delay, they are able to travel at some increased speed in order to recover the gap. While travelling along this gap, the lead vehicles are able to travel at free-flow speed, since there are no vehicles in front of them. They may travel at free-flow speed until they catch up to the lead group, who are travelling at the original flow (referred to as Standard Conditions in Illustration 4: Traffic Conditions During Delay). The following vehicles, who are also able to pass the stopped transit vehicle during a delay will travel at or near free-flow speed as well. These vehicles will eventually return to their approximate place in the original flow, assuming that they do not experience an intersection delay that they would have not experienced without the queuing delay described here. Any delay imposed by a traffic control device that would not have been otherwise experienced by a vehicle had it not been delayed by a transit vehicle must thus also be incorporated in this estimation.

After the transit vehicle delay, both the transit vehicle and the vehicles behind it now have an opportunity to travel at or near free-flow speeds in order to recover the delay imposed by transit. In this case, the vehicles will leave the delay point at a rate equal to the maximum flow of the roadway, and be able to travel some distance until the recovery section has been filled with vehicles.

Illustration 4: Traffic Conditions During Delay

To calculate this recovered delay, we consider as a first term that all of the delay experienced by vehicles that are able to pass the transit vehicle during the delay. This was given as the first subtractive term in Equation 10,

$$\frac{q_d D^2}{2}$$

Where q_d is the departure flow during the delay, and D is the duration of the delay. The second term is more complex. Conceptually, the lead vehicle (in this case, the transit vehicle) is able to travel at the free flow speed of the roadway until it catches up to the standard flow at the other end of the recovery gap. This distance is equal to the length of the transit vehicle delay minus the sums of the headways of the vehicles that passed it during the delay. Developing this into an implementable formula becomes the following:

$$x = (u_f - u_a)D - \left[q_f D \frac{(h_a - h_d)}{L} \right]$$

Equation 15: Recovery Section Distance

Where, x is the distance over which the recovery can be made, u_f is the free flow speed of the roadway, u_a is the speed of the traffic prior to (and ahead of) the delay, D is the length of the transit vehicle delay, q_f is the departure flow after the delay, h_a is the headway spacing in the original flow, and L is the number of lanes on the roadway.

The calculation of the recovery time for the remainder of the vehicles is calculated similarly. Each vehicle will be able to recover some portion of the delay while travelling along the length of the recovery period. It should be considered though, that the distance headways between vehicles in the maximum flow condition is shorter than the distance headways in the original flow condition. As such, the i^{th} vehicle in the queue will be able to travel a distance equal to the original gap in the traffic, plus i times the headway in the congested flow, minus i times the headway in the original flow. A relation can be developed from the gap distance calculated for the transit vehicle. Since the delay imposed on all vehicles that are able to pass the transit vehicle are fully recovered, this analysis begins when the delay is released and has similar conditions. Given that the length of gap ahead of the first vehicle is given by:

$$x_1 = u_a D - \left[\frac{q_f D (h_d - h_a)}{L} \right]$$

Equation 16: Recovery Section Distance - Vehicle 1

It can be inferred that the gap ahead of the next vehicle can be given by:

$$x_2 = u_a D - \left[\frac{q_f D (h_a - h_d) + (h_a - h_d)}{L} \right]$$

Equation 17: Recovery Section Distance - Vehicle 2

And thus, the gap ahead of the i th vehicle can be given by:

$$x_i = u_a D - \left[\frac{q_f D (h_a - h_d) + (i - 1)(h_a - h_b)}{L} \right]$$

Equation 18: Recovery Section Distance - Vehicle i

And the maximum number of vehicles that can make up any delay before the recovery section has been filled is given by N_{max} :

$$N_{max} = \frac{L[x_1 - (u_f - u_a)D] + q_f D h_a}{(h_a - h_b)} - 1, N_{max} \leq N_q, N_{max} \leq N_s$$

Equation 19: Maximum Number of Recovery Vehicles

The maximum value for N_{max} is subject to two conditions: It must be equal or less than to the maximum number of vehicles in the queue, or, the number of vehicles in the queue when the transit vehicle delay has ended. This value is given by N_q :

$$N_q = (q_a - q_d)D$$

Equation 20: Maximum Number of Vehicles in Queue

N_{max} must also be equal or less than the number of vehicles that can recover their delay prior to the next transit stop or red signal. To calculate this, the velocity of the original flow section must be determined. This velocity is given by u_a , and is known at the start of the analysis, however, vehicles are also joining the flow and reducing this velocity. Given x_{stop} as the distance to the next transit stop or red signal, the number of vehicles that will recover their delay is given by N_s :

$$N_s = \frac{x_{stop}}{(u_a) - \left[\frac{(q_f h_a)}{L} \right]}$$

Equation 21: Maximum Number of Recovery Vehicles Before Next Transit Stop or Red Signal

Finally, it can be seen that the delay recovered by the vehicles that do not pass the transit vehicle during the delay is a linear function. It can be most simply described using summation notation:

$$R = \sum_{i=1}^{N_{max}} \frac{x_i}{(u_f - u_a)}$$

Equation 22: Summation Recovery of Delay

This summation can also be defined as the average value of all recovery times after the transit delay, multiplied by the number of recovery times, as follows:

$$R = \frac{N_{max} \left[\left[(u_f - u_a)D - \left[q_f D \frac{h_a}{L} \right] \right] + \left[(u_f - u_a)D - \left[\frac{q_f D h_a + (i-1)(h_a - h_b)}{L} \right] \right] \right]}{2(u_f - u_a)}$$

Equation 23: Algebraic Recovery of Delay

And the total recovered time is given by the delay recovered by vehicles able to pass the transit vehicle during the delay plus the delay recovered by vehicles once the delay has completed. This is given by:

$$R_{total} = \frac{q_d D^2}{2} + \sum_{i=1}^{N_{max}} \frac{x_i}{(u_f - u_a)}$$

Equation 24: Total Recovered Time

From the above work, the total imposed delay by the transit vehicle on the traffic flow can now be calculated. Given the total delay experienced by the vehicles in the queue during a transit stop, given by:

$$D_{total} = \frac{q_a t^2}{2} - \left[\frac{q_d D^2}{2} + \frac{q_f (t - D)^2}{2} + q_d D(t - D) \right]$$

And the recovered delay experienced by these vehicles:

$$R_{total} = \frac{q_d D^2}{2} + \sum_{i=1}^{N_{max}} \frac{x_i}{(u_f - u_a)}$$

We can develop a relation for the total non-recoverable delay resulting from the imposed transit vehicle delay as follows:

$$D_{nr} = D_{total} - R_{total}$$

Equation 25: Total Non-Recoverable Delay

3.1.3 General Solution

Now that an expression has been developed for the total non-recoverable delay, (see Equation 25) the model framework can be completed. The average non-recoverable delay experienced by the vehicles under consideration is an effective way to bridge the gap between the above delay model, and developing boundary conditions for maximizing passenger throughput.

Given the total non-recoverable delay, D_{nr} , we can divide by the number of vehicles under consideration in the delay model as follows:

$$D_{avg} = \frac{D_{nr}}{q_d D + N_{max}}$$

Equation 26: Average Non-Recoverable Delay

The average non-recoverable delay refers to the average amount each vehicle is travelling behind its original position in the traffic flow at the end of the analysis period, that is, until all delay is recovered, or until the next transit stop. To find the marginal effect on the traffic flow rates, this average delay is multiplied by the original flow rate as follows:

$$q_R = D_{avg} q_a$$

Equation 27: Marginal Traffic Flow Reduction

Finally, the model is able to be correlated to total passenger throughput. By considering the projected transit vehicle occupancy, and the projected passenger vehicle occupancy, boundary conditions can be set for the maximization of passenger throughput.

For transit occupancy o_{tr} , and average vehicle occupancy o_{pc} :

The implementation of transit is effective at maximizing passenger throughput for the case where:

$$o_{tr} > o_{pc} q_R$$

Equation 28: Transit Maximizes Passenger Throughput

The implementation of transit is not effective at maximizing passenger throughput for the case where:

$$o_{tr} < o_{pc}q_R$$

Equation 29: Transit Does Not Maximize Passenger Throughput

In the case where $o_{tr} = o_{pc}q_R$, there are no differences between the two methods.

3.1.4 Limitations of the General Solution

Although the general solution uses first principles to calculate the vehicle delay and determine whether passenger throughput was maximized, it suffers from some generalizations that make its use impractical:

1. *The General Solution assumes that the arterial road is straight and infinitely long:*
Given a sufficiently long road and no intersections, the delay caused by any traffic-transit interaction is minimized. While an automobile or other component of traffic may be delayed by a transit vehicle under these conditions, there is an infinite amount of time remaining for the vehicle to recover the delay imposed. There would only be a delay in the case where the prevailing traffic flow both before and after the transit vehicle delay are in excess of the travel speed of a given vehicle, or in the scenario where a transit / ridesharing vehicle's delay affects the next transit / ridesharing vehicle in sequence.
2. *The General Solution attempts to solve for the Average Vehicle Delay:* While the principles are valid, there will be vehicles that experience a delay but do not pass the transit vehicle, allowing themselves to experience a delay again from farther downstream, and there are vehicles who will be traveling alongside the transit vehicle at the time of the stop and will experience no delay. The calculated delay and

associated recovery need to be calculated on an individual basis and the results compiled using summation notation for the model to be specific.

Despite these generalizations, this effort provides an excellent basis for the development of a specific model that can be applied to real-world scenarios. In particular, if we introduce a variable representing the length of the arterial under analysis; that is, the length of the arterial between signalized intersections, and we include timing of the signalized intersections, this general solution begins to describe the actual unrecoverable delay experienced by vehicles in a given situation. Lastly, if the model then evaluates each vehicle one at a time and evaluates its delay before summation, we are able to predict the cumulative delay experienced by all vehicles on the arterial during the time period in question.

3.2 Specific Model of Imposed Delay

In this chapter, a specific, implementable model of network delay will be developed and applied to obtain predictive analytical solutions for a number of variables. Since a number of design and traffic variables are present in the model, as shown in the previous chapter, a number of scenarios are developed and discussed here individually, and the set of results are compared.

In this analysis, the variable identified as relevant to this analysis are as follows:

1. Number of traffic lanes (one direction)

In a scenario where a transit or a ridesharing vehicle comes to a stop on a single lane road without a bus-bay, traffic behind the transit or ridesharing vehicle must necessarily come to a stop and wait behind the vehicle for it to proceed. This results in a different

delay model from a two-lane road, where vehicles may pass the transit vehicle as they are able during the stop, reducing the capacity of the roadway, rather than blocking traffic entirely. It can be seen logically that the delay imposed by a transit or ridesharing vehicle will be necessarily larger on a single lane road, however, the implications of this delay with respect to the total delay of traffic will be discussed later.

2. Free-Flow Speed of Automobile Traffic (km/h)

The Free-Flow speed of automobile traffic occurs when flow-rate is zero. While this speed cannot occur, by definition, this variable is used to calculate predicted travel times under various congested conditions and will have an impact on the delay experienced by vehicles.

3. Arrival Traffic Flow (veh/hr or veh/s)

Depending on the data available, each network link will have an arrival traffic flow which will be expressed in vehicles per hour or vehicles per second. For the purposes of this analysis, actual arrival times from the previous signalized intersection are preferred, as this research uses a direct summation of data to calculate expected delay. For the purposes of developing this model, vehicles per second is the preferred unit of measure, and this arrival rate is averaged based on control of the previous signalized intersection.

4. Length of Road Network Segment (m)

The length of the road network segment is important to the analysis. As previously discussed, on an infinitely long road, all vehicles are able to make up the delay imposed on them under the conditions that they can eventually pass the transit vehicle under analysis. Since each network segment is constrained by signalized intersections both before and after, the length of the network segment can allow us to determine whether or

not vehicles are able to recover the delay imposed on them prior to the next signalized intersection.

5. Transit Stop Location – Distance from Origin (m)

As discussed in previous chapters, the Transportation Association of Canada has published different guidelines for intersection capacity based on whether a stop is located on the far-side of the intersection (near the origin) or on the near-side of the intersection (far from the origin). This model uses this parameter in order to determine the length of road under which vehicles may recover the delay imposed on them. In cases where a ridesharing vehicle is under analysis, its stop location relative to the origin is used here in place of a fixed transit stop.

6. Length of Signal Cycle & Green Cycle – Origin (seconds)

At the origin, the signalized intersection is the control under which the traffic flow rates are generated. Clearly, traffic flows are higher during a green control than under a red. This variable represents the total length of time from the start of a green signal to the start of the next green signal.

7. Length of Signal Cycle & Green Cycle – Destination (seconds)

At the destination, the signalized intersection is the control under which traffic is allowed to exit the network link. Clearly, the longer a signal remains green, the higher the likelihood that a vehicle will be able to recover its delay on a later link, as it will have sufficient time to catch the platoon of vehicles that it belonged to prior to delay.

8. Signal Timing Offset (seconds)

This variable represents the time offset between the start of green signals at both the origin and the destination. Occasionally on priority thoroughfares at peak times, green

signals will be synchronized in a manner that permits vehicles travelling along the road to receive a green signal at every intersection so long as they maintain the posted speed limit. In these cases, vehicles that are delayed by transit or ridesharing vehicles must recover sufficient delay prior to approaching the intersection in order to “catch” the green light and make the remaining delay recoverable. In situations where there is no offset or where priority is given to a different roadway, the likelihood of delay is different, and this model aims to capture these differences.

9. Transit Vehicle Headway (seconds)

This variable represents the time difference between successive arrivals of a transit or ridesharing vehicle. In the case of transit vehicles, operationally they run on a schedule-basis or a headway-basis, and in each case, the next arrival can be predicted. This allows us to calculate both whether the delay caused by a transit vehicle is significant enough to affect the schedule and headway of the next vehicle, and also to estimate how many cycles will have returned to normal operation before the next delay event. In the case of ridesharing vehicles, if these vehicles can be identified in real-time prior to their stop, data can be collected on their frequency and analysis performed subsequently.

10. Initial Transit / Ridesharing Vehicle Position in Queue:

The position of the transit vehicle or ridesharing vehicle is important to this analysis. If a transit or ridesharing vehicle is situated at the back of the platoon, the delay it can impose on vehicles is minimized, whereas if a vehicle is at the front of the platoon, every vehicle behind it may be subject to some delay. The impacts of this position are investigated in this model.

Through an analysis of the impact of these variables on the results, we can then determine which factors have the greatest impact on imposed delay and travel times, and make the appropriate decisions to mitigate this delay where desirable. Later, a sensitivity analysis will be performed to identify which design factors have the largest impact on delay.

11. Number of passengers on vehicle:

The number of passengers expected on the vehicle (or the change in number of passengers expected per vehicle) influences a reduction in automotive traffic for the purposes of this analysis. This parameter is necessary for our analysis, as if traffic increased to match previous levels, any delay would make transit or ridesharing infeasible.

3.3 Case 1: Single Lane, Curb Side Stop

Case 1, the most simple case, involves the implementation of transit service on a single-lane arterial road, where the design of the stop involves a curb-side stop. In this scenario, automobile traffic is unable to overtake the transit vehicle during boarding or alighting action, and all delay experienced by the transit vehicle is imposed on the automobile traffic. This case represents the maximum delay imposed by transit on arterial traffic.

3.3.1 Baseline Scenario

For this baseline scenario, we will use Fisher Avenue in Ottawa, north of Baseline Road. This road is a single lane in each direction, with largely insufficient road widths to facilitate passing. The following real-world parameters will be used for this analysis:

Parameter	Value
Jam Density (k_j)	120 veh/km/lane
Free Flow Speed (u_f)	60 km/hr
Arrival Flow Rate (q_a)	851 vehicles / hour / direction
Vehicle Occupancy	1.6 passengers / vehicle
Transit Utilization	20 passengers / bus
Transit Headways	1200 seconds / bus (3 buses/hour)
Maximum Flow Rate (q_{max})	1800 vehicles/hour
Transit Vehicle Location in Queue	6
Departure Green Cycle	75
Departure Total Cycle	120
Link Distance	304 metres
Distance To Stop	299 metres
Transit Dwell Time	30 seconds

Table 2: Example Parameters - Fisher Avenue

The first step in the analysis is to determine the baseline scenario against which to run the delay analysis. Unencumbered by delays imposed by transit and between intersections, the given time a vehicle enters the network segment can be given by:

$$t_0 = \frac{3600}{u_o} \times (n - 1)$$

Equation 30: Network Segment Entrance Time- Case 1: Baseline

Where, u_o is the calculated speed (in m/s) on that segment from Equation 14: Inverted Greenshields' Speed-Flow Relation, and n is the vehicle's place in queue. Alternatively, this model supports the input of arrivals to occur at any time, even non-uniform arrivals, and could be used to model real-world traffic conditions by using real arrival data. This is notable, as the arrival rate of vehicles is likely to be highest immediately after the green signal begins at the origin (due to queuing), and lowest during the red signal (when only turning vehicles are entering the network link).

The time a vehicle exits the network is of the most interest, however, as this is the baseline from which each vehicle's delay will be calculated. Unencumbered by delays imposed by transit and between intersections, the given time a vehicle exists the network segment is given by the amount of time it takes to traverse the segment, plus the time it took the vehicle to enter the segment. Expressed as an equation, the exit time is given by:

$$t_f = t_0 + \frac{\Delta x}{u}$$

Equation 31: Network Segment Exit Time – Case 1: Baseline

Where, u is the calculated speed (in m/s) on that segment from Equation 14: Inverted Greenshields' Speed-Flow Relation, t_0 is the network segment entrance time (in seconds) and x is the length (in meters) of the network segment. t_f will now be the baseline time for which a vehicle exits the network segment, and we will compare this time to the future scenario. As an example, we will consider Fisher Ave. in Ottawa between McCooey Lane and the Experimental Farm Pathway. 24-hour traffic counts indicate that the Average Annual Daily Traffic is 16,000 cars [43]. Taking into consideration both directions, the 24-hour factor, the 12-hour factor, and the peak hour factor, we can conclude that peak-hour AM traffic is 851 cars per hour per direction [44], which is quite significant. If we consider that these vehicles are equally spaced, we get an arrival every 4.23 seconds. The length of this network link is 304m, and the posted speed limit is 50 km/h. We will assume that the free flow speed is 20% higher than the posted speed limit for the purposes of this analysis. At AM peak hour, assuming that the exit signal is always green, the arrivals and departures may look like the following:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	21.13
2	4.23	25.36
3	8.47	29.60
4	12.70	33.83
5	16.93	38.06
6	21.16	42.30
7	25.40	46.53
8	29.63	50.76
9	33.86	54.99
10	38.09	59.23

Table 3: Unimpeded Arrivals and Departures, Fisher Avenue

Now, consider the case where the signal at the exit turns red for 30 seconds at the 45 second mark of our analysis. Not counting for deceleration delay, the arrivals and departures may instead look like the following:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	21.13
2	4.23	25.36
3	8.47	29.60
4	12.70	33.83
5	16.93	38.06
6	21.16	42.30
7	25.40	76.53
8	29.63	80.76
9	33.86	84.99
10	38.09	89.23

Table 4: Unimpeded Arrivals and Departures with Red Signal, Fisher Avenue

The particular properties governing exits and entrances will be explored later, but this method will serve as a baseline for measuring the scenario where transit is implemented. For this scenario, actual traffic counts were also conducted during the AM peak on September 21, 2016 and yielded similar results as shown in Table 5. Because vehicles are not permitted to pass in this scenario, deterministic approximations should perform equally well to actual results if traffic flows are approximately the same.

Vehicle Number	Arrival (s)	Departure (s)
1	2.83	23.96
2	6.71	27.84
3	9.14	30.27
4	13.10	34.23
5	17.94	39.07
6	21.16	42.30
7	26.33	47.46
8	30.71	51.84
9	33.58	54.71
10	37.01	58.14

Table 5: Actual Arrival Times: Case 1: Baseline

3.3.2 Future Scenario

Calculating the future scenario is a more complicated calculation than the baseline scenario, as there are four separate traffic paradigms involved. They are as follows:

Initial Conditions: As in the baseline scenario, the network segment entrance time is the preliminary calculation for evaluation of a scenario, using rates where real-world data is unavailable. Setting the network segment entrance time to zero, and considering the segment entrance time is calculated as:

$$t_0 = \frac{3600}{u_0} \times (n - 1)$$

Equation 32: Network Segment Entrance Time: Case 2: Future

In this case, we expect the rate of vehicle arrivals, u_0 to be reduced by the expected ridership on the transit segment, resulting in slightly fewer arrivals. Considering the previous example of Fisher Avenue, we have assumed that three buses per hour will be added to the service, and that each bus will carry 20 people. These parameters would be projected by transportation planners designing the service. We should note also that we have assumed 1.6 passengers are in each vehicle. As a result, we have a traffic reduction

of 60 people per hour, which correlates to a reduction in 38 cars. The new arrival rate is thus 813 vehicles per hour per direction.

The arrival rate in this future scenario would thus look like the following in Table 6:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	20.96
2	4.43	25.39
3	8.86	29.82
4	13.28	34.24
5	17.71	38.67
6	22.14	43.10
7	26.57	47.53
8	30.99	51.95
9	35.42	56.38
10	39.85	60.81

Table 6: Arrivals and Departures, Fisher Avenue - Future

Comparing the arrivals for the first ten vehicles (with no red signal) from this future scenario to the baseline scenario developed earlier, we can see that there is an expected reduction in travel times due to the implementation of transit and the corresponding reduction in traffic volumes:

Baseline				Future				Difference
Vehicle Number	Arrival (s)	Departure (s)	Travel Time	Vehicle Number	Arrival (s)	Departure (s)	Travel Time	
1	0.00	21.13	21.13	1	0.00	20.96	20.96	-0.17
2	4.23	25.36	21.13	2	4.43	25.39	20.96	-0.17
3	8.47	29.60	21.13	3	8.86	29.82	20.96	-0.17
4	12.70	33.83	21.13	4	13.28	34.24	20.96	-0.17
5	16.93	38.06	21.13	5	17.71	38.67	20.96	-0.17
6	21.16	42.30	21.13	6	22.14	43.10	20.96	-0.17
7	25.40	46.53	21.13	7	26.57	47.53	20.96	-0.17
8	29.63	50.76	21.13	8	30.99	51.95	20.96	-0.17
9	33.86	54.99	21.13	9	35.42	56.38	20.96	-0.17
10	38.09	59.23	21.13	10	39.85	60.81	20.96	-0.17
				TOTAL				-2

Table 7: Comparison of Travel Times after Initial Conditions Change - Fisher

Baseline				Future				Difference
Vehicle Number	Arrival (s)	Departure (s)	Travel Time	Vehicle Number	Arrival (s)	Departure (s)	Travel Time	
1	0.00	21.13	21.13	1	0.00	20.96	20.96	-0.17
2	4.23	25.36	21.13	2	4.43	25.39	20.96	-0.17
3	8.47	29.60	21.13	3	8.86	29.82	20.96	-0.17
4	12.70	33.83	21.13	4	13.28	34.24	20.96	-0.17
5	16.93	38.06	21.13	5	17.71	38.67	20.96	-0.17
6	21.16	42.30	21.13	6	22.14	43.10	20.96	-0.17
7	25.40	46.53	21.13	7	26.57	47.53	20.96	-0.17
8	29.63	50.76	21.13	8	30.99	51.95	20.96	-0.17
9	33.86	54.99	21.13	9	35.42	56.38	20.96	-0.17
10	38.09	59.23	21.13	10	39.85	60.81	20.96	-0.17
				TOTAL				-2

Table 7 shows that for the first ten vehicles through an intersection, we should expect a net travel time savings of nearly two seconds. Extrapolated across all traffic for the day, this would amount to significant savings, and indeed, many transportation plans use this logic to support the implementation of transit, ridesharing, and microtransit projects in Canada. As discussed earlier, however, this approach to planning does not take into

account the negative externalities that are created by these services. The next step, queuing, will begin to develop this idea further.

Queueing: In the baseline scenario, the next step was to calculate the time required for each particular vehicle to reach the end of the road segment. In this case, however, some amount of queuing is expected, and as such, calculation of the length of the road to the start of the queue is required, and the distance to the queue can be given by the following relations:

Considering the assumption that transit vehicles can not overtake automobile traffic on a single lane road, vehicles entering the road segment ahead of the transit vehicle, and the transit vehicle itself experience no queue, and:

$$\Delta x_q = \Delta x_s$$

For vehicles entering the road segment behind the transit / ridesharing vehicle, we can calculate the location a vehicle will encounter the queue for each vehicle using the following:

$$\Delta x_{q0} = \Delta x_s - (n - n_t) \times \left(\frac{1000}{k_j}\right)$$

Equation 33: Distance to Formed Queue - Case 1: Future

Where, Δx_{q0} is the distance to the formed queue, Δx_s is the distance to the stop, n is the position a vehicle is from the front of the platoon, n_t is the position of the transit or ridesharing vehicle to the front of the platoon, and k_j is the jam density on the roadway.

In the example on Fisher Avenue, assuming the transit vehicle is 6th in queue, and stops for an infinitely long time (as the remainder of this analysis is developing) at a stop located 299m past the origin, the distance at which each vehicle arrives at the queue is shown here in Table 8.

Vehicle Number	Queue Start
1	299.00
2	299.00
3	299.00
4	299.00
5	299.00
6	299.00
7	290.67
8	282.33
9	274.00
10	265.67

Table 8: Queue Arrival Distance - Fisher Avenue

Next, the travel time to the start of the queue is calculated for each vehicle in sequence. Since vehicles may travel at their prescribed speed developed earlier, the time to the start of the queue is a function of just time and distance and is given by:

$$\Delta t_q = \frac{\Delta x_q}{u}$$

Equation 34: Travel time to Queue - Case 1: Future

The travel time from the arrival in the network link of interest to the queue, as well as the real-time arrival at the queue is now available and is shown in Table 9.

Vehicle Number	Arrival (s)	Queue Start	Time to Queue	Queue Arrival
1	0.00	299.00	20.61	20.61
2	4.43	299.00	20.61	25.04
3	8.86	299.00	20.61	29.47
4	13.28	299.00	20.61	33.90
5	17.71	299.00	20.61	38.33
6	22.14	299.00	20.61	42.75
7	26.57	290.67	20.04	46.61
8	30.99	282.33	19.47	50.46
9	35.42	274.00	18.89	54.31
10	39.85	265.67	18.32	58.17

Table 9: Queue Arrival Times - Fisher Avenue: Future

For vehicles that arrive after the transit / ridesharing vehicle, and also for the transit /ridesharing vehicle, there is some time that they will spend stopped behind the transit /ridesharing vehicle before the vehicle begins to move again. This “stopped time” is given by t_D as follows:

$$\Delta t_D = t_s - t_q + D, \quad t_q > t_s$$

Equation 35: Stopped time in Queue - Case 1: Future

From Table 8 and Table 9 above, we now know the position of all vehicles when the queue is released. This represents the maximum negative externality, and is shown in Table 10.

Vehicle Number	Arrival (s)	Departure (s)	Adjusted Departure	Travel Time	Difference
1	0.00	20.96	20.96	20.96	-0.17
2	4.43	25.39	25.39	20.96	-0.17
3	8.86	29.82	29.82	20.96	-0.17
4	13.28	34.24	34.24	20.96	-0.17
5	17.71	38.67	38.67	20.96	-0.17
6	22.14	73.10	73.10	50.96	29.83
7	26.57	77.95	77.95	51.39	30.25
8	30.99	82.81	82.81	51.81	30.68
9	35.42	87.66	87.66	52.24	31.10
10	39.85	92.51	92.51	52.66	31.53
TOTAL					152.53

Table 10: Travel Time of All Vehicles Including Queue

As can be seen in Table 10, the implementation of transit service in this case has resulted in significant negative externalities for the first ten vehicles. While the delay on vehicle 6 is unavoidable, because it is the transit vehicle making the stop, it imposes delays on the downstream vehicles causing them to be delayed as well. At this point, vehicles 6-10 have all been delayed significantly more individually than the reduction in traffic helped cumulatively, and, they have not yet reached the exit point of the arterial link. These negative externalities are still underestimated, however, as the vehicles have not yet exited the network link.

Queue Release: Next, calculation of the distance to the queue release is calculated. Because the length of the queue is greatest at the moment the transit or ridesharing vehicle begins to move again, the location of the end of the queue experienced by a particular vehicle is given by the following relation:

$$\Delta x_{qf} = \Delta x_s - (n - n_t - n_{max}) \times \left(\frac{q_{max} - q_f}{k_j} \right), \Delta x_{qf} < \Delta x_s$$

Equation 36: Length of Queue - Case 1: Future

Where n_{max} is given by:

$$n_{max} = \frac{D \times q_f}{3600}$$

Equation 37: Maximum Length of Queue - Case 1: Future

If $\Delta x_{qf} \leq \Delta x_{q0}$, then there is no queue, and therefore no delay.

This distance is variable from vehicle to vehicle, and the speed is equivalent to the speed at maximum flow rate [2], or half of the free-flow speed so the time required to traverse the distance from queue start to queue release is given by:

$$\Delta t_{qf} = \frac{2 \times \Delta x_{qf}}{u_f}$$

Equation 38: Time spent moving in queue - Case 1: Future

The length of time remaining for each vehicle to cross a road segment is the required time to travel from the queue release to the end of the road segment. This distance is variable due to shockwave theory, and can be calculated by:

$$\Delta x_e = x - \Delta x_{qf} - \Delta x_{q0}$$

Equation 39: Distance from Queue Release to End of Road Segment - Case 1: Future

The amount of time required to traverse this distance is given by:

$$\Delta t_e = \frac{\Delta x_e}{u}$$

Equation 40: Time from Queue Release to End of Road Segment - Case 1: Future

The total time required to traverse the road segment for a given vehicle i is the sum of all times calculated in the above steps, and is given by the formula:

$$t_i = \Delta t_{q,i} + \Delta t_{D,i} + \Delta t_{qf,i} + \Delta t_{e,i}$$

Equation 41: Total time required to Traverse Road Segment for Vehicle i - Case 1: Future

For the Fisher Avenue Scenario, our first 10 vehicles now have the following travel times, shown in Table 11.

Baseline		Future		Difference
Vehicle Number	Travel Time	Vehicle Number	Travel Time	
1	21.13	1	20.96	-0.17
2	21.13	2	20.96	-0.17
3	21.13	3	20.96	-0.17
4	21.13	4	20.96	-0.17
5	21.13	5	20.96	-0.17
6	21.13	6	50.96	29.83
7	21.13	7	51.39	30.25
8	21.13	8	51.81	30.68
9	21.13	9	52.24	31.10
10	21.13	10	52.66	31.53
			TOTAL	152.53

Table 11: Total Travel Time, Fisher Avenue

As can be seen here, with the imposed negative externalities, these first ten vehicles that we have been following through this analysis have now been delayed by 153 seconds, significantly less than the two-second benefit envisioned in the first step.

Lastly, we need to consider that the signalized control device at the exit may show some of these vehicles a red light where they would otherwise catch a green. In actuality, the traffic control devices at these locations are actuated, meaning they only change if a vehicle has triggered the sensor on the cross-street. For this example, instead, let us assume that the departure signal is green for 75 seconds, and has a 120-second cycle. In this case, 13 vehicles would exit the intersection normally. In the future state, only six vehicles receive a green signal, and the remaining 7 receive a red. These vehicles will

now be delayed by the traffic light, and this delay must be considered. The additional volume for the next light cycle becomes part of the signal analysis problem not discussed in this research. Overall, these 13 vehicles now have a delay as follows:

Baseline		Future		Difference
Vehicle Number	Travel Time	Vehicle Number	Travel Time	
1	21.13	1	20.96	-0.17
2	21.13	2	20.96	-0.17
3	21.13	3	20.96	-0.17
4	21.13	4	20.96	-0.17
5	21.13	5	20.96	-0.17
6	21.13	6	50.96	29.83
7	21.13	7	96.39	75.25
8	21.13	8	96.81	75.68
9	21.13	9	97.24	76.10
10	21.13	10	97.66	76.53
11	21.13	11	98.09	76.95
12	21.13	12	98.51	77.38
13	21.13	13	97.64	76.51
			TOTAL	563

Table 12: Total Travel Times, Fisher Avenue

It can now be seen that the transit vehicle imposed a delay of 563 seconds on the 13 vehicles that entered the network link during this cycle. If this traffic is indicative of the peak hour, we can generalize to solve for the overall hourly gain or loss. In this case, we have a 120 second light cycle, so there are 30 cycles per hour, and 3 of these cycles will impose this type of delay. Taking the net benefit from 27 cycles and the net loss from 3 cycles, and creating an hourly summary, we get the following:

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Travel Time	8242	seconds
Future Total Hourly Travel Time	9871	seconds
Difference	1630	seconds

As can be seen here, the implementation (or increase) of transit service on a single lane thoroughfare like Fisher Avenue in Ottawa results in a net hourly increase of travel times of 1,630 seconds, in this case nearly half an hour of delay.

In the next section, it is shown how design factors and traffic conditions affect the results of this analysis.

3.4 Case 2: Double Lane, Curb Side Stop

Case 2 involves the implementation of transit service on a double-lane arterial road, where the design of the stop involves a curb-side stop. In this scenario, automobile traffic is able to overtake the transit vehicle during boarding and alighting action, and only some delay from the transit vehicle is imposed on the traffic.

3.4.1 Baseline Scenario

For this baseline scenario, we will consider Taunton Rd. E in Oshawa, Ontario, West of Wilson Avenue. This road is a major arterial through Oshawa with two lanes in each direction. Since it is in a built up area, there are no bus bays provided for transit vehicles to stop. There is sufficient parking, however to permit ridesharing services to pick up

and drop off passengers off the roadside. In this geographical zone the posted speed limit is 60km/h, and there are currently two transit stops at 145m and 755m, and the total length of the roadway between signalized intersections is 800m. The following real-world parameters will be used for this analysis:

Parameter	Value
Jam Density (k_j)	100 veh/km/lane
Free Flow Speed (u_f)	72 km/hr
Arrival Flow Rate (q_a)	1 731 vehicles / hour
Vehicle Occupancy	1.6 passengers / vehicle
Transit Utilization	20 passengers / bus
Transit Headways	1200 seconds / bus (3 buses/hour)
Maximum Flow Rate (q_{max})	1800 vehicles/hour
Transit Vehicle Location in Queue	4
Departure Green Cycle	80
Departure Total Cycle	120
Link Distance	800 metres
Distance To Stop	145 metres / 755 metres
Transit Dwell Time	20 seconds

Table 13: Example Parameters - Taunton Ave E

The first step in the analysis is to determine the baseline scenario against which to run the delay analysis. Unencumbered by delays imposed by transit and between intersections, the given time a vehicle enters the network segment can be approximated by:

$$t_0 = \frac{3600}{u_o} \times (n - 1)$$

Equation 42: Network Segment Entrance Time - Case 2: Baseline

Where, u_o is the calculated speed (in m/s) on that segment from Equation 12: Inverted Greenshields' Speed-Flow Relation, and n is the vehicles place in the queue. Again, this model supports any arrival paradigm, including direct measurement, deterministic, and stochastic methods, as appropriate for the data available to do the analysis.

The time at which a vehicle would exit the network under these baseline conditions is of the most interest, however, as this is the baseline against which each vehicles' delay will be measured. Unencumbered by delays imposed by transit and between intersections, the given time a vehicle exits the network segment is given by the amount of time it takes to traverse the segment, plus the time it took the vehicle to enter the segment. Expressed as an equation, exit time is given by:

$$t_f = t_0 + \frac{\Delta x}{u}$$

Equation 43: Network Segment Exit Time - Case 2: Baseline

Where u is the calculated speed (in m/s) on the segment under analysis, derived from Equation 14, t_0 is the network segment entrance time (in seconds) and Δx is the length (in metres) of the network segment. t_f is now the baseline time that will be compared to the future scenario. In this example, as mentioned above, we are considering Taunton Road East between Wilson Avenue and Ritson Road. 24-hour traffic counts indicate the average annual daily traffic (AADT) is 32,570 cars [45]. Taking into consideration both directions, the 24-hour factor, the 12-hour factor, and the peak hour factor, we can conclude that peak-hour AM traffic Westbound is 1,731 cars per hour. If we consider these vehicles to be equally spaced, we get an arrival every 2.08 seconds. The length of this network link is exactly 800m, and the posted speed limit is 60 km/h, and as in Case 1, it is assumed that the free-flow speed is 20% higher than the posted speed limit. At the AM peak hour, assuming the exit signal is always green, the arrivals and departures are as follows:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	46.50

2	2.08	48.58
3	4.16	50.66
4	6.24	52.74
5	8.32	54.82
6	10.40	56.90
7	12.48	58.97
8	14.55	61.05
9	16.63	63.13
10	18.71	65.21

Table 14: Unimpeded Arrivals and Departures, Taunton Road

Now, considering the case where the signal at the exit turns red for 40 seconds at the 60 second mark of the analysis, Not counting for any deceleration delay, the arrivals and departures may instead look like the following:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	46.50
2	2.08	48.58
3	4.16	50.66
4	6.24	52.74
5	8.32	54.82
6	10.40	56.90
7	12.48	58.97
8	14.55	101.05
9	16.63	103.13
10	18.71	105.21

Table 15: Unimpeded Arrivals and Departures with Red Signal, Taunton Road

The particular properties governing exits and entrances will be refined in later sections.

3.4.2 Future Scenario

As with the first case, calculating the future scenario is more complicated, involving four separate paradigms. They are: **Initial Conditions**, **Queue Formation**, **Queue Release**, and **Final Conditions**.

Initial Conditions: As with the baseline scenario, the network segment entrance time is the preliminary calculation for the evaluation of a scenario. These entrance times can be

taken from real-world data, or estimated using deterministic or stochastic methods. Since actual arrival times are not available on Taunton Road, a deterministic method is used. Setting the first network entrance time to zero, the segment entrance time is calculated as:

$$t_0 = \frac{3600}{u_0} \times (n - 1)$$

Equation 44: Network Segment Entrance Time: Case 2: Future

In this case, we expect the rate of vehicle arrivals, u_0 , to be reduced by the expected ridership on the transit segment, resulting in slightly fewer arrivals. In this example, we have assumed that three buses per hour will be added to the service, and that each bus will carry 20 people. These parameters would be proposed, through other methods, by transportation planners designing the service. It is also assumed that there are an average of 1.6 travellers per automobile. As a result, we have a traffic reduction of 60 people per hour, corresponding to a reduction by 38 cars. The new arrival rate is thus, 1,694 vehicles.

The arrival rate in the future scenario would thus look like the following:

Vehicle Number	Arrival (s)	Departure (s)
1	0.00	46.31
2	2.13	48.43
3	4.25	50.56
4	6.38	82.68
5	8.50	85.02
6	10.63	87.35
7	12.75	89.69
8	14.88	92.03
9	17.00	94.36
10	19.13	96.70

Table 16: Arrivals and Departures - Taunton Road – Future

Comparing arrivals for the first ten vehicles (with no red signal) from this future scenario to the baseline scenario developed earlier, we can see that there is an expected reduction in travel times due to the implementation of transit and the corresponding reduction in traffic volumes:

Baseline			Future				Difference
Vehicle Number	Arrival (s)	Departure (s)	Vehicle Number	Arrival (s)	Departure (s)	Travel Time	
1	0.00	46.50	1	0.00	46.31	46.31	-0.19
2	2.08	48.58	2	2.13	48.43	46.31	-0.19
3	4.16	50.66	3	4.25	50.56	46.31	-0.19
4	6.24	52.74	4	6.38	52.68	46.31	-0.19
5	8.32	54.82	5	8.50	54.81	46.31	-0.19
6	10.40	56.90	6	10.63	56.93	46.31	-0.19
7	12.48	58.97	7	12.75	59.06	46.31	-0.19
8	14.55	61.05	8	14.88	61.18	46.31	-0.19
9	16.63	63.13	9	17.00	63.31	46.31	-0.19
10	18.71	65.21	10	19.13	65.43	46.31	-0.19

Table 17: Comparison of Travel Times after Initial Conditions Change – Taunton

Table 17 shows us that for the first ten vehicles to travel through this network link, there is an expected travel time savings of 0.19 seconds per vehicle, similar to the results found in Case 1. As discussed previously, this approach does not yet take into account the negative externalities created by the transit vehicles, and the next step develops this idea further.

Queuing: In the baseline scenario, the next step was to calculate the expected time required for each particular vehicle to reach the end of the road segment. In this case, however, some amount of queuing is expected, and as in Case 1, calculation of the distance to the start of the queue is required. This step is different from Case 1, however. Firstly, we hold the assumption that transit vehicles may not pass automobile traffic, and all vehicles entering the road segment ahead of the transit vehicle experience no queue:

$$\Delta x_q = \Delta x_s$$

For vehicles entering the road segment behind the transit vehicle, we can calculate the location a vehicle will encounter the queue. In contrast with Case 1, because vehicles can now pass the transit vehicle, the queue length must incorporate some number of vehicles exiting the queue as well. The location a vehicle will enter the queue is given by:

$$\Delta x_{q0} = \Delta x_s - \left[\sum_{n_t}^n \left(\frac{q_n - q_{max}}{q_n} \right) \times \left(\frac{1000}{k_j} \right) \right], q \geq q_{max}$$

Equation 45: Distance to Formed Queue - Case 2: Future

Where Δx_{q0} is the distance to the formed queue, Δx_s is the distance to the stop, n is the position a vehicle is from the front of the platoon, n_t is the position of the transit or ridesharing vehicle to the front of the platoon, q_n is the instantaneous arrival flow rate (i.e. 3600 / time behind previous vehicle), q_{max} is the departure flow rate (during the vehicle delay, this would be half of the max flow rate for both lanes), and k_j is the jam density on the roadway. In this case, if the instantaneous arrival flows are less than the departure flow, there is no queue.

In this example on Taunton Road, the flow rate, as determined deterministically from traffic counts, is less than the flow rate during the blockage, so there is no queue. Real-world conditions, however, are rarely deterministic. Actual arrival times during the PM peak will instead be used here. These counts were generated on November 29, 2016 for the Eastbound lanes, during a 25 second green cycle and are as follows:

Vehicle Number	Arrival (s)
1	1.63
2	2.18
3	2.75
4	4.19
5	5.65
6	6.88
7	7.81
8	8.63
9	9.13
10	10.44
11	10.69
12	12.93
13	13.18
14	15.35
15	15.66
16	16.69
17	18.31
18	18.65
19	19.23
20	20.82
21	22.00
22	22.65
23	24.61
24	25.01
25	25.84

Table 18: Traffic Counts - Case 2: Taunton Road

The corresponding queue starts are shown in Table 19.

Vehicle Number	Queue Start
1	755.00
2	755.00
3	755.00
4	745.48
5	744.13
6	742.20
7	739.53
8	736.58
9	732.83
10	731.10
11	726.73
12	727.33
13	722.95
14	723.38
15	719.15
16	716.73
17	715.78
18	711.63
19	708.08
20	707.05
21	705.00
22	701.63
23	701.53
24	697.53
25	694.60

Table 19: Queue Start Locations - Case 2: Taunton Road

Next, the travel time to the start of the queue is calculated for each vehicle in sequence. Since vehicles will travel at their initial speed developed earlier, the time to the start of the queue is a function of just time and distance and is given by:

$$\Delta t_q = \frac{\Delta x_q}{u}$$

Equation 46: Travel Time to Queue - Case 2: Future

The travel time from the arrival in the network link of interest to the start of the vehicle queue, as well as the real time arrival at the queue is now easily calculated and is shown for the first 10 vehicles in Table 20.

Vehicle Number	Time to Queue	Queue Arrival
1	43.70	45.33
2	43.70	45.88
3	43.70	46.45
4	43.15	47.34
5	43.07	48.72
6	42.96	49.84
7	42.81	50.62
8	42.63	51.26
9	42.42	51.55
10	42.32	52.76

Table 20: Time to Queue Start - Case 2: Future

Queue Release: Case 2 differs in the sense that the vehicles are able to continue moving during the dwell time, and the position of the queue release needs to be calculated in order to calculate vehicle travel time through the queue. This distance is variable by vehicle travel time is given by distance divided by half of the free-flow speed as follows:

$$\Delta t_{qf} = \frac{2 \times \Delta x_{qf}}{u_f}$$

Equation 47: Time Spent Moving in Queue - Case 2: Future

and for vehicles 25-31, the results are shown in Table 21.

Vehicle Number	Queue Distance	Queue Time
25	60.40	6.04
26	40.13	4.01
27	27.08	2.71
28	19.08	1.91
29	11.00	1.10
30	7.45	0.75
31	0.00	0.00

Table 21: Queue Travel Times - Case 2: Future

Final Conditions: The length of time remaining for each vehicle to cross a road segment is the required time to travel from the queue release to the end of the road segment. This distance is variable due to the effects of shockwave theory, and can be calculated by:

$$\Delta x_e = x - \Delta x_{qf} - \Delta x_{q0}$$

Equation 48: Distance from Queue Release to End of Road Segment - Case 2: Future

The amount of time required to traverse this distance is given by:

$$\Delta t_e = \frac{\Delta x_e}{u}$$

Equation 49: Time from Queue Release to End of Road Segment - Case 2: Future

The total time required to traverse the road segment for a given vehicle i is the sum of all times calculated in the above steps, and is given by the formula:

$$t_i = \Delta t_{q,i} + \Delta t_{qf,i} + \Delta t_{e,i}$$

Equation 50: Total time required to Traverse Road Segment for Vehicle i - Case 2: Future

For the Taunton Road example, our first 25 vehicles, representing the vehicles that would catch the departure green signal have the following travel times, as shown in Table 22.

Vehicle Number	Travel Time	Difference
1	46.31	-0.19
2	46.31	-0.19
3	46.31	-0.19
4	46.71	0.21
5	46.76	0.26
6	46.84	0.35
7	46.96	0.46
8	47.08	0.58
9	47.24	0.74
10	47.31	0.81
11	47.50	1.00
12	47.47	0.97
13	47.66	1.16
14	47.64	1.14
15	47.82	1.32
16	47.92	1.42
17	47.96	1.46
18	48.13	1.63
19	48.28	1.78
20	48.33	1.83
21	48.41	1.91
22	48.55	2.05
23	48.56	2.06
24	48.73	2.23
25	48.85	2.35
TOTAL		20.50

Table 22: Total Travel Time, Taunton Road

As can be seen here, the negative externalities for the first 25 vehicles in this analysis amount to 20.5 seconds, and as expected, are lower than that experienced by roads under a Case 1 analysis. Furthermore, unlike the Case 1 analysis, all vehicles that would have caught the green signal at departure in the initial case also catch the green signal in the future case.

It can be seen from Table 22 that the transit vehicle imposed a delay of 20.5 seconds cumulatively on the 25 vehicles that entered the network link during this cycle. Since

this is real data from the peak period, and we have a 120 second light cycle, and 3 out of 30 hourly cycles will impose this type of delay. Taking the net benefit from 27 cycles and the net loss from 3 cycles to create an hourly summary, we get the following:

Measure	Qty	Unit
Baseline Single Cycle Travel Time	1162.5	seconds
Future Unimpeded Travel Time	1157.6	seconds
Future Impeded Travel Time	1189.6	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	34874	seconds
Future Total Hourly Vehicle Travel Time	34825	seconds
Difference	-49	seconds

Table 23: Summary Results - Case 2: Taunton Road

As can be seen here, the implementation of transit service on a double-lane thoroughfare like Taunton Road results in a net decrease in all travel times, though this difference appears negligible.

3.5 Note on Left Lane Streetcar Operation

In the City of Toronto, Streetcars operate in the downtown core on a shared right-of-way, in the left lane exclusively. While these vehicles operate on four-lane roads (two in each direction), when they come to a stop, vehicles are not allowed to pass on the right, as that is where passengers board and disembark. Because of this, left-lane streetcar operation falls under a Case 1 analysis.

4 Sensitivity Analysis

4.1 Introduction

In this section, the sensitivity of the model with respect to stop location parameters, vehicle location parameters, flow rate parameters and transit operation parameters is discussed. Once these results have been discussed, overall results are presented in detail. While this analysis does not evaluate every possible scenario, these parameters were presumed to have the greatest impact on performance.

4.2 Sensitivity to Stop Location Parameters

The Highway Capacity Manual indicates that the space between the transit stop and the intersection has an effect on the intersection capacity. As a result, this model attempts to include variations in transit stop locations to attempt to evaluate whether or not the transit stop location can be predicted to have an impact on the overall travel times through the network link.

For Case 1, the transit stop is located 299m from the origin, and this scenario has been evaluated above during the description of the model. Moving the stop to a far-side stop near the origin would result in reduced intersection capacity, which is outside the scope of this research, so a mid-block stop was considered instead. Evaluating Case 1 for base conditions, we find that the location of the transit stop has no impact on a Case 1 analysis. See Table 24.

	Stop Distance from origin	hourly delay	Change
Base Conditions	299m	1630 sec	--
Relocated Transit Stop	150m	1630 sec	0%

Table 24: Impact of Transit Stop Location on Delay – Case 1

Similarly, when evaluating Case 2 for the impact of a change in transit stop location, we consider a stop located 755m from the origin and re-run the analysis for a transit stop located at 450m. Like Case 1, we find that the location of the transit stop has no impact on a Case 2 analysis. See Table 25.

	Stop Distance from origin	hourly delay	Change
Base Conditions	755m	-49 sec	--
Relocated Transit Stop	450m	-49 sec	0%

Table 25: Impact of Transit Stop Location on Delay - Case 2

These results hold true for all possible variations in other variables, and we can conclude that transit stop location has no impact on the capacity and travel times on arterial links. More detailed calculations can be found in Appendix A.

4.3 Sensitivity to Vehicle Position Parameters

Clearly, it should be expected that if a transit vehicle is located at the beginning of a traffic platoon, the vehicles located behind the transit vehicle will experience delay. As the vehicle moves towards the back of the platoon, the imposed delay on the traffic should be reduced, as there will be fewer vehicles behind the transit vehicle available to be delayed. This, of course is not true in situations where there is a constant flow of traffic, however, these situations typically do not happen because of signalized intersection control.

Using the base conditions for Case 1, we can evaluate the change of delay due to transit position for various positions in the platoon, in this case for position 2, 4, 6, 8, 10, and 12, as this baseline case presumes the end of the platoon at position 13.

Evaluating these scenarios, we find that as the transit vehicle is moved nearer the back of the platoon, we calculate increasing reductions in vehicle delay. See Table 26 and Figure 12.

Transit Position	Cycle Delay	Hourly
2	683	1990
4	623	1810
6	563	1630
8	454	1302
10	300	840
12	148	384

Table 26: Impact of Transit Vehicle Position on Delay - Case 1

Figure 12: Impact of Transit Vehicle Position on Delay - Case 1

Similar analysis performed on Case 2 yields similar results, though delays calculated for Case 2 are significantly smaller. We still find, however, that as the vehicle is moved

towards the back of the platoon, the total delay is reduced. See Table 27 and Figure 13.

Detailed results can be found in Appendix A.

Transit Position	Cycle Delay	Hourly
2	52	48
4	52	47
6	51	44
8	50	41
10	48	36
12	46	30
14	44	23
16	41	14

Table 27: Impact of Transit Vehicle Position on Delay - Case 2

Figure 13: Impact of Transit Vehicle Position on Delay - Case 2

As expected, as the transit vehicle is moved to the back of the platoon, it imposes a delay on fewer vehicles, and thus the total delay is reduced. It is clearly advantageous to practitioners to use control devices and management where possible to position vehicles at the end of the platoon.

4.4 Sensitivity to Flow Rate Parameters

Since the Highway Capacity Manual articulates that overall vehicle delay increases with an increase in flow rate, it is evident that an increase in flow rate will increase the delays experienced by vehicles on a roadway. This is, of course, independent from any transit-imposed delay. Evaluating a number of scenarios for various flow rates, we can quantify the impact of flow rate on imposed delay. For case 1, where there is a single lane, an evaluation of the imposed delay as a function of flow rate was performed for flow rates from 200 – 1600 in increments of 200. The results are shown in Table 28 and Figure 14.

Flow Rate	Cycle Delay	Hourly Delay
200	45	122
400	150	426
600	378	1097
800	562	1628
1000	672	1933
1200	899	2571
1400	1005	2820
1600	1151	3097

Table 28: Impact of Flow Rate on Delay - Case 1

Figure 14: Impact of Flow Rate on Delay - Case 2

As expected, as traffic flow rates increase, the delay imposed by transit on the traffic flow also increases, as the delay effect impacts more vehicles. This relationship is nearly linear, which is interesting as the traffic delay is not linear in the absence of traffic, using the formulas provided in the Highway Capacity Manual. In general, it can be said that as flow rate increases, delay increases proportionately.

This analysis is also performed for Case 2. Because vehicles are able to pass the transit vehicle in Case 2, any traffic flow below the maximum flow rate of the road will result in minimal delay due to merging. As a result, this analysis has been performed for flow rates from 1800 to 2600 in 200 vehicle increments. The results are shown in Table 29 and Figure 15 below.

Flow Rate	Cycle Delay	Hourly Delay
1800	51	28
2000	52	47
2200	52	28
2400	52	2
2600	50	-29

Table 29: Impact of Flow Rate on Delay - Case 2

Figure 15: Impact of Flow Rate on Delay - Case 2

Interestingly, as flow rate is increased, there is an initial increase in vehicle delay, followed by a reduction in vehicle delay. As flow rate is increased, the delay impacts on more downstream buses is more pronounced at first, but eventually the travel time benefits due to the reduction in traffic outweighs the impact of the imposed delay. This is a promising result for the feasibility of transit on the shared right-of-way. More detailed calculations are available in Appendix A.

4.5 Sensitivity to Transit Operation Parameters

Presumably, the impact of increasing the number of transit vehicles serving a particular roadway would result in a corresponding increase in hourly delay, as more delay events will occur per hour, however, the individual delays imposed by each transit vehicle should remain the same.

Analyzing this for case 1, using transit frequencies of 3, 6, 12, and 24 buses per hour, we get the following result as shown in

TV/hr	Cycle Delay	Hourly Delay
3	568	1630
6	560	3252
12	600	7045
24	589	14042

Table 30 and Figure 16.

TV/hr	Cycle Delay	Hourly Delay
3	568	1630
6	560	3252
12	600	7045
24	589	14042

Table 30: Impact of Transit Frequency on Delay: Case 1

Figure 16: Impact of Transit Frequency on Delay: Case 1

As expected, while the individual cycle delays stay relatively stable, the hourly delays are multiplicative of the number of vehicles per hour. If these delays impact the prior intersection, it could serve to significantly reduce the capacity of the signalized intersection, however, the main takeaway from this result is that if a vehicle imposes a delay on downstream traffic, additional vehicles will likely compound that delay.

For Case 2, as the transit vehicle frequency is increased, the corresponding delay is also increased, as the cumulative total of delays imposed exceed the cumulative traffic benefit from migrating passengers to transit. See

TV/hr	Cycle Delay	Hourly Delay
3	52	47
6	48	97
9	44	150
12	40	205
15	77	864
18	73	1045
21	70	1229
24	66	1415

Table 31 and Figure 17.

TV/hr	Cycle Delay	Hourly Delay
3	52	47
6	48	97
9	44	150
12	40	205
15	77	864
18	73	1045
21	70	1229
24	66	1415

Table 31: Impact of Transit Frequency on Delay - Case 2

Figure 17: Impact of Transit Frequency on Delay - Case 2

This result is perhaps the most interesting of the sensitivity analysis results. This indicates that for a given set of conditions, there comes a point at which the delay imposed by transit is increased significantly with frequency. The ultimate outcome of this result is that there is a set of traffic flows and transit demand that would logically result in a frequency that would cause additional delay. It is at this point that transportation planners and decision makers should consider a project to move to grade-separated transit facilities.

5 Results and Conclusions

5.1 Results

Based on the outcomes of the analyses discussed earlier, there are a number of major observations that can be made.

- 1. Transit, and by extension, ridesharing, is not effective at reducing congestion on single lane roadways unless the design permits vehicles to pass.** As can be seen from the results of Case 1, where vehicles are not permitted to pass the transit vehicle, all dwell time for transit is imposed on downstream traffic. In this case, the greater the frequency of transit service and the greater the traffic, the greater the impacts of the delay on traffic. Some of this delay may be mitigated with the implementation of bus bays, however, even still, the re-entry delay imposed by the vehicle is unrecoverable. Any Case 1 transit implementations should be done for reasons other than the reduction of congestion, and transit service should be planned for multi-lane arterials where traffic levels are consistent with commuter and arterial traffic volumes. Additionally, ridesharing services should not be permitted to stop on single-lane arterials for the purpose of boarding or alighting passengers under any circumstances unless specific infrastructure can be implemented in order to permit vehicles to pass (taxi stands, etc.).
- 2. Transit service, and by extension, ridesharing, is effective at reducing congestion on multi-lane roadways when the flow rate on both lanes is below the maximum flow rate for one lane.** In the case where the instantaneous flow rate on a roadway is less than the maximum flow rate of a single lane, there is no delay experienced by vehicles in the flow of traffic, with the exception of other traffic parameters like merge delay. As a result, if the transit service is being utilized, it will be successful at reducing congestion and provide an overall benefit to the users. As a corollary, traffic flows of this type are sufficiently small that addressing congestion

is likely not a concern, and from a planning standpoint, choice riders are not as likely to select transit as their travel mode.

3. **At traffic flows in excess of the Single Lane maximum flow, the delay imposed by transit and the traffic reduction caused by transit service are approximately equal.** For all situations analyzed in this research, implementation of transit in a Case 2 situation resulted in minimal benefit or delay, leading to the conclusion that transit service is not particularly effective at reducing congestion on Case 2 roadways. As a corollary, this analysis frequently used a passenger load of 20 passengers per transit vehicle. As this number increases, the delay remains nearly the same, but the benefit is increased. High ridership numbers and high-capacity buses could provide a measurable benefit if frequency of service is sufficiently high.
4. **Transit Stop location does not significantly impact imposed delay on network links. Near-side and Far-side stops affect the capacity of intersections, but does not impose delay on the link.** This is a complex observation. While the location of a transit stop has no effect on the performance of the network link in moving traffic, a near-side stop may reduce the capacity of an outgoing intersection and a far-side stop may reduce the capacity of the incoming intersection.
5. **Both the Victoria Transport Institute method and the method proposed by the Highway Capacity Manual underestimate the negative externalities in all cases, however, this is most pronounced in Case 1.** In all cases analyzed, both the VTI and HCM methods estimated an insignificant benefit or delay to vehicular traffic. The generalizations used by both documents with respect to deterministic analysis are not specific enough to calculate vehicle delay in the context of this research.

Furthermore, any microscopic analysis of multimodal traffic should consider rejecting these methods and either utilize the method articulated in this research or develop a more suitable delay model.

6. **At sufficiently high transit frequencies and flow volumes, there is significant unrecoverable delay on Case 2 roadways.** Perhaps the most important observation of this research is shown in Figure 17, where an increase in transit vehicle frequency from 3 to 6 per hour provided no significant impact to transit delay, however, the increase in frequency from 12 to 24 resulted in a massive delay impact. The implications of this observation is that for a particular traffic flow and a particular transit demand, there is a point at which it becomes beneficial to implement grade-separated transit (BRT, LRT, subway) in order to avoid exponential congestion. Predicting at which point a roadway will reach this point should be the key determining factor for transit planning, especially when it comes to grade-separated mass transit. Using computer software to determine which links in a particular transport network are subject to this condition could assist with route planning as well.

5.2 Conclusions

Evaluating the economic impact of marginal changes to transit service is an important component of a robust business case for improvements to transit. Municipal and Regional governments seeking a solution to the problem of congested streets should strongly consider adopting a similar approach when evaluating the efficacy of any proposed solution. Despite large improvements in computing power and sensor technology, benefits cases are still largely based on macroscopic data and

generalizations about demand and traffic corridors. Sensors and cameras used to count traffic and provide adaptive transportation system management can also be used to time-stamp and geo-code arrivals and departures on network links. The aggregation of this data and subsequent analysis can be a very powerful decision-making tool for transportation infrastructure improvement decision making.

The framework developed here can be used at a number of planning and engineering levels, from outdenting roads to facilitate bus bays and taxi stands, to determining the point at which only grade-separated transit will provide congestion relief. This procedure, and related microscopic data-collection techniques will be required, as well as a similar procedure for signalized intersections. Using this real-world data with the methodology presented in this research would permit the use of GIS tools to identify where infrastructure money is best spent to serve the citizens of urban areas.

By using these tools to make transit decisions, we can provide clear data in support of various improvements, including for grade-separated transit, limiting the number of cancelled projects and ineffective projects that are built. Further, because this methodology supports application with real-time and real-world traffic data, a calibrated model for each planning scenario and associated errors caused by extrapolation can be eliminated, and much of the work can be accomplished automatically.

It should be noted here that both VISSIM and AIMSUN were used extensively to validate the processes involved in this research. Ultimately, the complexities of the traffic models implemented in VISSIM and AIMSUN necessarily mirrored the results of this analysis and yielded the exact same results, with significant additional modelling required.

5.3 Contributions of this Research

The major contribution of this research concerns the articulation of a usable framework that would permit Municipal and Regional governments to evaluate various improvements in terms of transportation user benefits on a microscopic scale. Conversely, existing sensors and cameras could be interfaced with a data warehouse and business analytics suite to continuously evaluate or propose candidate routes according to this framework.

More specifically, this research contributes the following:

1. Using first principles, this research identified that to reduce congestion and increase aggregate user benefits on a mixed use multi-lane arterial, the platoon of cars travelling through the network link must be sufficiently small. That is, if the platoon is sufficiently small and there is adequate time-space at the back of the platoon, all vehicles that have delay imposed on them by transit service will have sufficient time to recover all of their delay, and pass through the forward intersection under the same green condition.
2. In the condition when the platoon is too long or transit service is too frequent, delay imposed by the transit vehicles will be significant enough to force vehicles to join the platoon of vehicles behind them, and will be unable to pass through the intersection under the same green condition as they would have unimpeded. In certain traffic conditions, this may cause delays that exceed any user benefits received from a decrease in auto users, and the effectiveness of this measure should be evaluated using this framework.

3. In the case where the improvement being measured will operate in its own right of way, in a BRT or LRT mode, for example, the transit service must carry sufficient quantities of passengers to replace the reduction in overall road capacity on the link, inclusive of pedestrian and other traffic impacts. Likewise, it can be determined at which point a grade-separated transit service will be more effective at reducing congestion than an increase in shared-right of way transit or ridesharing services.
4. Where the objective is not to reduce congestion or maximize user benefit, such as to increase accessibility or mobility, or for urban planning reasons, there may be an economic cost to implementing or augmenting transit facilities in terms of transportation user benefits. These benefits need to be considered during feasibility studies.

5.4 Future Research

The results of this research demonstrate that a network link analysis approach to evaluating the impact of a marginal change in transit infrastructure leads to a more robust analysis of the effectiveness of transit with respect to transportation user benefits. This framework lends itself to a number of future research topics, suited to better evaluating the impact of various transportation network changes on the overall transportation economy. In particular:

Most traffic count efforts do not count vehicles as discrete units with entry and exit time stamps. Leveraging camera and sensor data at intersections, data can be generated for each traffic light cycle on a discrete vehicle basis allowing for a more detailed analysis

of the effects of various changes to infrastructure and traffic patterns. How to accomplish this and aggregate the data is not clear and could be evaluated further.

This methodology can and should be expanded to evaluate various other effects that would require discrete vehicle data to analyze. For example, the effect of heavy trucks on transportation user benefits, the effect of pedestrians on user benefits, and the effect that a marginal change in transit service has on trucking volumes, to name a few examples.

The methodology here and its results can be incorporated into an adaptive urban bimodal transportation system management model. Mucsi notes that to increase transit ridership and modal share, the relative advantage of transit travel times compared to car travel times needs to improve, and his research utilizes fuzzy logic to improve traffic control to achieve these ends through a transit queue jump methodology [8]. Combining this approach with a real-time user benefits approach would serve to minimize travel times for all users and maximize the total passenger throughput on the network, thereby maximizing the economic benefit realized through the use of the transportation network. These findings could help revolutionize traffic control methodologies, but would require significant professional and academic effort to execute. This problem could spawn a number of PhD dissertations and professional publications.

Lastly, as connected vehicles and autonomous vehicles become pervasive throughout society, their performance and organization will have to be heavily analyzed to ensure the perceived benefit is realized. One particular advantage of these vehicles is that they would be able to manage dynamic speed limits and lessen the impact of any unavoidable delay caused by other vehicles operating in a different paradigm. This methodology and

corresponding follow-up research could serve to help program autonomous vehicles and develop operating rules that would thus increase the capacity of arterial roads.

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Appendix A: Supporting Data And Tables

Case 1: Modify Stop Position

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	299
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline				
Vehicle Number	Arrival (s)	Travel Time	Travel Time	Difference
1	0.00	21.13	20.96	-0.17
2	4.23	21.13	20.96	-0.17
3	8.47	21.13	20.96	-0.17
4	12.70	21.13	20.96	-0.17
5	16.93	21.13	20.96	-0.17
6	21.16	21.13	50.96	29.83
7	25.40	21.13	96.39	75.25
8	29.63	21.13	96.81	75.68
9	33.86	21.13	97.24	76.10
10	38.09	21.13	97.66	76.53
11	42.33	21.13	98.09	76.95
12	46.56	21.13	98.51	77.38
13	50.79	21.13	97.64	76.51
			TOTAL	563

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9871	seconds
Difference	1630	seconds

Victoria Transport Institute		
Vehicle Number	Travel Time	Difference
1	21.02	-0.11
2	21.02	-0.11
3	21.02	-0.11
4	21.02	-0.11
5	21.02	-0.11
6	21.02	-0.11
7	21.02	-0.11
8	21.02	-0.11
9	21.02	-0.11
10	21.02	-0.11
11	21.02	-0.11
12	21.02	-0.11
13	21.02	-0.11
TOTAL		-1.4

Highway Capacity Manual		
Vehicle Number	Travel Time	Difference
1	20.96	-0.17
2	20.96	-0.17
3	20.96	-0.17
4	20.96	-0.17
5	20.96	-0.17
6	20.96	-0.17
7	20.96	-0.17
8	20.96	-0.17
9	20.96	-0.17
10	20.96	-0.17
11	20.96	-0.17
12	20.96	-0.17
13	20.96	-0.17
TOTAL		-2.2

Modified Stop Location to Midblock

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline				
Vehicle Number	Arrival (s)	Travel Time	Travel Time	Difference
1	0.00	21.13	20.96	-0.17
2	4.23	21.13	20.96	-0.17
3	8.47	21.13	20.96	-0.17
4	12.70	21.13	20.96	-0.17
5	16.93	21.13	20.96	-0.17
6	21.16	21.13	50.96	29.83
7	25.40	21.13	96.39	75.25
8	29.63	21.13	96.81	75.68
9	33.86	21.13	97.24	76.10
10	38.09	21.13	97.66	76.53
11	42.33	21.13	98.09	76.95
12	46.56	21.13	98.51	77.38
13	50.79	21.13	97.64	76.51
			TOTAL	563

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9871	seconds
Difference	1630	seconds

Victoria Transport Institute		
Vehicle Number	Travel Time	Difference
1	21.02	-0.11
2	21.02	-0.11
3	21.02	-0.11
4	21.02	-0.11
5	21.02	-0.11
6	21.02	-0.11
7	21.02	-0.11
8	21.02	-0.11
9	21.02	-0.11
10	21.02	-0.11
11	21.02	-0.11
12	21.02	-0.11
13	21.02	-0.11
TOTAL		-1.4

Highway Capacity Manual		
Vehicle Number	Travel Time	Difference
1	20.96	-0.17
2	20.96	-0.17
3	20.96	-0.17
4	20.96	-0.17
5	20.96	-0.17
6	20.96	-0.17
7	20.96	-0.17
8	20.96	-0.17
9	20.96	-0.17
10	20.96	-0.17
11	20.96	-0.17
12	20.96	-0.17
13	20.96	-0.17
TOTAL		-2.2

Note: A change in stop location has no effect on the delay in Case 1.

Case 2: Modify Stop Position

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	61.9	Speed (km/h)	62.2
Speed (m/s)	17.2	Speed (m/s)	17.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1694
Flow (pc/hour)	1731	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	755
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	14
		Number of Lanes	2
		Transit Vehicle Stop Time	47.34
		Transit Vehicle Release Time	77.34

Measure	Qty	Unit
Baseline Single Cycle Travel Time	1162.5	seconds
Future Unimpeded Travel Time	1157.6	seconds
Future Impeded Travel Time	1189.6	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	34874	seconds
Future Total Hourly Vehicle Travel Time	34825	seconds
Difference	-49	seconds

Baseline		Future		
Vehicle Number	Travel Time	Vehicle Number	Travel Time	Difference
1	46.50	1	46.31	-0.19
2	46.50	2	46.31	-0.19
3	46.50	3	46.31	-0.19
4	46.50	4	46.71	0.21
5	46.50	5	46.76	0.26
6	46.50	6	46.84	0.35
7	46.50	7	46.96	0.46
8	46.50	8	47.08	0.58
9	46.50	9	47.24	0.74
10	46.50	10	47.31	0.81
11	46.50	11	47.50	1.00
12	46.50	12	47.47	0.97
13	46.50	13	47.66	1.16
14	46.50	14	47.64	1.14
15	46.50	15	47.82	1.32
16	46.50	16	47.92	1.42
17	46.50	17	47.96	1.46
18	46.50	18	48.13	1.63
19	46.50	19	48.28	1.78
20	46.50	20	48.33	1.83
21	46.50	21	48.41	1.91
22	46.50	22	48.55	2.05
23	46.50	23	48.56	2.06
24	46.50	24	48.73	2.23
25	46.50	25	48.85	2.35
		TOTAL		20.50

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	46.37	-0.12	1	46.31	-0.19
2	46.37	-0.12	2	46.31	-0.19
3	46.37	-0.12	3	46.31	-0.19
4	46.37	-0.12	4	46.31	-0.19
5	46.37	-0.12	5	46.31	-0.19
6	46.37	-0.12	6	46.31	-0.19
7	46.37	-0.12	7	46.31	-0.19
8	46.37	-0.12	8	46.31	-0.19
9	46.37	-0.12	9	46.31	-0.19
10	46.37	-0.12	10	46.31	-0.19
11	46.37	-0.12	11	46.31	-0.19
12	46.37	-0.12	12	46.31	-0.19
13	46.37	-0.12	13	46.31	-0.19
14	46.37	-0.12	14	46.31	-0.19
15	46.37	-0.12	15	46.31	-0.19
16	46.37	-0.12	16	46.31	-0.19
17	46.37	-0.12	17	46.31	-0.19
18	46.37	-0.12	18	46.31	-0.19
19	46.37	-0.12	19	46.31	-0.19
20	46.37	-0.12	20	46.31	-0.19
21	46.37	-0.12	21	46.31	-0.19
22	46.37	-0.12	22	46.31	-0.19
23	46.37	-0.12	23	46.31	-0.19
24	46.37	-0.12	24	46.31	-0.19
25	46.37	-0.12	25	46.31	-0.19
	TOTAL	-2.7		TOTAL	-4.3

Modified Stop Location to Midblock (450m)

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	61.9	Speed (km/h)	62.2
Speed (m/s)	17.2	Speed (m/s)	17.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1694
Flow (pc/hour)	1731	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	14
		Number of Lanes	2
		Transit Vehicle Stop Time	29.69
		Transit Vehicle Release Time	59.69

Measure	Qty	Unit
Baseline Single Cycle Travel Time	1162.5	seconds
Future Unimpeded Travel Time	1157.6	seconds
Future Impeded Travel Time	1189.6	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	34874	seconds
Future Total Hourly Vehicle Travel Time	34825	seconds
Difference	-49	seconds

Baseline		Future		
Vehicle Number	Travel Time	Vehicle Number	Travel Time	Difference
1	46.50	1	46.31	-0.19
2	46.50	2	46.31	-0.19
3	46.50	3	46.31	-0.19
4	46.50	4	46.71	0.21
5	46.50	5	46.76	0.26
6	46.50	6	46.84	0.35
7	46.50	7	46.96	0.46
8	46.50	8	47.08	0.58
9	46.50	9	47.24	0.74
10	46.50	10	47.31	0.81
11	46.50	11	47.50	1.00
12	46.50	12	47.47	0.97
13	46.50	13	47.66	1.16
14	46.50	14	47.64	1.14
15	46.50	15	47.82	1.32
16	46.50	16	47.92	1.42
17	46.50	17	47.96	1.46
18	46.50	18	48.13	1.63
19	46.50	19	48.28	1.78
20	46.50	20	48.33	1.83
21	46.50	21	48.41	1.91
22	46.50	22	48.55	2.05
23	46.50	23	48.56	2.06
24	46.50	24	48.73	2.23
25	46.50	25	48.85	2.35
		TOTAL		20.50

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	46.37	-0.12	1	46.31	-0.19
2	46.37	-0.12	2	46.31	-0.19
3	46.37	-0.12	3	46.31	-0.19
4	46.37	-0.12	4	46.31	-0.19
5	46.37	-0.12	5	46.31	-0.19
6	46.37	-0.12	6	46.31	-0.19
7	46.37	-0.12	7	46.31	-0.19
8	46.37	-0.12	8	46.31	-0.19
9	46.37	-0.12	9	46.31	-0.19
10	46.37	-0.12	10	46.31	-0.19
11	46.37	-0.12	11	46.31	-0.19
12	46.37	-0.12	12	46.31	-0.19
13	46.37	-0.12	13	46.31	-0.19
14	46.37	-0.12	14	46.31	-0.19
15	46.37	-0.12	15	46.31	-0.19
16	46.37	-0.12	16	46.31	-0.19
17	46.37	-0.12	17	46.31	-0.19
18	46.37	-0.12	18	46.31	-0.19
19	46.37	-0.12	19	46.31	-0.19
20	46.37	-0.12	20	46.31	-0.19
21	46.37	-0.12	21	46.31	-0.19
22	46.37	-0.12	22	46.31	-0.19
23	46.37	-0.12	23	46.31	-0.19
24	46.37	-0.12	24	46.31	-0.19
25	46.37	-0.12	25	46.31	-0.19
TOTAL		-2.7	TOTAL		-4.3

Note: A change in Stop Location has no effect on the delay in Case 2.

Case 1: Modify Transit Vehicle Position in Queue

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	299
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline				
Vehicle Number	Arrival (s)	Travel Time	Travel Time	Difference
1	0.00	21.13	20.96	-0.17
2	4.23	21.13	20.96	-0.17
3	8.47	21.13	20.96	-0.17
4	12.70	21.13	20.96	-0.17
5	16.93	21.13	20.96	-0.17
6	21.16	21.13	50.96	29.83
7	25.40	21.13	96.39	75.25
8	29.63	21.13	96.81	75.68
9	33.86	21.13	97.24	76.10
10	38.09	21.13	97.66	76.53
11	42.33	21.13	98.09	76.95
12	46.56	21.13	98.51	77.38
13	50.79	21.13	97.64	76.51
			TOTAL	563

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9871	seconds
Difference	1630	seconds

Victoria Transport Institute		
Vehicle Number	Travel Time	Difference
1	21.02	-0.11
2	21.02	-0.11
3	21.02	-0.11
4	21.02	-0.11
5	21.02	-0.11
6	21.02	-0.11
7	21.02	-0.11
8	21.02	-0.11
9	21.02	-0.11
10	21.02	-0.11
11	21.02	-0.11
12	21.02	-0.11
13	21.02	-0.11
TOTAL		-1.4

Highway Capacity Manual		
Vehicle Number	Travel Time	Difference
1	20.96	-0.17
2	20.96	-0.17
3	20.96	-0.17
4	20.96	-0.17
5	20.96	-0.17
6	20.96	-0.17
7	20.96	-0.17
8	20.96	-0.17
9	20.96	-0.17
10	20.96	-0.17
11	20.96	-0.17
12	20.96	-0.17
13	20.96	-0.17
TOTAL		-2.2

Transit Vehicle Position Changed to 2

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	2
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.96		-0.17
2	4.23	21.13	4.43	50.96		29.83
3	8.47	21.13	8.86	51.39		30.25
4	12.70	21.13	13.28	51.81		30.68
5	16.93	21.13	17.71	52.24		31.10
6	21.16	21.13	22.14	52.66		31.53
7	25.40	21.13	26.57	98.09		76.95
8	29.63	21.13	30.99	98.51		77.38
9	33.86	21.13	35.42	97.64		76.51
10	38.09	21.13	39.85	95.96		74.83
11	42.33	21.13	44.28	95.96		74.83
12	46.56	21.13	48.71	95.96		74.83
13	50.79	21.13	53.13	95.96		74.83
TOTAL						683

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	958.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	10231	seconds
Difference	1990	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 4

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.96		-0.17
2	4.23	21.13	4.43	20.96		-0.17
3	8.47	21.13	8.86	20.96		-0.17
4	12.70	21.13	13.28	50.96		29.83
5	16.93	21.13	17.71	51.39		30.25
6	21.16	21.13	22.14	51.81		30.68
7	25.40	21.13	26.57	97.24		76.10
8	29.63	21.13	30.99	97.66		76.53
9	33.86	21.13	35.42	98.09		76.95
10	38.09	21.13	39.85	98.51		77.38
11	42.33	21.13	44.28	97.64		76.51
12	46.56	21.13	48.71	95.96		74.83
13	50.79	21.13	53.13	95.96		74.83
TOTAL						623

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	898.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	10051	seconds
Difference	1810	seconds

Note: Delay Decreases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 8

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	8
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.96		-0.17
2	4.23	21.13	4.43	20.96		-0.17
3	8.47	21.13	8.86	20.96		-0.17
4	12.70	21.13	13.28	20.96		-0.17
5	16.93	21.13	17.71	20.96		-0.17
6	21.16	21.13	22.14	20.96		-0.17
7	25.40	21.13	26.57	20.96		-0.17
8	29.63	21.13	30.99	95.96		74.83
9	33.86	21.13	35.42	96.39		75.25
10	38.09	21.13	39.85	96.81		75.68
11	42.33	21.13	44.28	97.24		76.10
12	46.56	21.13	48.71	97.66		76.53
13	50.79	21.13	53.13	98.09		76.95
TOTAL						454

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	728.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9543	seconds
Difference	1302	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 10

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	10
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.96		-0.17
2	4.23	21.13	4.43	20.96		-0.17
3	8.47	21.13	8.86	20.96		-0.17
4	12.70	21.13	13.28	20.96		-0.17
5	16.93	21.13	17.71	20.96		-0.17
6	21.16	21.13	22.14	20.96		-0.17
7	25.40	21.13	26.57	20.96		-0.17
8	29.63	21.13	30.99	20.96		-0.17
9	33.86	21.13	35.42	20.96		-0.17
10	38.09	21.13	39.85	95.96		74.83
11	42.33	21.13	44.28	96.39		75.25
12	46.56	21.13	48.71	96.81		75.68
13	50.79	21.13	53.13	97.24		76.10
TOTAL						300

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	575.0	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9082	seconds
Difference	840	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 12

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	12
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.96		-0.17
2	4.23	21.13	4.43	20.96		-0.17
3	8.47	21.13	8.86	20.96		-0.17
4	12.70	21.13	13.28	20.96		-0.17
5	16.93	21.13	17.71	20.96		-0.17
6	21.16	21.13	22.14	20.96		-0.17
7	25.40	21.13	26.57	20.96		-0.17
8	29.63	21.13	30.99	20.96		-0.17
9	33.86	21.13	35.42	20.96		-0.17
10	38.09	21.13	39.85	20.96		-0.17
11	42.33	21.13	44.28	20.96		-0.17
12	46.56	21.13	48.71	95.96		74.83
13	50.79	21.13	53.13	96.39		75.25
TOTAL						148

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	422.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	8626	seconds
Difference	384	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Case 2: Modify Transit Vehicle Position In Queue

Baseline Conditions (Instantaneous Flow Rate = 2000)

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	31.81
		Transit Vehicle Release Time	61.81

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	60.2	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1976	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1963
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	7020

Baseline		Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	
1	0.00	48.00	0.00	-0.22
2	1.80	48.00	1.83	-0.22
3	3.60	48.00	3.67	-0.22
4	5.40	48.00	5.50	0.16
5	7.20	48.00	7.34	0.22
6	9.00	48.00	9.17	0.29
7	10.80	48.00	11.01	0.40
8	12.60	48.00	12.84	0.52
9	14.40	48.00	14.68	0.67
10	16.20	48.00	16.51	0.74
11	18.00	48.00	18.34	0.92
12	19.80	48.00	20.18	0.89
13	21.60	48.00	22.01	1.07
14	23.40	48.00	23.85	1.05
15	25.20	48.00	25.68	1.22
16	27.00	48.00	27.52	1.32
17	28.80	48.00	29.35	1.36
18	30.60	48.00	31.18	41.52
			TOTAL	51.68

Victoria Transport Institute			Highway Capacity Manual	
Vehicle Number	Travel Time	Difference	Vehicle Number	Difference
1	47.86	-0.14	1	-0.22
2	47.86	-0.14	2	-0.22
3	47.86	-0.14	3	-0.22
4	47.86	-0.14	4	-0.22
5	47.86	-0.14	5	-0.22
6	47.86	-0.14	6	-0.22
7	47.86	-0.14	7	-0.22
8	47.86	-0.14	8	-0.22
9	47.86	-0.14	9	-0.22
10	47.86	-0.14	10	-0.22
11	47.86	-0.14	11	-0.22
12	47.86	-0.14	12	-0.22
13	47.86	-0.14	13	-0.22
14	47.86	-0.14	14	-0.22
15	47.86	-0.14	15	-0.22
16	47.86	-0.14	16	-0.22
17	47.86	-0.14	17	-0.22
18	47.86	-0.14	18	-0.22
TOTAL		-2.6		-4.0

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	915.7	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25967	seconds
Difference	47	seconds

Transit Vehicle Position Changed to 2

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	2
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	28.44
		Transit Vehicle Release Time	58.44

Baseline		Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	
1	0.00	48.00	0.00	-0.22
2	1.80	48.00	1.83	-0.04
3	3.60	48.00	3.67	0.10
4	5.40	48.00	5.50	0.16
5	7.20	48.00	7.34	0.22
6	9.00	48.00	9.17	0.29
7	10.80	48.00	11.01	0.40
8	12.60	48.00	12.84	0.52
9	14.40	48.00	14.68	0.67
10	16.20	48.00	16.51	0.74
11	18.00	48.00	18.34	0.92
12	19.80	48.00	20.18	0.89
13	21.60	48.00	22.01	1.07
14	23.40	48.00	23.85	1.05
15	25.20	48.00	25.68	1.22
16	27.00	48.00	27.52	1.32
17	28.80	48.00	29.35	1.36
18	30.60	48.00	31.18	41.52
		TOTAL		52.19

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	916.2	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25968	seconds
Difference	48	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 6

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	35.28
		Transit Vehicle Release Time	65.28

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	48.29	0.29
7	10.80	48.00	11.01	48.40	0.40
8	12.60	48.00	12.84	48.52	0.52
9	14.40	48.00	14.68	48.67	0.67
10	16.20	48.00	16.51	48.74	0.74
11	18.00	48.00	18.34	48.92	0.92
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		50.86

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	914.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25964	seconds
Difference	44	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 8

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	8
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	38.62
		Transit Vehicle Release Time	68.62

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	47.78	-0.22
7	10.80	48.00	11.01	47.78	-0.22
8	12.60	48.00	12.84	48.52	0.52
9	14.40	48.00	14.68	48.67	0.67
10	16.20	48.00	16.51	48.74	0.74
11	18.00	48.00	18.34	48.92	0.92
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
				TOTAL	49.72

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	913.7	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25961	seconds
Difference	41	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 10

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	10
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	41.96
		Transit Vehicle Release Time	71.96

Baseline			Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	47.78	-0.22
7	10.80	48.00	11.01	47.78	-0.22
8	12.60	48.00	12.84	47.78	-0.22
9	14.40	48.00	14.68	47.78	-0.22
10	16.20	48.00	16.51	48.74	0.74
11	18.00	48.00	18.34	48.92	0.92
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		48.09

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	912.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25956	seconds
Difference	36	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 12

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	12
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	45.40
		Transit Vehicle Release Time	75.40

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	47.78	-0.22
7	10.80	48.00	11.01	47.78	-0.22
8	12.60	48.00	12.84	47.78	-0.22
9	14.40	48.00	14.68	47.78	-0.22
10	16.20	48.00	16.51	47.78	-0.22
11	18.00	48.00	18.34	47.78	-0.22
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		45.98

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	910.0	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25950	seconds
Difference	30	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 14

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	14
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	48.83
		Transit Vehicle Release Time	78.83

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	47.78	-0.22
7	10.80	48.00	11.01	47.78	-0.22
8	12.60	48.00	12.84	47.78	-0.22
9	14.40	48.00	14.68	47.78	-0.22
10	16.20	48.00	16.51	47.78	-0.22
11	18.00	48.00	18.34	47.78	-0.22
12	19.80	48.00	20.18	47.78	-0.22
13	21.60	48.00	22.01	47.78	-0.22
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
				TOTAL	43.58

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	907.6	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25943	seconds
Difference	23	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Transit Vehicle Position Changed to 16

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	16
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	52.10
		Transit Vehicle Release Time	82.10

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	47.78	-0.22
5	7.20	48.00	7.34	47.78	-0.22
6	9.00	48.00	9.17	47.78	-0.22
7	10.80	48.00	11.01	47.78	-0.22
8	12.60	48.00	12.84	47.78	-0.22
9	14.40	48.00	14.68	47.78	-0.22
10	16.20	48.00	16.51	47.78	-0.22
11	18.00	48.00	18.34	47.78	-0.22
12	19.80	48.00	20.18	47.78	-0.22
13	21.60	48.00	22.01	47.78	-0.22
14	23.40	48.00	23.85	47.78	-0.22
15	25.20	48.00	25.68	47.78	-0.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		40.86

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	904.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25934	seconds
Difference	14	seconds

Note: Delay Increases when transit position is decreased – no change to Victoria Transport Institute or Highway Capacity Manual Predictions

Case 1: Modify Flow Rate

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	299
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline				
Vehicle Number	Arrival (s)	Travel Time	Travel Time	Difference
1	0.00	21.13	20.96	-0.17
2	4.23	21.13	20.96	-0.17
3	8.47	21.13	20.96	-0.17
4	12.70	21.13	20.96	-0.17
5	16.93	21.13	20.96	-0.17
6	21.16	21.13	50.96	29.83
7	25.40	21.13	96.39	75.25
8	29.63	21.13	96.81	75.68
9	33.86	21.13	97.24	76.10
10	38.09	21.13	97.66	76.53
11	42.33	21.13	98.09	76.95
12	46.56	21.13	98.51	77.38
13	50.79	21.13	97.64	76.51
			TOTAL	563

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9871	seconds
Difference	1630	seconds

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Vehicle Number	Travel Time	Difference
1	21.02	-0.11
2	21.02	-0.11
3	21.02	-0.11
4	21.02	-0.11
5	21.02	-0.11
6	21.02	-0.11
7	21.02	-0.11
8	21.02	-0.11
9	21.02	-0.11
10	21.02	-0.11
11	21.02	-0.11
12	21.02	-0.11
13	21.02	-0.11
TOTAL		-1.4

Highway Capacity Manual		
Vehicle Number	Travel Time	Difference
1	20.96	-0.17
2	20.96	-0.17
3	20.96	-0.17
4	20.96	-0.17
5	20.96	-0.17
6	20.96	-0.17
7	20.96	-0.17
8	20.96	-0.17
9	20.96	-0.17
10	20.96	-0.17
11	20.96	-0.17
12	20.96	-0.17
13	20.96	-0.17
TOTAL		-2.2

Change Flow Rate to 200

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	58.3	Speed (km/h)	58.6
Speed (m/s)	16.2	Speed (m/s)	16.3
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	163
Flow (pc/hour)	200	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	1

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	58.5	Speed (km/h)	58.6
Speed (m/s)	16.2	Speed (m/s)	16.3
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120	Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	176	Passengers Per Auto	1.6
Length of Network Link	304	Flow (pc/hr)	163
Departure Red Signal Start	0	Length of Network Link	304
Departure Green Signal Start	75	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	75
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	1755

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	18.78	0.00	18.67		-0.11
2	18.00	18.78	22.15	18.67		-0.11
3	36.00	18.78	44.31	18.67		-0.11
4	54.00	18.78	66.46	63.67		44.89
5	72.00	63.78	88.62	63.67		-0.11
TOTAL						45

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	18.71	-0.07	1	18.67	-0.11
2	18.71	-0.07	2	18.67	-0.11
3	18.71	-0.07	3	18.67	-0.11
4	63.71	44.93	4	63.67	44.89
5	63.71	-0.07	5	63.67	-0.11
TOTAL		44.7	TOTAL		44.6

Measure	Qty	Unit
Baseline Single Cycle Travel Time	75.1	seconds
Future Unimpeded Travel Time	74.7	seconds
Future Impeded Travel Time	119.7	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	2253	seconds
Future Total Hourly Vehicle Travel Time	2376	seconds
Difference	122	seconds

Note: At low flow rates, the VTI and HCM predict the delay well. As expected, lower flow rates equate to lower delay.

Change Flow Rate to 400

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	56.5	Speed (km/h)	56.8
Speed (m/s)	15.7	Speed (m/s)	15.8
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	363
Flow (pc/hour)	400	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	3

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Variable	Value
Speed (km/h)	56.8
Speed (m/s)	15.8
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Transit Dwell Time	30
Passengers Per Auto	1.6
Flow (pc/hr)	363
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120
Number of Lanes	1
Bus Blockage Adjustment Factor	0.975
Maximum Flow Rate	1755

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Variable	Value
Speed (km/h)	56.7
Speed (m/s)	15.7
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6
Flow (pc/hr)	376
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120

Baseline

Future

Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	19.38	0.00	19.26	-0.12
2	9.00	19.38	9.93	19.26	-0.12
3	18.00	19.38	19.86	19.26	-0.12
4	27.00	19.38	29.79	19.26	-0.12
5	36.00	19.38	39.72	19.26	-0.12
6	45.00	19.38	49.66	94.26	74.88
7	54.00	19.38	59.59	94.74	75.35
TOTAL					150

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	19.31	-0.08	1	19.26	-0.12
2	19.31	-0.08	2	19.26	-0.12
3	19.31	-0.08	3	19.26	-0.12
4	19.31	-0.08	4	19.26	-0.12
5	19.31	-0.08	5	19.26	-0.12
6	19.31	-0.08	6	19.26	-0.12
7	64.31	44.92	7	64.26	44.88
TOTAL		44.5	TOTAL		44.2

Measure	Qty	Unit
Baseline Single Cycle Travel Time	135.7	seconds
Future Unimpeded Travel Time	134.9	seconds
Future Impeded Travel Time	285.3	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	4071	seconds
Future Total Hourly Vehicle Travel Time	4497	seconds
Difference	426	seconds

Note: As flow rate increases, delay increases, and VTI and HCM are less accurate at predicting delay times.

Change Flow Rate to 600

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	54.5	Speed (km/h)	54.9
Speed (m/s)	15.1	Speed (m/s)	15.2
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	563
Flow (pc/hour)	600	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	5

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Variable	Value	Variable	Value
Speed (km/h)	54.7	Speed (km/h)	54.9
Speed (m/s)	15.2	Speed (m/s)	15.2
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120	Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	576	Passengers Per Auto	1.6
Length of Network Link	304	Flow (pc/hr)	563
Departure Red Signal Start	0	Length of Network Link	304
Departure Green Signal Start	75	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	75
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	1755

Baseline			Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	
1	0.00	20.08	0.00	19.94	-0.14
2	6.00	20.08	6.40	19.94	-0.14
3	12.00	20.08	12.80	19.94	-0.14
4	18.00	20.08	19.20	19.94	-0.14
5	24.00	20.08	25.60	19.94	-0.14
6	30.00	20.08	32.00	94.94	74.86
7	36.00	20.08	38.40	95.40	75.31
8	42.00	20.08	44.80	95.85	75.77
9	48.00	20.08	51.20	96.30	76.22
10	54.00	20.08	57.60	96.76	76.67
				TOTAL	378

Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	19.99	-0.09	1	19.94	-0.14
2	19.99	-0.09	2	19.94	-0.14
3	19.99	-0.09	3	19.94	-0.14
4	19.99	-0.09	4	19.94	-0.14
5	19.99	-0.09	5	19.94	-0.14
6	19.99	-0.09	6	19.94	-0.14
7	19.99	-0.09	7	19.94	-0.14
8	19.99	-0.09	8	19.94	-0.14
9	19.99	-0.09	9	19.94	-0.14
10	64.99	44.91	10	64.94	44.86
TOTAL		44.1	TOTAL		43.6

Measure	Qty	Unit
Baseline Single Cycle Travel Time	200.8	seconds
Future Unimpeded Travel Time	199.4	seconds
Future Impeded Travel Time	579.0	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	6025	seconds
Future Total Hourly Vehicle Travel Time	7122	seconds
Difference	1097	seconds

Note: As flow rate increases, delay increases, and VTI and HCM are less accurate at predicting delay times as they do not change significantly with flow rate.

Change Flow Rate to 800

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	52.4	Speed (km/h)	52.8
Speed (m/s)	14.5	Speed (m/s)	14.7
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	763
Flow (pc/hour)	800	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	6

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Variable	Value
Speed (km/h)	52.8
Speed (m/s)	14.7
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Transit Dwell Time	30
Passengers Per Auto	1.6
Flow (pc/hr)	763
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120
Number of Lanes	1
Bus Blockage Adjustment Factor	0.975
Maximum Flow Rate	1755

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Variable	Value
Speed (km/h)	52.6
Speed (m/s)	14.6
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6
Flow (pc/hr)	776
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	20.90	0.00	20.74		-0.16
2	4.50	20.90	4.72	20.74		-0.16
3	9.00	20.90	9.44	20.74		-0.16
4	13.50	20.90	14.16	20.74		-0.16
5	18.00	20.90	18.89	20.74		-0.16
6	22.50	20.90	23.61	50.74		29.84
7	27.00	20.90	28.33	96.17		75.27
8	31.50	20.90	33.05	96.60		75.70
9	36.00	20.90	37.77	97.03		76.13
10	40.50	20.90	42.49	97.46		76.56
11	45.00	20.90	47.21	97.89		76.99
12	49.50	20.90	51.93	98.33		77.42
13	54.00	20.90	56.66	95.74		74.84
TOTAL						562

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	20.80	-0.11	1	20.74	-0.16
2	20.80	-0.11	2	20.74	-0.16
3	20.80	-0.11	3	20.74	-0.16
4	20.80	-0.11	4	20.74	-0.16
5	20.80	-0.11	5	20.74	-0.16
6	20.80	-0.11	6	20.74	-0.16
7	20.80	-0.11	7	20.74	-0.16
8	20.80	-0.11	8	20.74	-0.16
9	20.80	-0.11	9	20.74	-0.16
10	20.80	-0.11	10	20.74	-0.16
11	20.80	-0.11	11	20.74	-0.16
12	20.80	-0.11	12	20.74	-0.16
13	65.80	44.89	13	65.74	44.84
TOTAL		43.6	TOTAL		42.9

Measure	Qty	Unit
Baseline Single Cycle Travel Time	271.7	seconds
Future Unimpeded Travel Time	269.6	seconds
Future Impeded Travel Time	833.6	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8151	seconds
Future Total Hourly Vehicle Travel Time	9779	seconds
Difference	1628	seconds

Note: As flow rate increases, delay increases, and VTI and HCM are less accurate at predicting delay times as they do not change significantly with flow rate.

Change Flow Rate to 1000

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	50.0	Speed (km/h)	50.5
Speed (m/s)	13.9	Speed (m/s)	14.0
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	963
Flow (pc/hour)	1000	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	8

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Variable	Value	Variable	Value
Speed (km/h)	50.3	Speed (km/h)	50.5
Speed (m/s)	14.0	Speed (m/s)	14.0
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120	Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	976	Passengers Per Auto	1.6
Length of Network Link	304	Flow (pc/hr)	963
Departure Red Signal Start	0	Length of Network Link	304
Departure Green Signal Start	75	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	75
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	1755

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.89	0.00	21.69		-0.20
2	3.60	21.89	3.74	21.69		-0.20
3	7.20	21.89	7.48	21.69		-0.20
4	10.80	21.89	11.22	21.69		-0.20
5	14.40	21.89	14.96	21.69		-0.20
6	18.00	21.89	18.70	51.69		29.80
7	21.60	21.89	22.44	52.09		30.20
8	25.20	21.89	26.18	97.50		75.61
9	28.80	21.89	29.92	97.90		76.02
10	32.40	21.89	33.66	98.31		76.42
11	36.00	21.89	37.40	98.71		76.83
12	39.60	21.89	41.14	99.12		77.23
13	43.20	21.89	44.88	99.53		77.64
TOTAL						672

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	21.76	-0.13	1	21.69	-0.20
2	21.76	-0.13	2	21.69	-0.20
3	21.76	-0.13	3	21.69	-0.20
4	21.76	-0.13	4	21.69	-0.20
5	21.76	-0.13	5	21.69	-0.20
6	21.76	-0.13	6	21.69	-0.20
7	21.76	-0.13	7	21.69	-0.20
8	21.76	-0.13	8	21.69	-0.20
9	21.76	-0.13	9	21.69	-0.20
10	21.76	-0.13	10	21.69	-0.20
11	21.76	-0.13	11	21.69	-0.20
12	21.76	-0.13	12	21.69	-0.20
13	21.76	-0.13	13	21.69	-0.20
TOTAL		-1.9	TOTAL		-3.0

Measure	Qty	Unit
Baseline Single Cycle Travel Time	328.3	seconds
Future Unimpeded Travel Time	325.3	seconds
Future Impeded Travel Time	999.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	9850	seconds
Future Total Hourly Vehicle Travel Time	11783	seconds
Difference	1933	seconds

Note: As flow rate increases, delay increases, and VTI and HCM are less accurate at predicting delay times as they do not change significantly with flow rate.

Change Flow Rate to 1200

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	47.3	Speed (km/h)	47.9
Speed (m/s)	13.1	Speed (m/s)	13.3
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1163
Flow (pc/hour)	1200	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	10

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Variable	Value
Speed (km/h)	47.9
Speed (m/s)	13.3
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Transit Dwell Time	30
Passengers Per Auto	1.6
Flow (pc/hr)	1163
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120
Number of Lanes	1
Bus Blockage Adjustment Factor	0.975
Maximum Flow Rate	1755

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Variable	Value
Speed (km/h)	47.7
Speed (m/s)	13.2
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6
Flow (pc/hr)	1176
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120

Measure	Qty	Unit
Baseline Single Cycle Travel Time	416.3	seconds
Future Unimpeded Travel Time	411.7	seconds
Future Impeded Travel Time	1314.9	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	12489	seconds
Future Total Hourly Vehicle Travel Time	15059	seconds
Difference	2571	seconds

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	23.13	0.00	22.87		-0.26
2	3.00	23.13	3.10	22.87		-0.26
3	6.00	23.13	6.19	22.87		-0.26
4	9.00	23.13	9.29	22.87		-0.26
5	12.00	23.13	12.39	22.87		-0.26
6	15.00	23.13	15.48	52.87		29.74
7	18.00	23.13	18.58	53.24		30.12
8	21.00	23.13	21.68	98.62		75.49
9	24.00	23.13	24.77	98.99		75.86
10	27.00	23.13	27.87	99.36		76.23
11	30.00	23.13	30.97	99.74		76.61
12	33.00	23.13	34.06	100.11		76.98
13	36.00	23.13	37.16	100.48		77.35
14	39.00	23.13	40.26	100.85		77.73
15	42.00	23.13	43.35	101.23		78.10
16	45.00	23.13	46.45	99.34		76.21
17	48.00	23.13	49.55	97.87		74.74
18	51.00	23.13	52.65	97.87		74.74
TOTAL						899

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Vehicle Number	Travel Time	Difference
1	22.96	-0.17
2	22.96	-0.17
3	22.96	-0.17
4	22.96	-0.17
5	22.96	-0.17
6	22.96	-0.17
7	22.96	-0.17
8	22.96	-0.17
9	22.96	-0.17
10	22.96	-0.17
11	22.96	-0.17
12	22.96	-0.17
13	22.96	-0.17
14	22.96	-0.17
15	22.96	-0.17
16	22.96	-0.17
17	22.96	-0.17
18	67.96	44.83
TOTAL		42.0

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Vehicle Number	Travel Time	Difference
1	22.87	-0.26
2	22.87	-0.26
3	22.87	-0.26
4	22.87	-0.26
5	22.87	-0.26
6	22.87	-0.26
7	22.87	-0.26
8	22.87	-0.26
9	22.87	-0.26
10	22.87	-0.26
11	22.87	-0.26
12	22.87	-0.26
13	22.87	-0.26
14	22.87	-0.26
15	22.87	-0.26
16	22.87	-0.26
17	22.87	-0.26
18	67.87	44.74
TOTAL		40.4

Note: As flow rate increases, delay increases, and VTI and HCM are less accurate at predicting delay times as they do not change significantly with flow rate.

Change Flow Rate to 1400

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	44.1	Speed (km/h)	44.8
Speed (m/s)	12.3	Speed (m/s)	12.4
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1363
Flow (pc/hour)	1400	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	11

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Variable	Value
Speed (km/h)	44.8
Speed (m/s)	12.4
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Transit Dwell Time	30
Passengers Per Auto	1.6
Flow (pc/hr)	1363
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120
Number of Lanes	1
Bus Blockage Adjustment Factor	0.975
Maximum Flow Rate	1755

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Variable	Value
Speed (km/h)	44.6
Speed (m/s)	12.4
Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6
Flow (pc/hr)	1376
Length of Network Link	304
Departure Red Signal Start	0
Departure Green Signal Start	75
Departure Signal Length	120

Measure	Qty	Unit
Baseline Single Cycle Travel Time	495.9	seconds
Future Unimpeded Travel Time	488.7	seconds
Future Impeded Travel Time	1500.5	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	14876	seconds
Future Total Hourly Vehicle Travel Time	17696	seconds
Difference	2820	seconds

Baseline			Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	
1	0.00	24.79	0.00	24.43	-0.36
2	2.57	24.79	2.64	24.43	-0.36
3	5.14	24.79	5.28	24.43	-0.36
4	7.71	24.79	7.93	24.43	-0.36
5	10.29	24.79	10.57	24.43	-0.36
6	12.86	24.79	13.21	54.43	29.64
7	15.43	24.79	15.85	54.76	29.97
8	18.00	24.79	18.50	55.09	30.30
9	20.57	24.79	21.14	100.42	75.63
10	23.14	24.79	23.78	100.75	75.96
11	25.71	24.79	26.42	101.08	76.29
12	28.29	24.79	29.06	101.42	76.62
13	30.86	24.79	31.71	101.75	76.95
14	33.43	24.79	34.35	102.08	77.28
15	36.00	24.79	36.99	102.41	77.61
16	38.57	24.79	39.63	102.74	77.94
17	41.14	24.79	42.28	103.07	78.27
18	43.71	24.79	44.92	99.43	74.64
19	46.29	24.79	47.56	99.43	74.64
20	48.86	24.79	50.20	99.43	74.64
			TOTAL		1005

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	24.56	-0.23	1	24.43	-0.36
2	24.56	-0.23	2	24.43	-0.36
3	24.56	-0.23	3	24.43	-0.36
4	24.56	-0.23	4	24.43	-0.36
5	24.56	-0.23	5	24.43	-0.36
6	24.56	-0.23	6	24.43	-0.36
7	24.56	-0.23	7	24.43	-0.36
8	24.56	-0.23	8	24.43	-0.36
9	24.56	-0.23	9	24.43	-0.36
10	24.56	-0.23	10	24.43	-0.36
11	24.56	-0.23	11	24.43	-0.36
12	24.56	-0.23	12	24.43	-0.36
13	24.56	-0.23	13	24.43	-0.36
14	24.56	-0.23	14	24.43	-0.36
15	24.56	-0.23	15	24.43	-0.36
16	24.56	-0.23	16	24.43	-0.36
17	24.56	-0.23	17	24.43	-0.36
18	24.56	-0.23	18	24.43	-0.36
19	24.56	-0.23	19	24.43	-0.36
20	24.56	-0.23	20	24.43	-0.36
TOTAL		-4.7	TOTAL		-7.2

Change Flow Rate to 1600

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	40.0	Speed (km/h)	40.9
Speed (m/s)	11.1	Speed (m/s)	11.4
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1563
Flow (pc/hour)	1600	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	13

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	40.6	Speed (km/h)	40.9
Speed (m/s)	11.3	Speed (m/s)	11.4
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/ln/km)	120	Jam Density (pc/ln/km)	120
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1576	Passengers Per Auto	1.6
Length of Network Link	304	Flow (pc/hr)	1563
Departure Red Signal Start	0	Length of Network Link	304
Departure Green Signal Start	75	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	75
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	1755

Measure	Qty	Unit
Baseline Single Cycle Travel Time	601.9	seconds
Future Unimpeded Travel Time	588.7	seconds
Future Impeded Travel Time	1753.0	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	18058	seconds
Future Total Hourly Vehicle Travel Time	21154	seconds
Difference	3097	seconds

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	27.36	0.00	26.76		-0.60
2	2.25	27.36	2.30	26.76		-0.60
3	4.50	27.36	4.61	26.76		-0.60
4	6.75	27.36	6.91	26.76		-0.60
5	9.00	27.36	9.22	26.76		-0.60
6	11.25	27.36	11.52	56.76		29.40
7	13.50	27.36	13.82	57.03		29.67
8	15.75	27.36	16.13	57.29		29.93
9	18.00	27.36	18.43	102.56		75.20
10	20.25	27.36	20.74	102.83		75.47
11	22.50	27.36	23.04	103.09		75.73
12	24.75	27.36	25.34	103.36		76.00
13	27.00	27.36	27.65	103.62		76.26
14	29.25	27.36	29.95	103.89		76.53
15	31.50	27.36	32.26	104.16		76.80
16	33.75	27.36	34.56	104.42		77.06
17	36.00	27.36	36.86	104.69		77.33
18	38.25	27.36	39.17	104.96		77.60
19	40.50	27.36	41.47	105.22		77.86
20	42.75	27.36	43.78	101.76		74.40
21	45.00	27.36	46.08	101.76		74.40
22	47.25	27.36	48.38	101.76		74.40
TOTAL						1151

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	26.97	-0.39	1	26.76	-0.60
2	26.97	-0.39	2	26.76	-0.60
3	26.97	-0.39	3	26.76	-0.60
4	26.97	-0.39	4	26.76	-0.60
5	26.97	-0.39	5	26.76	-0.60
6	26.97	-0.39	6	26.76	-0.60
7	26.97	-0.39	7	26.76	-0.60
8	26.97	-0.39	8	26.76	-0.60
9	26.97	-0.39	9	26.76	-0.60
10	26.97	-0.39	10	26.76	-0.60
11	26.97	-0.39	11	26.76	-0.60
12	26.97	-0.39	12	26.76	-0.60
13	26.97	-0.39	13	26.76	-0.60
14	26.97	-0.39	14	26.76	-0.60
15	26.97	-0.39	15	26.76	-0.60
16	26.97	-0.39	16	26.76	-0.60
17	26.97	-0.39	17	26.76	-0.60
18	26.97	-0.39	18	26.76	-0.60
19	26.97	-0.39	19	26.76	-0.60
20	26.97	-0.39	20	26.76	-0.60
21	26.97	-0.39	21	26.76	-0.60
22	71.97	44.61	22	71.76	44.40
TOTAL		36.4	TOTAL		31.8

Case 2: Modify Flow Rate

Change Flow Rate to 1800

Note: At flow rates below maximum flow for a single lane, there is no queue and therefore no delay.

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	61.5	Speed (km/h)	61.7
Speed (m/s)	17.1	Speed (m/s)	17.1
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1763
Flow (pc/hour)	1800	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	15
		Number of Lanes	2
		Transit Vehicle Stop Time	31.82
		Transit Vehicle Release Time	61.82

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	61.6	Speed (km/h)	61.7
Speed (m/s)	17.1	Speed (m/s)	17.1
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1776	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1763
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	7020

Measure	Qty	Unit
Baseline Single Cycle Travel Time	796.7	seconds
Future Unimpeded Travel Time	793.3	seconds
Future Impeded Travel Time	836.5	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	23900	seconds
Future Total Hourly Vehicle Travel Time	23928	seconds
Difference	28	seconds

Baseline			Future			
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference	
1	0.00	46.86	0.00	46.66	-0.20	
2	2.00	46.86	2.04	46.66	-0.20	
3	4.00	46.86	4.09	46.66	-0.20	
4	6.00	46.86	6.13	47.06	0.20	
5	8.00	46.86	8.17	47.12	0.25	
6	10.00	46.86	10.21	47.20	0.33	
7	12.00	46.86	12.26	47.31	0.44	
8	14.00	46.86	14.30	47.43	0.57	
9	16.00	46.86	16.34	47.59	0.72	
10	18.00	46.86	18.38	47.66	0.80	
11	20.00	46.86	20.43	47.84	0.98	
12	22.00	46.86	22.47	47.82	0.95	
13	24.00	46.86	24.51	48.00	1.14	
14	26.00	46.86	26.55	47.98	1.12	
15	28.00	46.86	28.60	48.16	1.29	
16	30.00	46.86	30.64	48.26	1.39	
17	32.00	46.86	32.68	88.30	41.43	
			TOTAL			51

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	46.73	-0.13	1	46.66	-0.20
2	46.73	-0.13	2	46.66	-0.20
3	46.73	-0.13	3	46.66	-0.20
4	46.73	-0.13	4	46.66	-0.20
5	46.73	-0.13	5	46.66	-0.20
6	46.73	-0.13	6	46.66	-0.20
7	46.73	-0.13	7	46.66	-0.20
8	46.73	-0.13	8	46.66	-0.20
9	46.73	-0.13	9	46.66	-0.20
10	46.73	-0.13	10	46.66	-0.20
11	46.73	-0.13	11	46.66	-0.20
12	46.73	-0.13	12	46.66	-0.20
13	46.73	-0.13	13	46.66	-0.20
14	46.73	-0.13	14	46.66	-0.20
15	46.73	-0.13	15	46.66	-0.20
16	46.73	-0.13	16	46.66	-0.20
17	46.73	-0.13	17	46.66	-0.20
TOTAL		-2.2	TOTAL		-3.4

Change Flow Rate to 2000

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	31.81
		Transit Vehicle Release Time	61.81

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	60.2	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1976	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1963
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	7020

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	915.7	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25967	seconds
Difference	47	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	48.16	0.16
5	7.20	48.00	7.34	48.22	0.22
6	9.00	48.00	9.17	48.29	0.29
7	10.80	48.00	11.01	48.40	0.40
8	12.60	48.00	12.84	48.52	0.52
9	14.40	48.00	14.68	48.67	0.67
10	16.20	48.00	16.51	48.74	0.74
11	18.00	48.00	18.34	48.92	0.92
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		52

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	47.86	-0.14	1	47.78	-0.22
2	47.86	-0.14	2	47.78	-0.22
3	47.86	-0.14	3	47.78	-0.22
4	47.86	-0.14	4	47.78	-0.22
5	47.86	-0.14	5	47.78	-0.22
6	47.86	-0.14	6	47.78	-0.22
7	47.86	-0.14	7	47.78	-0.22
8	47.86	-0.14	8	47.78	-0.22
9	47.86	-0.14	9	47.78	-0.22
10	47.86	-0.14	10	47.78	-0.22
11	47.86	-0.14	11	47.78	-0.22
12	47.86	-0.14	12	47.78	-0.22
13	47.86	-0.14	13	47.78	-0.22
14	47.86	-0.14	14	47.78	-0.22
15	47.86	-0.14	15	47.78	-0.22
16	47.86	-0.14	16	47.78	-0.22
17	47.86	-0.14	17	47.78	-0.22
18	47.86	-0.14	18	47.78	-0.22
TOTAL		-2.6	TOTAL		-4.0

Note: As expected, delay increases with increased flow rate, though there is less delay than in Case 1 since vehicles are allowed to pass.

Change Flow Rate to 2200

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	58.4	Speed (km/h)	58.7
Speed (m/s)	16.2	Speed (m/s)	16.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	2163
Flow (pc/hour)	2200	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	18
		Number of Lanes	2
		Transit Vehicle Stop Time	31.99
		Transit Vehicle Release Time	61.99

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	58.6	Speed (km/h)	58.7
Speed (m/s)	16.3	Speed (m/s)	16.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	2176	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	2163
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	7020

Measure	Qty	Unit
Baseline Single Cycle Travel Time	936.2	seconds
Future Unimpeded Travel Time	931.4	seconds
Future Impeded Travel Time	988.3	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	28086	seconds
Future Total Hourly Vehicle Travel Time	28113	seconds
Difference	28	seconds

Baseline			Future		Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	
1	0.00	49.27	0.00	49.02	-0.25
2	1.64	49.27	1.66	49.02	-0.25
3	3.27	49.27	3.33	49.02	-0.25
4	4.91	49.27	4.99	49.39	0.12
5	6.55	49.27	6.66	49.44	0.17
6	8.18	49.27	8.32	49.52	0.25
7	9.82	49.27	9.99	49.62	0.35
8	11.45	49.27	11.65	49.74	0.46
9	13.09	49.27	13.32	49.88	0.61
10	14.73	49.27	14.98	49.95	0.67
11	16.36	49.27	16.65	50.12	0.84
12	18.00	49.27	18.31	50.09	0.82
13	19.64	49.27	19.98	50.26	0.99
14	21.27	49.27	21.64	50.25	0.97
15	22.91	49.27	23.31	50.41	1.14
16	24.55	49.27	24.97	50.50	1.23
17	26.18	49.27	26.64	50.54	1.27
18	27.82	49.27	28.30	50.70	1.43
19	29.45	49.27	29.97	90.84	41.57
				TOTAL	52

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	49.11	-0.16	1	49.02	-0.25
2	49.11	-0.16	2	49.02	-0.25
3	49.11	-0.16	3	49.02	-0.25
4	49.11	-0.16	4	49.02	-0.25
5	49.11	-0.16	5	49.02	-0.25
6	49.11	-0.16	6	49.02	-0.25
7	49.11	-0.16	7	49.02	-0.25
8	49.11	-0.16	8	49.02	-0.25
9	49.11	-0.16	9	49.02	-0.25
10	49.11	-0.16	10	49.02	-0.25
11	49.11	-0.16	11	49.02	-0.25
12	49.11	-0.16	12	49.02	-0.25
13	49.11	-0.16	13	49.02	-0.25
14	49.11	-0.16	14	49.02	-0.25
15	49.11	-0.16	15	49.02	-0.25
16	49.11	-0.16	16	49.02	-0.25
17	49.11	-0.16	17	49.02	-0.25
18	49.11	-0.16	18	49.02	-0.25
19	49.11	-0.16	19	49.02	-0.25
TOTAL		-3.1	TOTAL		-4.8

Note: No significant delay increase over the case at 1800.

Change Flow Rate to 2400

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	56.8	Speed (km/h)	57.1
Speed (m/s)	15.8	Speed (m/s)	15.9
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	2363
Flow (pc/hour)	2400	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	20
		Number of Lanes	2
		Transit Vehicle Stop Time	32.34
		Transit Vehicle Release Time	62.34

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	57.0	Speed (km/h)	57.1
Speed (m/s)	15.8	Speed (m/s)	15.9
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	2376	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	2363
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	3510

Measure	Qty	Unit
Baseline Single Cycle Travel Time	1014.4	seconds
Future Unimpeded Travel Time	1008.6	seconds
Future Impeded Travel Time	1066.5	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	30431	seconds
Future Total Hourly Vehicle Travel Time	30433	seconds
Difference	2	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	50.72	0.00	50.43	-0.29
2	1.50	50.72	1.52	50.43	-0.29
3	3.00	50.72	3.05	50.43	-0.29
4	4.50	50.72	4.57	50.78	0.07
5	6.00	50.72	6.10	50.83	0.12
6	7.50	50.72	7.62	50.90	0.19
7	9.00	50.72	9.14	51.00	0.29
8	10.50	50.72	10.67	51.11	0.39
9	12.00	50.72	12.19	51.25	0.53
10	13.50	50.72	13.71	51.32	0.60
11	15.00	50.72	15.24	51.48	0.76
12	16.50	50.72	16.76	51.45	0.74
13	18.00	50.72	18.29	51.62	0.90
14	19.50	50.72	19.81	51.60	0.88
15	21.00	50.72	21.33	51.76	1.04
16	22.50	50.72	22.86	51.85	1.13
17	24.00	50.72	24.38	51.88	1.16
18	25.50	50.72	25.90	52.03	1.32
19	27.00	50.72	27.43	52.17	1.45
20	28.50	50.72	28.95	92.20	41.49
			TOTAL		52

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	50.53	-0.18	1	50.43	-0.29
2	50.53	-0.18	2	50.43	-0.29
3	50.53	-0.18	3	50.43	-0.29
4	50.53	-0.18	4	50.43	-0.29
5	50.53	-0.18	5	50.43	-0.29
6	50.53	-0.18	6	50.43	-0.29
7	50.53	-0.18	7	50.43	-0.29
8	50.53	-0.18	8	50.43	-0.29
9	50.53	-0.18	9	50.43	-0.29
10	50.53	-0.18	10	50.43	-0.29
11	50.53	-0.18	11	50.43	-0.29
12	50.53	-0.18	12	50.43	-0.29
13	50.53	-0.18	13	50.43	-0.29
14	50.53	-0.18	14	50.43	-0.29
15	50.53	-0.18	15	50.43	-0.29
16	50.53	-0.18	16	50.43	-0.29
17	50.53	-0.18	17	50.43	-0.29
18	50.53	-0.18	18	50.43	-0.29
19	50.53	-0.18	19	50.43	-0.29
20	50.53	-0.18	20	50.43	-0.29
TOTAL		-3.7	TOTAL		-5.7

Note: As Instantaneous Flow rate approaches max flow rate of the road, imposed delay appears to decrease.

Change Flow Rate to 2600

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	55.0	Speed (km/h)	55.3
Speed (m/s)	15.3	Speed (m/s)	15.4
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	2563
Flow (pc/hour)	2600	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	21
		Number of Lanes	2
		Transit Vehicle Stop Time	32.88
		Transit Vehicle Release Time	62.88

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	55.2	Speed (km/h)	55.3
Speed (m/s)	15.3	Speed (m/s)	15.4
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	2576	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	2563
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	3510

Measure	Qty	Unit
Baseline Single Cycle Travel Time	1047.8	seconds
Future Unimpeded Travel Time	1041.1	seconds
Future Impeded Travel Time	1098.0	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	31433	seconds
Future Total Hourly Vehicle Travel Time	31404	seconds
Difference	-29	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	52.39	0.00	52.05	-0.33
2	1.38	52.39	1.40	52.05	-0.33
3	2.77	52.39	2.81	52.05	-0.33
4	4.15	52.39	4.21	52.39	0.00
5	5.54	52.39	5.62	52.43	0.05
6	6.92	52.39	7.02	52.50	0.11
7	8.31	52.39	8.43	52.60	0.21
8	9.69	52.39	9.83	52.70	0.31
9	11.08	52.39	11.24	52.83	0.44
10	12.46	52.39	12.64	52.89	0.50
11	13.85	52.39	14.05	53.04	0.65
12	15.23	52.39	15.45	53.02	0.63
13	16.62	52.39	16.86	53.17	0.79
14	18.00	52.39	18.26	53.16	0.77
15	19.38	52.39	19.67	53.31	0.92
16	20.77	52.39	21.07	53.39	1.00
17	22.15	52.39	22.48	53.43	1.04
18	23.54	52.39	23.88	53.57	1.18
19	24.92	52.39	25.29	53.69	1.31
20	26.31	52.39	26.69	93.73	41.34
				TOTAL	50

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	52.17	-0.21	1	52.05	-0.33
2	52.17	-0.21	2	52.05	-0.33
3	52.17	-0.21	3	52.05	-0.33
4	52.17	-0.21	4	52.05	-0.33
5	52.17	-0.21	5	52.05	-0.33
6	52.17	-0.21	6	52.05	-0.33
7	52.17	-0.21	7	52.05	-0.33
8	52.17	-0.21	8	52.05	-0.33
9	52.17	-0.21	9	52.05	-0.33
10	52.17	-0.21	10	52.05	-0.33
11	52.17	-0.21	11	52.05	-0.33
12	52.17	-0.21	12	52.05	-0.33
13	52.17	-0.21	13	52.05	-0.33
14	52.17	-0.21	14	52.05	-0.33
15	52.17	-0.21	15	52.05	-0.33
16	52.17	-0.21	16	52.05	-0.33
17	52.17	-0.21	17	52.05	-0.33
18	52.17	-0.21	18	52.05	-0.33
19	52.17	-0.21	19	52.05	-0.33
20	52.17	-0.21	20	52.05	-0.33
TOTAL		-4.3	TOTAL		-6.7

Note: As more vehicles experience an increase in travel speeds compared with the number of vehicles delayed, the benefits seem to increase.

Case 1: Increase Transit Frequency

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.2
Speed (m/s)	14.4	Speed (m/s)	14.5
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	813
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	299
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	7

Baseline				
Vehicle Number	Arrival (s)	Travel Time	Travel Time	Difference
1	0.00	21.13	20.96	-0.17
2	4.23	21.13	20.96	-0.17
3	8.47	21.13	20.96	-0.17
4	12.70	21.13	20.96	-0.17
5	16.93	21.13	20.96	-0.17
6	21.16	21.13	50.96	29.83
7	25.40	21.13	96.39	75.25
8	29.63	21.13	96.81	75.68
9	33.86	21.13	97.24	76.10
10	38.09	21.13	97.66	76.53
11	42.33	21.13	98.09	76.95
12	46.56	21.13	98.51	77.38
13	50.79	21.13	97.64	76.51
			TOTAL	563

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	21.02	-0.11	1	20.96	-0.17
2	21.02	-0.11	2	20.96	-0.17
3	21.02	-0.11	3	20.96	-0.17
4	21.02	-0.11	4	20.96	-0.17
5	21.02	-0.11	5	20.96	-0.17
6	21.02	-0.11	6	20.96	-0.17
7	21.02	-0.11	7	20.96	-0.17
8	21.02	-0.11	8	20.96	-0.17
9	21.02	-0.11	9	20.96	-0.17
10	21.02	-0.11	10	20.96	-0.17
11	21.02	-0.11	11	20.96	-0.17
12	21.02	-0.11	12	20.96	-0.17
13	21.02	-0.11	13	20.96	-0.17
TOTAL		-1.4	TOTAL		-2.2

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	272.5	seconds
Future Impeded Travel Time	838.1	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	9871	seconds
Difference	1630	seconds

Increase Transit Vehicles to 6 Per Hour

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	52.6
Speed (m/s)	14.4	Speed (m/s)	14.6
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	6
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	776
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	6

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.80		-0.34
2	4.23	21.13	4.64	20.80		-0.34
3	8.46	21.13	9.28	20.80		-0.34
4	12.69	21.13	13.92	20.80		-0.34
5	16.92	21.13	18.56	20.80		-0.34
6	21.15	21.13	23.20	50.80		29.66
7	25.38	21.13	27.84	96.23		75.09
8	29.61	21.13	32.47	96.66		75.52
9	33.84	21.13	37.11	97.09		75.95
10	38.07	21.13	41.75	97.52		76.38
11	42.30	21.13	46.39	97.95		76.81
12	46.53	21.13	51.03	98.37		77.24
13	50.76	21.13	55.67	95.84		74.70
TOTAL						560

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	270.3	seconds
Future Impeded Travel Time	834.4	seconds
Impeded Cycles / hour	6.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	11495	seconds
Difference	3252	seconds

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	20.91	-0.22	1	20.80	-0.34
2	20.91	-0.22	2	20.80	-0.34
3	20.91	-0.22	3	20.80	-0.34
4	20.91	-0.22	4	20.80	-0.34
5	20.91	-0.22	5	20.80	-0.34
6	20.91	-0.22	6	20.80	-0.34
7	20.91	-0.22	7	20.80	-0.34
8	20.91	-0.22	8	20.80	-0.34
9	20.91	-0.22	9	20.80	-0.34
10	20.91	-0.22	10	20.80	-0.34
11	20.91	-0.22	11	20.80	-0.34
12	20.91	-0.22	12	20.80	-0.34
13	65.91	44.78	13	65.80	44.66
TOTAL		42.1	TOTAL		40.6

Increase Transit Frequency to 12

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	53.4
Speed (m/s)	14.4	Speed (m/s)	14.8
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	12
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	701
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	6

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	20.48		-0.66
2	4.23	21.13	5.14	20.48		-0.66
3	8.46	21.13	10.27	20.48		-0.66
4	12.69	21.13	15.41	20.48		-0.66
5	16.92	21.13	20.54	20.48		-0.66
6	21.15	21.13	25.68	95.48		74.34
7	25.38	21.13	30.81	95.92		74.78
8	29.61	21.13	35.95	96.36		75.22
9	33.84	21.13	41.08	96.79		75.66
10	38.07	21.13	46.22	97.23		76.10
11	42.30	21.13	51.36	97.67		76.54
12	46.53	21.13	56.49	97.30		76.16
13	50.76	21.13	61.63	95.48		74.34
				TOTAL		600

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	266.2	seconds
Future Impeded Travel Time	874.6	seconds
Impeded Cycles / hour	12.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	15287	seconds
Difference	7045	seconds

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	20.70	-0.43	1	20.48	-0.66
2	20.70	-0.43	2	20.48	-0.66
3	20.70	-0.43	3	20.48	-0.66
4	20.70	-0.43	4	20.48	-0.66
5	20.70	-0.43	5	20.48	-0.66
6	20.70	-0.43	6	20.48	-0.66
7	20.70	-0.43	7	20.48	-0.66
8	20.70	-0.43	8	20.48	-0.66
9	20.70	-0.43	9	20.48	-0.66
10	20.70	-0.43	10	20.48	-0.66
11	20.70	-0.43	11	20.48	-0.66
12	65.70	44.57	12	65.48	44.34
13	65.70	44.57	13	65.48	44.34
TOTAL		84.4	TOTAL		81.5

Increase Transit Frequency to 24

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	51.8	Speed (km/h)	55.0
Speed (m/s)	14.4	Speed (m/s)	15.3
Free Flow Speed (km/h)	60	Free Flow Speed (km/h)	60
Free Flow Speed (m/s)	16.7	Free Flow Speed (m/s)	16.7
Jam Density (pc/km/ln)	120	Jam Density (pc/ln/km)	120
AADT	16000	Transit Vehicles Per Hour	24
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	551
Flow (pc/hour)	851	Length of Network Link	304
Length of Network Link (m)	304	Distance to Stop	150
Maximum Flow Rate	1800	Transit Vehicle Position	6
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	75	Departure Green Signal Start	75
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	1800
		Maximum Queue Length	5

Baseline			Future			Difference
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time		
1	0.00	21.13	0.00	19.90		-1.23
2	4.23	21.13	6.53	19.90		-1.23
3	8.46	21.13	13.07	19.90		-1.23
4	12.69	21.13	19.60	19.90		-1.23
5	16.92	21.13	26.13	19.90		-1.23
6	21.15	21.13	32.67	94.90		73.77
7	25.38	21.13	39.20	95.36		74.22
8	29.61	21.13	45.74	95.81		74.68
9	33.84	21.13	52.27	96.27		75.13
10	38.07	21.13	58.80	96.72		75.59
11	42.30	21.13	65.34	95.47		74.34
12	46.53	21.13	71.87	94.90		73.77
13	50.76	21.13	78.40	94.90		73.77
TOTAL						589

Measure	Qty	Unit
Baseline Single Cycle Travel Time	274.7	seconds
Future Unimpeded Travel Time	258.7	seconds
Future Impeded Travel Time	863.8	seconds
Impeded Cycles / hour	24.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	8242	seconds
Future Total Hourly Vehicle Travel Time	22284	seconds
Difference	14042	seconds

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Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	20.31	-0.82	1	19.90	-1.23
2	20.31	-0.82	2	19.90	-1.23
3	20.31	-0.82	3	19.90	-1.23
4	20.31	-0.82	4	19.90	-1.23
5	20.31	-0.82	5	19.90	-1.23
6	20.31	-0.82	6	19.90	-1.23
7	20.31	-0.82	7	19.90	-1.23
8	20.31	-0.82	8	19.90	-1.23
9	20.31	-0.82	9	19.90	-1.23
10	65.31	44.18	10	64.90	43.77
11	65.31	44.18	11	64.90	43.77
12	65.31	44.18	12	64.90	43.77
13	65.31	44.18	13	64.90	43.77
TOTAL		169.3	TOTAL		164.0

Case 2: Increase Transit Frequency

Base Conditions

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	3
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1963
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	31.81
		Transit Vehicle Release Time	61.81

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	60.2	Speed (km/h)	60.3
Speed (m/s)	16.7	Speed (m/s)	16.7
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	3	Transit Vehicles per Hour	3
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1976	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1963
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.975
		Maximum Flow Rate	7020

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	860.0	seconds
Future Impeded Travel Time	915.7	seconds
Impeded Cycles / hour	3.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	25967	seconds
Difference	47	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.78	-0.22
2	1.80	48.00	1.83	47.78	-0.22
3	3.60	48.00	3.67	47.78	-0.22
4	5.40	48.00	5.50	48.16	0.16
5	7.20	48.00	7.34	48.22	0.22
6	9.00	48.00	9.17	48.29	0.29
7	10.80	48.00	11.01	48.40	0.40
8	12.60	48.00	12.84	48.52	0.52
9	14.40	48.00	14.68	48.67	0.67
10	16.20	48.00	16.51	48.74	0.74
11	18.00	48.00	18.34	48.92	0.92
12	19.80	48.00	20.18	48.89	0.89
13	21.60	48.00	22.01	49.07	1.07
14	23.40	48.00	23.85	49.05	1.05
15	25.20	48.00	25.68	49.22	1.22
16	27.00	48.00	27.52	49.32	1.32
17	28.80	48.00	29.35	49.36	1.36
18	30.60	48.00	31.18	89.52	41.52
			TOTAL		52

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	47.86	-0.14	1	47.78	-0.22
2	47.86	-0.14	2	47.78	-0.22
3	47.86	-0.14	3	47.78	-0.22
4	47.86	-0.14	4	47.78	-0.22
5	47.86	-0.14	5	47.78	-0.22
6	47.86	-0.14	6	47.78	-0.22
7	47.86	-0.14	7	47.78	-0.22
8	47.86	-0.14	8	47.78	-0.22
9	47.86	-0.14	9	47.78	-0.22
10	47.86	-0.14	10	47.78	-0.22
11	47.86	-0.14	11	47.78	-0.22
12	47.86	-0.14	12	47.78	-0.22
13	47.86	-0.14	13	47.78	-0.22
14	47.86	-0.14	14	47.78	-0.22
15	47.86	-0.14	15	47.78	-0.22
16	47.86	-0.14	16	47.78	-0.22
17	47.86	-0.14	17	47.78	-0.22
18	47.86	-0.14	18	47.78	-0.22
TOTAL		-2.6	TOTAL		-4.0

Increase Transit Frequency to 6

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	60.6
Speed (m/s)	16.7	Speed (m/s)	16.8
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	6
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1925
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	16
		Number of Lanes	2
		Transit Vehicle Stop Time	31.80
		Transit Vehicle Release Time	61.80

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	60.4	Speed (km/h)	60.6
Speed (m/s)	16.8	Speed (m/s)	16.8
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	6	Transit Vehicles per Hour	6
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1952	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1925
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.95
		Maximum Flow Rate	3420

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	856.1	seconds
Future Impeded Travel Time	911.9	seconds
Impeded Cycles / hour	6.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	26017	seconds
Difference	97	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.56	-0.44
2	1.80	48.00	1.87	47.56	-0.44
3	3.60	48.00	3.74	47.56	-0.44
4	5.40	48.00	5.61	47.95	-0.05
5	7.20	48.00	7.48	48.00	0.00
6	9.00	48.00	9.35	48.08	0.08
7	10.80	48.00	11.22	48.19	0.19
8	12.60	48.00	13.09	48.31	0.31
9	14.40	48.00	14.96	48.46	0.46
10	16.20	48.00	16.83	48.53	0.53
11	18.00	48.00	18.70	48.71	0.71
12	19.80	48.00	20.57	48.68	0.68
13	21.60	48.00	22.44	48.86	0.86
14	23.40	48.00	24.31	48.84	0.84
15	25.20	48.00	26.18	49.01	1.01
16	27.00	48.00	28.05	49.11	1.11
17	28.80	48.00	29.92	49.15	1.15
18	30.60	48.00	31.79	89.32	41.32
				TOTAL	48.32

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	47.72	-0.28	1	47.56	-0.44
2	47.72	-0.28	2	47.56	-0.44
3	47.72	-0.28	3	47.56	-0.44
4	47.72	-0.28	4	47.56	-0.44
5	47.72	-0.28	5	47.56	-0.44
6	47.72	-0.28	6	47.56	-0.44
7	47.72	-0.28	7	47.56	-0.44
8	47.72	-0.28	8	47.56	-0.44
9	47.72	-0.28	9	47.56	-0.44
10	47.72	-0.28	10	47.56	-0.44
11	47.72	-0.28	11	47.56	-0.44
12	47.72	-0.28	12	47.56	-0.44
13	47.72	-0.28	13	47.56	-0.44
14	47.72	-0.28	14	47.56	-0.44
15	47.72	-0.28	15	47.56	-0.44
16	47.72	-0.28	16	47.56	-0.44
17	47.72	-0.28	17	47.56	-0.44
18	47.72	-0.28	18	47.56	-0.44
TOTAL		-5.1	TOTAL		-7.9

Increase Transit Frequency to 12

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	61.1
Speed (m/s)	16.7	Speed (m/s)	17.0
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	12
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1850
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	15
		Number of Lanes	2
		Transit Vehicle Stop Time	31.79
		Transit Vehicle Release Time	61.79

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	60.7	Speed (km/h)	61.1
Speed (m/s)	16.9	Speed (m/s)	17.0
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	12	Transit Vehicles per Hour	12
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1904	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1850
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.9
		Maximum Flow Rate	3240

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	848.4	seconds
Future Impeded Travel Time	904.4	seconds
Impeded Cycles / hour	12.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	26125	seconds
Difference	205	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	47.14	-0.86
2	1.80	48.00	1.95	47.14	-0.86
3	3.60	48.00	3.89	47.14	-0.86
4	5.40	48.00	5.84	47.53	-0.47
5	7.20	48.00	7.78	47.58	-0.42
6	9.00	48.00	9.73	47.66	-0.34
7	10.80	48.00	11.68	47.77	-0.23
8	12.60	48.00	13.62	47.89	-0.11
9	14.40	48.00	15.57	48.05	0.05
10	16.20	48.00	17.51	48.12	0.12
11	18.00	48.00	19.46	48.30	0.30
12	19.80	48.00	21.41	48.27	0.27
13	21.60	48.00	23.35	48.45	0.45
14	23.40	48.00	25.30	48.44	0.44
15	25.20	48.00	27.24	48.61	0.61
16	27.00	48.00	29.19	48.71	0.71
17	28.80	48.00	31.14	48.75	0.75
18	30.60	48.00	33.08	88.92	40.92
				TOTAL	40

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	47.44	-0.56	1	47.14	-0.86
2	47.44	-0.56	2	47.14	-0.86
3	47.44	-0.56	3	47.14	-0.86
4	47.44	-0.56	4	47.14	-0.86
5	47.44	-0.56	5	47.14	-0.86
6	47.44	-0.56	6	47.14	-0.86
7	47.44	-0.56	7	47.14	-0.86
8	47.44	-0.56	8	47.14	-0.86
9	47.44	-0.56	9	47.14	-0.86
10	47.44	-0.56	10	47.14	-0.86
11	47.44	-0.56	11	47.14	-0.86
12	47.44	-0.56	12	47.14	-0.86
13	47.44	-0.56	13	47.14	-0.86
14	47.44	-0.56	14	47.14	-0.86
15	47.44	-0.56	15	47.14	-0.86
16	47.44	-0.56	16	47.14	-0.86
17	47.44	-0.56	17	47.14	-0.86
18	47.44	-0.56	18	87.14	39.14
TOTAL		-10.1	TOTAL		24.4

Increase Transit Frequency to 24

Initial Conditions		Future Conditions	
Variable	Value	Variable	Value
Speed (km/h)	60.0	Speed (km/h)	62.2
Speed (m/s)	16.7	Speed (m/s)	17.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/km/ln)	200	Jam Density (pc/ln/km)	200
AADT	32570	Transit Vehicles Per Hour	24
24-Hour Traffic Factor	1.3	Transit Utilization Per Vehicle	20
12-Hour Traffic Factor	3.6	Dwell Time Per Vehicle	30
Peak Hour Factor	2	Passengers Per Auto	1.6
Number of Directions	2	Flow (pc/hr)	1700
Flow (pc/hour)	2000	Length of Network Link	800
Length of Network Link (m)	800	Distance to Stop	450
Maximum Flow Rate	3600	Transit Vehicle Position	4
Departure Red Signal Start	0	Departure Red Signal Start	0
Departure Green Signal Start	80	Departure Green Signal Start	80
Departure Cycle Length	120	Departure Signal Length	120
		Max Flow Rate	3600
		Maximum Queue Length	14
		Number of Lanes	2
		Transit Vehicle Stop Time	31.87
		Transit Vehicle Release Time	61.87

Victoria Transport Institute		Highway Capacity Manual	
Variable	Value	Variable	Value
Speed (km/h)	61.4	Speed (km/h)	62.2
Speed (m/s)	17.1	Speed (m/s)	17.3
Free Flow Speed (km/h)	72	Free Flow Speed (km/h)	72
Free Flow Speed (m/s)	20.0	Free Flow Speed (m/s)	20.0
Jam Density (pc/ln/km)	200	Jam Density (pc/km)	200
Transit Vehicles per Hour	24	Transit Vehicles per Hour	24
Transit Utilization Per Vehicle	20	Transit Utilization Per Vehicle	20
Passengers Per Auto	1.6	Transit Dwell Time	30
Flow (pc/hr)	1808	Passengers Per Auto	1.6
Length of Network Link	800	Flow (pc/hr)	1700
Departure Red Signal Start	0	Length of Network Link	800
Departure Green Signal Start	80	Departure Red Signal Start	0
Departure Signal Length	120	Departure Green Signal Start	80
		Departure Signal Length	120
		Number of Lanes	1
		Bus Blockage Adjustment Factor	0.8
		Maximum Flow Rate	2880

Measure	Qty	Unit
Baseline Single Cycle Travel Time	864.0	seconds
Future Unimpeded Travel Time	834.1	seconds
Future Impeded Travel Time	930.5	seconds
Impeded Cycles / hour	24.0	
Total Cycles / hour	30.0	
Baseline Total Hourly Vehicle Travel Time	25920	seconds
Future Total Hourly Vehicle Travel Time	27335	seconds
Difference	1415	seconds

Baseline			Future		
Vehicle Number	Arrival (s)	Travel Time	Arrival (s)	Travel Time	Difference
1	0.00	48.00	0.00	46.34	-1.66
2	1.80	48.00	2.12	46.34	-1.66
3	3.60	48.00	4.24	46.34	-1.66
4	5.40	48.00	6.35	46.74	-1.26
5	7.20	48.00	8.47	46.79	-1.21
6	9.00	48.00	10.59	46.88	-1.12
7	10.80	48.00	12.71	46.99	-1.01
8	12.60	48.00	14.82	47.11	-0.89
9	14.40	48.00	16.94	47.27	-0.73
10	16.20	48.00	19.06	47.34	-0.66
11	18.00	48.00	21.18	47.53	-0.47
12	19.80	48.00	23.29	47.50	-0.50
13	21.60	48.00	25.41	47.69	-0.31
14	23.40	48.00	27.53	47.67	-0.33
15	25.20	48.00	29.65	47.85	-0.15
16	27.00	48.00	31.76	47.95	-0.05
17	28.80	48.00	33.88	87.99	39.99
18	30.60	48.00	36.00	88.16	40.16
				TOTAL	66

Victoria Transport Institute			Highway Capacity Manual		
Vehicle Number	Travel Time	Difference	Vehicle Number	Travel Time	Difference
1	46.91	-1.09	1	46.34	-1.66
2	46.91	-1.09	2	46.34	-1.66
3	46.91	-1.09	3	46.34	-1.66
4	46.91	-1.09	4	46.34	-1.66
5	46.91	-1.09	5	46.34	-1.66
6	46.91	-1.09	6	46.34	-1.66
7	46.91	-1.09	7	46.34	-1.66
8	46.91	-1.09	8	46.34	-1.66
9	46.91	-1.09	9	46.34	-1.66
10	46.91	-1.09	10	46.34	-1.66
11	46.91	-1.09	11	46.34	-1.66
12	46.91	-1.09	12	46.34	-1.66
13	46.91	-1.09	13	46.34	-1.66
14	46.91	-1.09	14	46.34	-1.66
15	46.91	-1.09	15	46.34	-1.66
16	46.91	-1.09	16	46.34	-1.66
17	46.91	-1.09	17	86.34	38.34
18	86.91	38.91	18	86.34	38.34
TOTAL		20.3	TOTAL		50.1