Redesign Frequency for Fixed-time and Vehicle-actuated Signal Controllers at A Right-turning Channelized Intersection

Qi An Transport & Planning, Civil Engineering

Project Supervisors
Dr. Ir. Andreas Hegyi TU Delft, Supervisor
Dr. Ir. Maria Salomons TU Delft, Supervisor

05th, December 2019

Abstract:

Fixed-time control and vehicle-actuated control are two main signaling strategies implemented at intersections for urban traffic management. The timing and structure of the controllers are usually designed optimally based on average historical demand patterns at the intersection. Under the premise of performance quality assurance, both fixed-time and vehicle-actuated controllers can accommodate a certain degree of demand fluctuations. As a matter of fact, the demand change can be considerable over the years, which could exceed their capacities in adapting such degree of demand change, and thus the signal controllers should be regularly updated to fit the latest demand. To some extent, how much demand change can be adapted by both types of the signal controllers determines how frequent should the controllers be checked and improved. However, only qualitative comparison of the capabilities in demand adaptation between fixed-time and vehicle-actuated controllers are made in most of existing literatures, according to which vehicle-actuated controllers are expected to have higher capabilities in accommodating demand changes. In this research, a quantitative analysis and comparison were made for the fixed-time and vehicle-actuated controllers at a right-turning channelized intersection under various demand conditions. Since no useful studies could be found to predict the demand changes towards a specific intersection at current phase, the extra demand that could be accommodated by vehicle-actuated controllers were investigated instead. And it is found that the vehicle-actuated controllers can serve a 19% to 204% of more demand compared with the fixed-time controllers in scenarios defined in this research, according to which the redesign frequency can be further determined for both types of controllers to maintain comparable operation performance.

List of Contents：

1. Introduction

Traffic signal control is one of the most important urban traffic management measures to address congestion problems **(Gershenson, 2004)**. The signal control strategies are often categorized into two types: fixed-time and responsive strategies. The fixed-time control is a pre-timed control strategy, which works offline with an appropriate optimization coding based on historical demands at the intersection (**Papâgeorgiou, 2004)**, has the advantages including the low initial investment and low ongoing maintenance costs, as well as providing pedestrians with regular and consistent intervals to cross the intersection **(NACTO, 2015)**, and the fixed-time strategy is often used in network-wide control systems with coordinated signals. One of the most typical responsive strategies is the vehicle-actuated (VA) signal controllers, which are widely implemented at isolated intersections **(Papâgeorgiou et al., 2007)**. The signal timing of vehicle-actuated controllers, in comparison, varies according to the actual traffic demand, which can adapt the real-time traffic fluctuations depending on the presence of vehicles measured by inductive loop detectors.

As a matter of fact, this kind of adaptation is only limited to a certain degree of demand fluctuations. Even the day pattern volume change of vehicles passing through an intersection is not significant over days and weeks, substantial variations could be observed over long time periods **(Sunkari, 2004)**. With the expected substantial demand change after a long time period, the existing timing structures of the signal controller at an intersection might not fit the demands safely and efficiently anymore, and the poorly-designed signal control structures are believed to have the potential of increasing the congestions, emissions and chance of safety problems **(Bonneson, Sunkari, Pratt & Songchitruksa, 2011)**. Therefore, the signal controllers should be regularly updated to accommodate the latest demand patterns, in order to maintain the safety and control efficiency at the intersection.

Updating the signal timing is one of the most common and cost-effective strategies for the improvement of signal controllers, especially when compared with other physical strategies (for instance adding extra lanes and grade separation) **(TAMU Mobility, 2011)**. The updating for signal controllers can be achieved by adjusting factors such as the phase structure, the cycle length, and the green split for each movement **(2030 Committee, 2011)**, which is as much necessary as patching potholes, removing snow and restriping pavement lines and markings. The improvements in signal timing are also estimated to create up to 40 dollars of road-user benefit with every 1-dollar investment by the transportation agency **(Sunkari, 2004)**.

Literatures regarding the comparison among different signal control strategies indicate that vehicle-actuated controllers are expected to produce less intersection delay compared with fixed-time controllers given the same traffic demand, due to the random nature of vehicle arrival patterns and VA's ability in adapting traffic fluctuations. However, the comparison in these studies is based on the delays estimated at specific intersections with historical demand data, only qualitive conclusions were drawn that VA controllers are generally with a better control performance in terms of less vehicle delay. Therefore, even a higher frequency is necessary for the redesigning of the control structures and signal timings of fixed-time controllers can be expected, as the fixed-time controllers are expected to be with less capability in fitting the demand variations, it is unknown how high the redesigning or the updating frequency should be, compared with VA controllers.

Furthermore, the redesigning frequency is not only determined by how the demand pattern changes, but also determined by how fast the changing trend is. However, the changing pattern of the vehicle volume towards an intersection is complex and complicated. The demand change is influenced by various factors including residential development, availability of public transit, restrict regulations and so on. Besides, the demand change influences the operation performance of the signal controllers at the intersection on one hand, and on the other hand, the performance also affects the demand since a new equilibrium point has to be reached. **Day et al. (2010)** investigated the methodology to quantify the benefits introduced by the retiming of traffic signals with the use of high-resolution signal event data and the Bluetooth MAC address matching technology. In their study, a 2.6% increase in traffic volumes was observed over 21 months with a 1.9-minute (20%) decrease in travel time. It indicates that the increase of intensity can also lead to less travel time for certain routes when they are perceived with longer travel times, and thus cause less road users choose these routes that passing through the intersection. In short, even the overall demand can be expected with a growing trend, it is hard to accurately predict how fast of the demand growth is, especially for a specific intersection, and thus the redesigning frequency based on the demand change is hard to be determined.

Based on the fact of the difficulties in predicting the demand growth rate at specific intersections, instead of assuming specific growth rates for the volume change of the vehicles and calculating the redesigning frequency based on the predefined growth rates, the extra demand that can be served by vehicle-actuated controllers compared with fixed-time controllers under different demand conditions is investigated, according to which the redesigning frequency can be easily determined afterwards, once studies on demand growth prediction at a specific intersection are available to be used.

Overall, this research is conducted to derive a quantitative comparison for the control performance between fixed-time and vehicle-actuated signal controllers under different demand conditions, which can help to provide insights when determining the redesigning frequencies for both control strategies to maintain a comparable level of operation performance.

2. Research Questions

Based on the background and the objective of this research illustrated in previous section, the main research question and four sub research questions are formulated in this section, by answering which the research objective can be achieved.

2.1 Main Research Question

"How frequent should the fixed-time controllers and vehicle-actuated controllers be redesigned to maintain a comparable level of performance at an isolated intersection?"

2.2 Sub Research Questions

1) *What is the level of performance for the signal control at an intersection? And at which level improvements on signal timings or structures should be considered?*
2) *How fixed-time and vehicle-actuated controllers should be designed to accommodate the given demand optimally?*
3) *How much of the demand growth can be served by the fixed-time controller before it needs to be redesigned?*
4) *How much of the demand growth can be served by the vehicle-actuated controller before it needs to be redesigned?*

3. Research Methodology

The research is conducted and consisted of four major parts: literature review, intersection setup, designing of the signal controllers, simulation and result analysis. A detailed description of the research methodology is illustrated in this section.

3.1 Literature Review

The relevant literatures are expected to include the signal timing of the fixed-time and vehicle-actuated controllers, performance indicators for the signal controlling at the intersection, and previous study on the performance comparison between two types of controllers under different conditions.

3.2 Design of Signal Controllers

There are two aspects in the designing of signal controllers: the control structure (also called as phase sequence) and the timings (cycle length and splits), which are usually determined by the historical traffic volumes of each stream towards the intersection, proprieties of the traffic and local regulations. Instead of designing the control structures and calculating the timings manually, a computer aided design program VRIGEN (a Dutch acronym for traffic control program generator) is used in this research to obtain the optimal control structure and the green times for signalized intersections, which is designed and developed in the Faculty of Civil Engineering and Geosciences at TU Delft. All possible control structures within the prescribed maximum cycle time can be generated by VRIGEN for the involved streams at the intersection. The derived control structures are presented in "Control Sequences" Tab in the program and sorted by the minimum cycle time and flexibility.

A critical path approach is used to determine the minimum cycle time of a structure **(Salomons, 2008)**, which is adjusted iteratively until all streams can be completely proceed as shown in Figure 1. The flexibility introduced in VRIGEN reflects the degree in which certain green phases can have earlier, longer or extra realizations accordingly adapted to the volume of conflicting movements. The flexibility is increased by 1 if a traffic stream has a parallel stream in successive block, and the more parallel streams in successive blocks, the more flexibility exists in the control structure **(Muller & de Leeuw, 2006)**. VRIGEN also shows the cycle time for each control structure calculated by Webster formulas according to the critical conflict group. The maximum extension green times are determined in VRIGEN by increasing the relative volumes of all streams simultaneously in small percentage steps until the pre-defined maximum cycle is reached **(Muller & de Leeuw, 2006).**

The main inputs for VRIGEN in order to generate control structures for the intersection with given traffic include traffic streams at the intersection, conflict matrix with clearance times, vehicle volume per stream, saturation flow rates, timing constraints such as maximum cycle time, minimum green times, start/end lags and connection times.

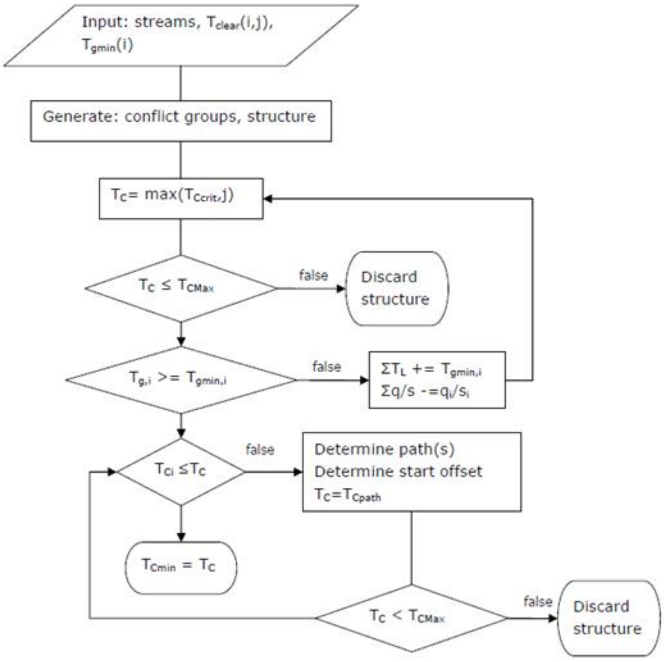


Figure 1. Minimum cycle time determination flow chart by VRIGEN

There are two control strategies possible with regarding to the signal phasing: stage-based control and stream-based control. In stage-based control systems, the green time is guaranteed for compatible streams that grouped in the same stage. Such group of streams are proceeded with green according to the leading/critical stream (stream with the highest demand), therefore, a proportion of green time will not be used by streams with lower demand, which causes wasted green and longer cycle time.

In comparison, stream-based control allocates the green time to each traffic movement without referring to a specific collection of traffic movements, and thus the condition with unused green could be avoid. As illustrated in previous study **(Tang & Nakamura, 2011)**, **(Bell & Brookes, 1993)**, **(Heydecker, 1996)**, **(Wong, Wong, Leung, & Tong, 2002)**, the stream-based (also called group-based, phase-based or movement-based) control system is more efficient than stage-based control system in operational performance such as vehicle delay because of the high flexibility in signal phasing and green time allocation.

To make the designed signal controllers optimally operated, a stream-based program TRAFCOD is used in this research, which allocates the green time to each stream and not stick to maintain a stage. TRAFOD is written and developed by TRAFCOL (TRAFfic COntrol Language), and has been used in the training of traffic engineering students in the Netherlands since 1978 **(Furth & Muller, 1999)**.

3.3 Intersection Setup and Simulation

A typical type of intersection implemented in modern mega cities (e.g. Zhengzhou, Hangzhou & Wuhan in China) is selected in this research, of which the right-turn traffic is channelized, shown as in Figure 2. The signal for the right-turning traffic is always green as there is no conflict with other vehicle streams, and it only turns red when there is need of waiting for passing pedestrians.

Figure 2. Typical layout of the channelized right-turning intersection

The demand input, together with the signal controllers designed with VRIGEN according to the demand profile will be simulated at this intersection with VISSIM. VISSIM is a microscopic traffic simulation software, individual vehicles are the basic units to be modelled in the simulation environment. It provides high level of details for traffic properties of vehicle units, including the position, velocity, and lane usage at any time at any location within the network **(Calvert et al, 2016)**. In the simulation of signal control at the intersection, various control performance indicators such as the cycle time, throughput volume, delay, stops and queue length can be directly derived from VISSIM by putting the corresponding detectors and data collection points into the network.

Scenarios are separated with different demand growing patterns, and the redesign frequencies for both types of signal controllers could be determined according to the performance results. A flowchart shown in Figure 3 can be taken as a conceptual strategy to determine the redesigning frequency for the signal controllers when the annual growth rate is predictable to be used. The initial structure of the controllers could be optimally designed via VRIGEN with the given initial demand. According the predefined threshold of the control performance and the simulation results from TRAFCOD and VISSIM, the times of the demand change can then be determined, which also represents the number of years. The changes in demand patterns for FT controller and VA controller should remain same, while the number of yearly changes that can be accommodated within a certain performance threshold by the FT and VA controllers is expected to be different. By comparing the year number “x” and “y” when the control performance could not satisfy the service requirement, the research questions can be answered.

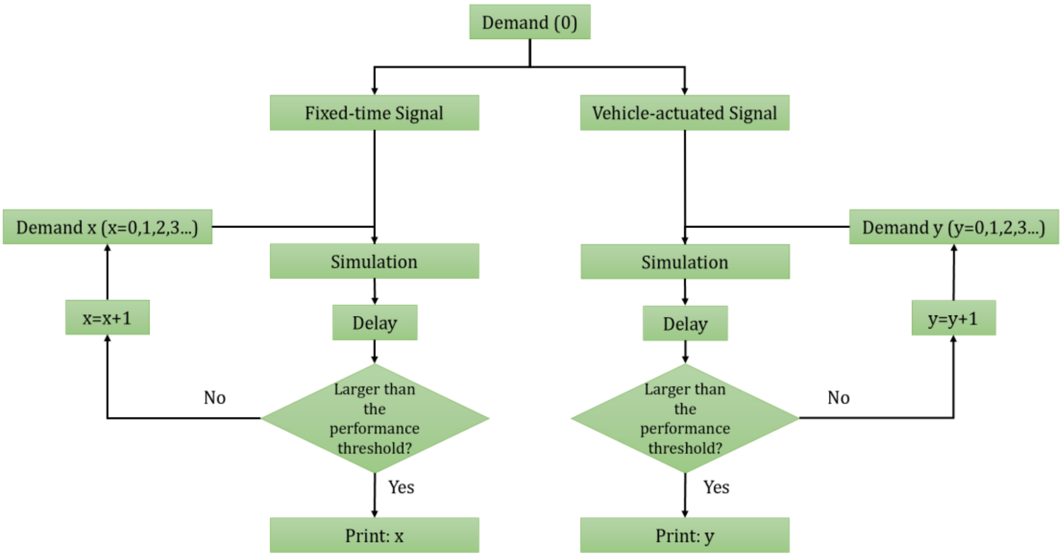


Figure 3. Flowchart of determining the redesign frequencies for FT and VA controllers

4. Literature Review

The literature review includes the introduction for the controlling algorithm of the fixed-time and vehicle-actuated controllers, previous performance evaluation and comparison studies for these two control strategies, as well as state-of-art of the calculation methods for the selected performance indicator (average vehicle delay) at an intersection.

4.1 Mechanism of Fixed-time and Vehicle-actuated Control Strategies

The signal control methods are used to build up the connection between observed traffic performance parameters (such as throughput, number of stops, delay and queue length) and signaling parameters including phase sequence/stage, cycle length and phase split **(Li, Yu, Tao & Chen, 2013).** These strategies are usually implemented with single-objective, weighted combination or even multi-objective optimization algorithms **(Cronje, 1983; Hu, Gao & Yang, 2010; Zhou & Cai, 2014)**.

The optimization strategies for fixed-time control are based on the prior knowledge of historical demands and turning rates of each stream (**Papâgeorgiou, 2004).** Webster method is one of the mostly used timing strategies for fixed-time controllers due to its form simplicity and acceptable level of accuracy, which is based on a single-objective optimization algorithm of minimizing the traffic delay for the whole intersection **(Dion and Hellinga, 2002).** In 1958, **Webster** derived an empirical model for calculating the vehicle delay based on steady-state queuing theory with an assumption of Poisson vehicle arrivals, by minimizing which formulas to determine the optimum cycle time and the green splits were introduced in his study, shown as following.

$$\mathrm{C} = \frac{1.5\mathrm{L} + 5}{1.0 - Y} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\, (4.1 - 1)$$

$$\mathrm{L} = nl + R \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\, (4.1 - 2)$$

C = optimum cycle length for a fixed-time control
Y = critical lane volume divided by the saturation flow, summed over the phases
L = lost time per cycle
n = number of phases (stages)
l = average lost time per phase due to starting delays
R = all red periods in each cycle

Once the optimum cycle length is obtained via equation (4.1 - 1) & (4.1 - 2), the green times are proportional split according to the volume/capacity ratios of the critical stream (stream with the highest volume) in each control phase. The optimum division of the cycle length should remain same degree of saturation for all phases of the intersection **(Webster, 1958).**

The Webster method is focusing on the overall intersection delay and optimizing the signal timings by equitably balancing the volume to capacity (V/C) ratios, which may lead to much longer delays for movements with low demand, and it is not desired for divers as they only perceive their own delay instead of the whole intersection delay. Considering the limitations of Webster or other similar V/C methods, optimization strategies based on individual movement delay were developed.

In the Highway Capacity Manual **(HCM)**, the procedures of critical movement delay methods were illustrated. Nowadays, the HCM delay methods (described later in section of overview introduction for delay calculation methods) has become the fundamental strategy to determine the optimum signal timing splits. The HCM method uses iterative procedures to balance the critical movement delays or prioritize certain movement delays without compromising other movement delays to exceed a certain level of acceptable performance **(Clark, 2007).**

Unlike the fixed-time controllers, the cycle time and the phase split of vehicle-actuated signals varies according to the presence of arriving vehicle activated by detectors, which can be on the basis of stage or stream control. A phase or stage can be skipped when no demand is detected **(Akcelik, 1994)**. Various settings of a vehicle-actuated controller including maximum cycle length, minimum green, gap time and green extension codetermine the efficiency of signal operation. The green times will be terminated when no vehicle is detected within the gap time or when the maximum green time is reached.

The green time of a VA signal is composed of a fixed *minimum green time* (normally 4 to 6 seconds) which is guaranteed to ensure a safe movement of the first vehicle at the stop line to move at the intersection, and a variable *actuation green time* to clear all vehicles between the detection point and the stop line. The extension of green time is constrained to the pre-defined maximum green **(Bullen, 1989)**. The maximum green is often determined based on local practice, and it should not be too restrictive for maximum possible flow rates with the consideration of inefficient operation during a unduly long green and cycle times **(Akcelik, 1994).**

4.2 Performance Comparison between FT and VA Controllers

The literature study on the control strategies comparison can give general information on the performance characteristics, limitations and difference between two types of signal controllers.

Previous studies have been done regarding the comparison between different control strategies. **Taale (2002)** compared the performance of four control strategies including the fixed-time control, normal vehicle-actuated control, adaptive control and control based on evolutionary algorithms at the crossing intersection of N57 and Oosterweg, Ouddorp. Fixed-time controller is found to be the worst with longest total delay of that intersection compared with other three control strategies under the three chosen demand scenarios.

Yulianto and Setiono (2012) introduced an adaptive signal control strategy based on Fuzzy logic at an isolated four-way intersection for mixed traffic (with high proportion of motorcycles) conditions, the control performance was also compared with fixed-time and vehicle-actuated controllers in terms of average delay per vehicle. The simulation results not only indicated the benefits introduced by the proposed adaptive controller, but also offered insights on the performance difference between vehicle-actuated control and fixed-time control in responding to same conditions. Vehicle-actuated controller generally leads to less delay comparing with fixed-time control when the traffic flows are not constant during the experiment period. However, the performance of vehicle-actuated controller tends to close to that of fixed-time controllers, as the added green could not be extended when the phase green has reached the pre-defined maximum green time. Their research shows that the increasing traffic flow will lower the control performance of vehicle-actuated controller.

4.3 Performance Evaluation Indicators for Signal Controllers

There are many operation objectives for the controlling of the intersection. A properly designed and timed signalized intersection should minimize the delay, stops, fuel consumption and emissions without compromising the safety requirements **(Bonneson, Sunkari, Pratt & Songchitruksa, 2011)**. A signal timing plan is called robust if its performance is less sensitive to traffic fluctuations, or if it performs better against the worst case without compromising optimality in the average sense **(Zhang, Yin & Lou, 2010)**. The signal controllers are recommended to be redesigned when the control performance can not satisfy certain requirements. Vehicle delay is one of the most commonly used performance indicators as it associates the direct relation to drivers' experience while crossing the intersection **(Zakariya & Rabia, 2016)**. It is defined as the difference between the actual travel time experienced by passing the intersection and travel time passing the intersection at the cruise speed without signal control, which is identified as an importance measure of operational effectiveness (MOE) for a signalized intersection, and it is well recognized as a reflective of the intersection performance **(NCHRP, 2001)**.

The *Highway Capacity Manual* **(HCM)** defined the vehicle Level of Service (LoS) at an intersection in terms of the delay experienced by a single vehicle to cross the intersection during the busiest 15 minutes of traffic of a day. It is required that designers and operators of the intersection should improve and maintain existing levels of service.

The **HCM** defines six ranks for the Level-of-Service for signalized intersections from A to F by representing a range of average control delay per vehicle, and it is generally not acceptable for most of the vehicles with LoS worse than rank D **(NCHRP, 2001; AASHTO, 2001)**. In this sense, the existing controllers "optimally" fitted for the initial demand should work "efficiently" with the control delay no worse than Rank D, therefore it can be defined that the controller needs to be redesigned when the control delay reaches the lower bounds of Rank D (35 seconds).

Table 1: Level-of-Service criteria for signalized intersections defined by HCM 2000. (Page. 164 or 351) Retrieved from https://sjnavarro.files.wordpress.com/2008/08/highway_capacital_manual.pdf

Level of Service (LoS)	**A**	**B**	**C**	**D**	**E**	**F**
Average Control delay * : Sec	**≤10**	**10–20**	**20–35**	**35–55**	**55–80**	**>80**

* "**Control delay**" defined in HCM 2000 refers to the component of delay that results when the control signal causes a lane group to reduce speed or to stop, including the slowing and waiting in the queue, and the accelerating time back to free-flow speed.

4.4 Development of Delay Calculation Methods

Though average vehicle delay is taken as the performance indicator which can be directly derived from microscopic simulation tool VISSIM, the mathematic methods for the delay calculation are also necessary to be illustrated to provide theoretical foundation of the delay measurement.

The delays are typically categorized by *stopped delays* which refer to the delay occurred when vehicle is standstill or moving at an extremely low speed, and *lost delays* which are defined as the lost times due to vehicles' acceleration and deceleration **(HCM, 2000)**. Those additional delays caused by drivers' reaction time and vehicle mechanical accelerating constraints are always hard to be estimated. Therefore, the operations of the signal are often expressed with effective signal intervals in delay models, the signal cycle is theoretically divided into stopped periods and traffic moving periods **(Dion, Rakha & Kang, 2004)**.

■ **State-of-art of delay models**

Dion et al. (2004) summarized several types of delay models and compared the consistency among those models. These models including *Deterministic queuing model*, *Shock wave delay model*, *Steady-state stochastic delay models* and *Time-dependent stochastic delay models*, all of which are compared with microscopic simulation delay models provided by INTEGRATION for a single-lane approach with fixed-timed control signal. In the reminder of this section, a brief introduction for these categories model is illustrated. Based on the comparison results and our research focus, the delay estimation by microscopic simulation model VISSIM can be justified.

1) Deterministic queuing model

By assuming a uniform arrival pattern of the vehicles at signalized intersection with high service rate and periodical stop interval, the deterministic queuing models are often used to predict the delay when the vehicle streams can be fully discharged during the green interval. Figure 4 is introduced to illustrate the resulting delays under deterministic queuing models, where the uniform delay is represented by the shaded area between the arrival and departure curves within a cycle.

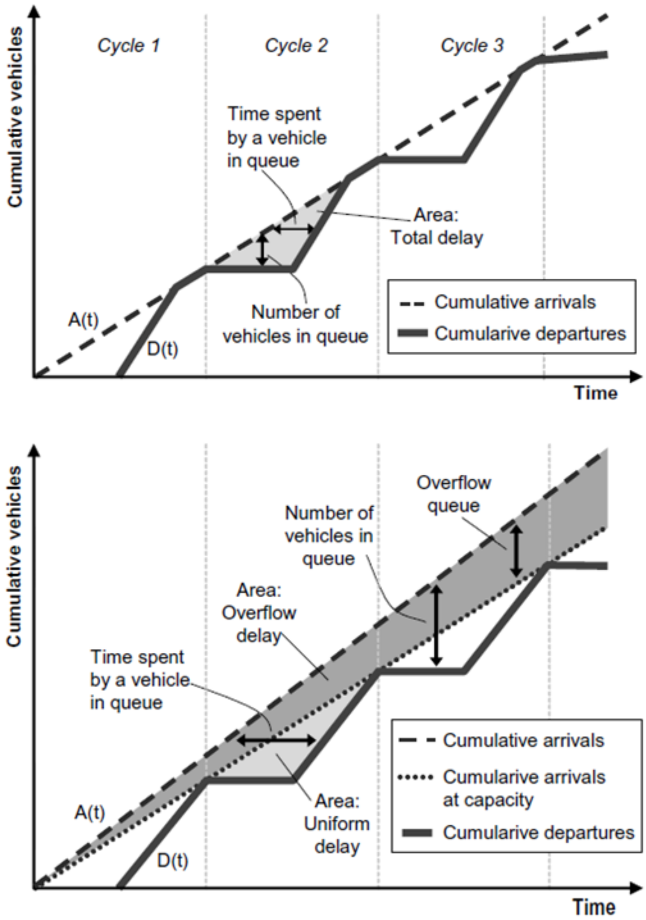


Figure 4. Cumulative vehicles for under- and over-saturated conditions (Dion, Rakha & Kang, 2004)

Equation (1) is derived to estimate the delays under non-saturated conditions.

$$\mathrm{d}_1 = \frac{c(1-\lambda)^2}{2(1-\lambda x)} = \frac{Tr_{eff}^2}{2 \cdot c(1-y)} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (1)$$

Where

d_1 = average delay per vehicle on particular approach of the intersection
c = cycle time
λ = proportion of effective green out of cycle time
x = degree of saturation. The ratio of the actual flow and the maximum flow that can pass the intersection given by x=q/ λs
Tr_{eff} = effective red interval (The time period when given traffic is directed to stop, which equals to the cycle length minus the effective green time).
y = load ratio, which is the ratio between arriving flow rate to saturation flow rate

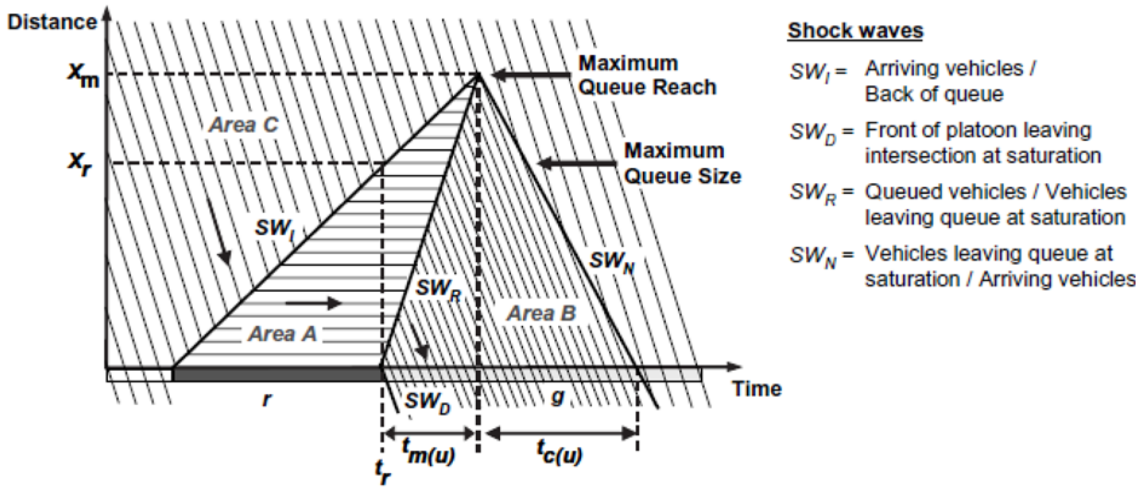
However, when the number of arriving vehicles exceeds the maximum number of vehicles that can be served with the traffic signal, the delay will continuously grow with the increasing evaluation period as a growing residual queue is expected to happen.

Therefore, equation (2) is derived to calculate the average delay over the discharged vehicles during the evaluation period T.

$$d_2 = \frac{3600T}{2}\left(\frac{q}{s \cdot \lambda} - 1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (2)$$

2) Shock wave delay model

Shock wave delay model was developed based on an analogy with fluid dynamics. Different with deterministic queuing models which assumes that the queue is vertical distributed, the horizontal extent of queue is also considered in shock wave delay model. Figure 5 illustrates the shock wave analysis, where the travel time can be estimated with the density and flow rate in each zone.



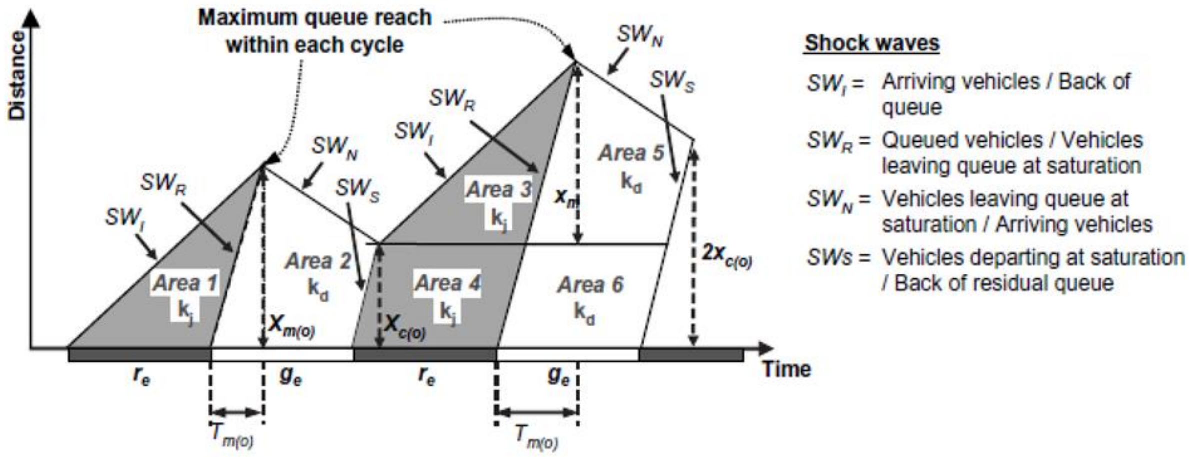


Figure 5. Shock wave analysis for under- and over-saturated approaches (Dion, Rakha & Kang, 2004)

$$d_3 = 3600\frac{|x_{m(u)}|}{2 \cdot q \cdot c} \cdot \left[Tr_{eff} \cdot (k_j - k_a) + (t_{m(u)} + t_{c(u)}) \cdot (k_d - k_a)\right] \ldots\ldots (3)$$

With:

$$x_{m(u)} = \frac{1}{3600} \cdot \left[\frac{-q \cdot Tr_{eff} \cdot s}{s(k_j - k_a) - q \cdot (k_j - k_d)}\right]$$

$$t_{m(u)} = \frac{q \cdot Tr_{eff} \cdot (k_j - k_d)}{s(k_j - k_a) - q \cdot (k_j - k_d)}$$

$$t_{c(u)} = 3600|x_{m(u)}| \cdot \frac{k_a - k_d}{q - s}$$

Where

$x_{m(u)}$ = maximum queue size within a signal cycle in under-saturated conditions (km)
k_j = jam density (veh/km)
k_a = density of approaching traffic (veh/km)
k_d = density of discharging traffic (veh/km)
$t_{m(u)}$ = time to maximum queue size in under-saturated conditions (s)
$t_{c(u)}$ = time to clear the queue in under-saturated conditions (s)

3) Steady-state stochastic delay models

Apart from deterministic queuing model and shock wave delay models which are derived by assuming uniform arrivals, stochastic delays models are developed to account for the randomness of the arrivals. **Webster (1958)** combined theoretical relationships of delay with simulation results, and derived a formula to estimate the vehicle delay which is expressed as:

$$\mathrm{d}_4 = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65\left(\frac{c}{q^2}\right)^{\frac{1}{3}} x^{(2+5\lambda)} \ldots\ldots\ldots\ldots\ldots (4)$$

Where

d_4 = average delay per vehicle on particular approach of the intersection
c = cycle time
λ = proportion of effective green out of cycle time
q = flow
s = saturation flow
x = degree of saturation. The ratio of the actual flow and the maximum flow that can pass the intersection given by x=q/ λs

The first term in equation (4) is theoretical derived from the assumption that vehicles are arriving at a uniform rate, while the second term is derived by assuming Poisson arrivals and a deterministic service rate to give some allowance for the random nature. The third term is empirically derived for the correction with higher flow values, which generally accounts for 10 percent of the first two terms.

Therefore, an approximate form of equation (4) can also expressed as:

$$\mathrm{d}_5 = 0.9\left\{\frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

However, such steady-state stochastic delay models assumed a constant distribution pattern (Poisson distribution) for the arrivals in given time intervals, headways between departures follow a known distribution with constant mean and similar driving behaviors, which leaves huge difference with reality (**Dion, Rakha & Kang, 2004).** Besides, the system is assumed to be under-saturated across the analysis period, and the calculated delay tends to infinity when the degree of saturation approaching one (as shown in Figure 6), which leads to the overestimation of optimal cycle time with highly saturated intersections **(Zakariya & Rabia, 2016)**.

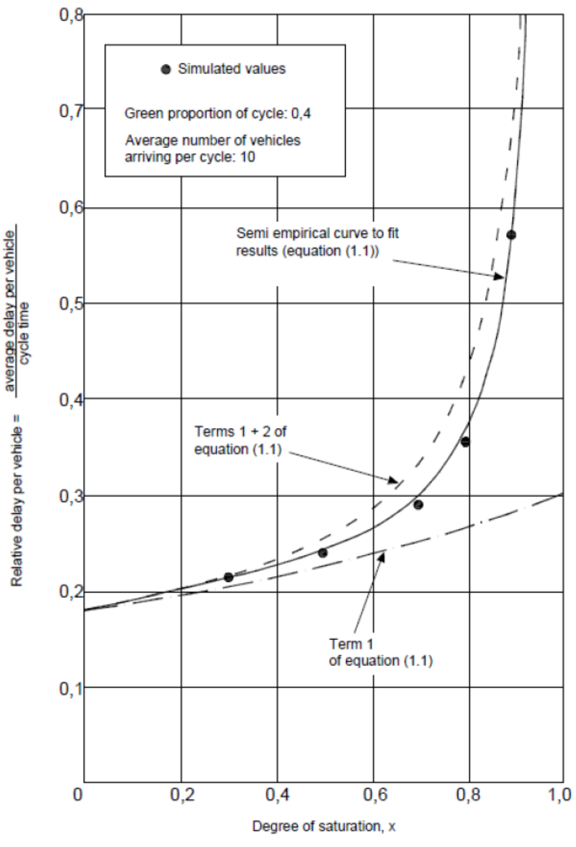


Figure 6. Typical fixed-time delay curve (Webster, 1958)

4) Time-dependent stochastic delay models

Owing to the limits of delay models described above, time-dependent delay models were derived to coordinate steady-state stochastic delay models in under-saturated conditions and deterministic delay models in over-saturated conditions, which is illustrated in Figure 7.

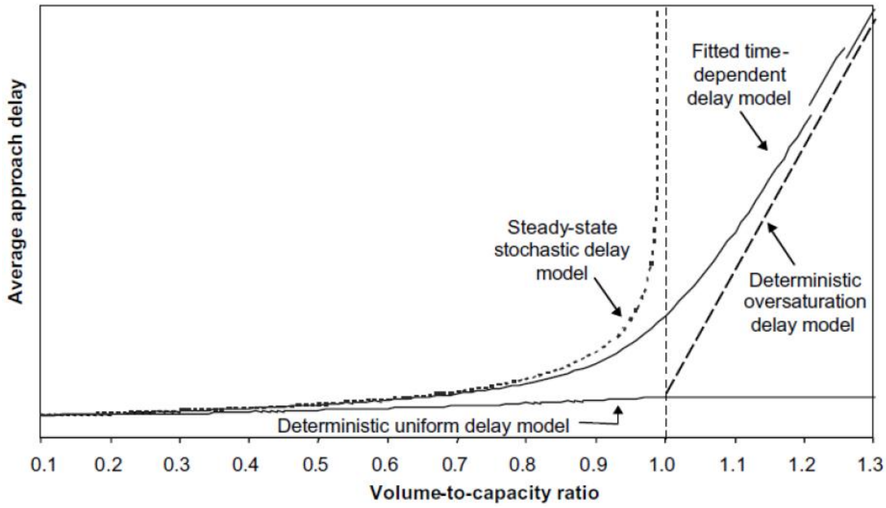


Figure 7. Stochastic time-dependent delay models (Dion, Rakha & Kang, 2004)

In Highway Capacity Manuals 2000 (HCM2000), the control delay is calculated with time-dependent stochastic delay models, which can be represented by the following equations (6) to (8).

$$\mathrm{d}_6 = d_6^1 \cdot PF + d_6^2 + d_6^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

With:

$$d_6^1 = \frac{0.5c\cdot(1-\frac{g_{eff}}{c})}{1-\frac{g_{eff}}{c}\cdot\min(1,x)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

$$d_6^2 = 900T\left[(x-1)+\sqrt{(x-1)^2+\frac{8k\cdot i\cdot x}{C\cdot T}}\right]\ldots\ldots\ldots\ldots \quad (8)$$

$$\mathrm{PF} = \frac{(1-P)f_{PA}}{1-\frac{g_{eff}}{c}}$$

Where

d_6^1 = uniform delay

d_6^2 = incremental delay caused by non-uniform arrivals, individual cycle failure and sustained over-saturated periods

d_6^3 = initial queue delay

PF = progression adjustment factor

g_{eff} = effective green time

P = proportion of all vehicles arriving during green

f_{PA} = supplement adjustment factor for platoon arrival during green

k = incremental delay adjustment for the actuated control

i = incremental delay adjustment for the filtering or metering by upstream signals

C = capacity of intersection approach, C = s · g_{eff}/c

T= analysis period

Note: The progression adjustment factor *PF*, *k-value* and *i-value* are specified in **HCM2000** (EXHIBIT 15-5, 15-6 and 15-7 (Page 320-321)).

In **Dion et al. (2004)**'s research, all involved delay models are believed to produce consistent delay estimations with microscopic traffic simulation models INTEGRATION when v/c ratio is below 0.6. When v/c ratio increases, only time-dependent stochastic models including *Australian Capacity Guide 1981*, *Canadian Capacity Guide 1995* and *HCM1997* still keep their consistency of delay estimation. However, deterministic models again show a visual consistency with time-dependent models when v/c ratio is larger than 1.1 (shown in Figure 8).

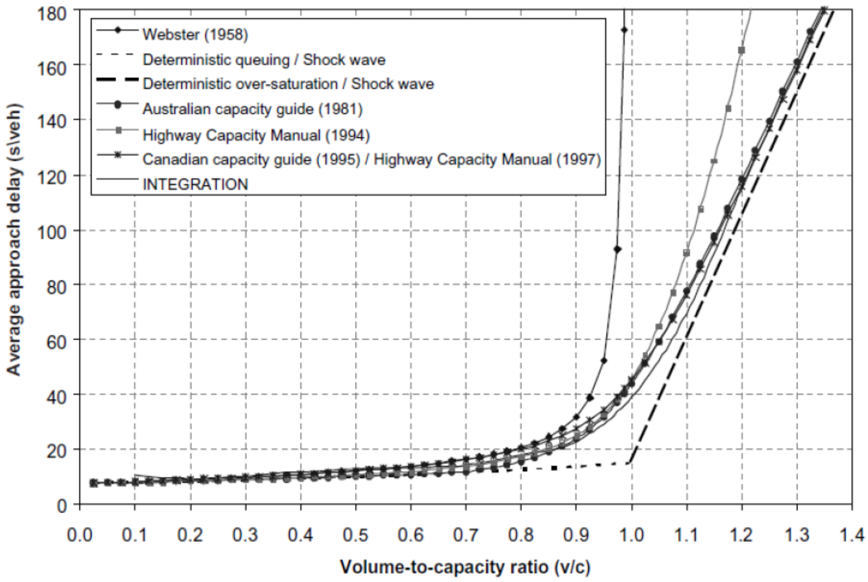


Figure 8. Delay estimations with different delay models

Based on their conclusion, time-dependent models are the best option to be chosen for analytical calculation of vehicle delays in both under-saturated and over-saturated conditions. However, one important involving parameter T in time-dependent models increases the computing complexity for average delays, especially in over-saturated conditions as estimated delays grow faster with increasing queue delays involved. As the high consistency of the delay results from microscopic simulation model INTEGRATION with the delay calculations by latest version of capacity manuals, it could be expected that the delay results from VISSIM (also a microscopic stochastic traffic model with similar capability in delay measurements) would also have consistent delay estimations with the capacity manuals **(Gao, 2008)**.

5. Signal Controller Design

As introduced in Chapter 3, the inputs required to determine the control structures by VRIGEN include the involving traffic streams at the intersection, the traffic volume per stream, saturation flow rates, clearance times for the conflict streams and timing elements (maximum cycle time, minimum green times, start/end lags, connection times). The physical layout of the intersection should be defined firstly before determining these inputs for the VRIGEN.

5.1 Intersection Layout

To modify an intersection with channelized right-turning lanes shown as in Figure 2, a simple four-approach intersection is implemented. As the degree of the operation improvements by signal timing is also affected by the intersection design, the key intersection design elements (listed below) should be satisfied to ensure the safe operation **(Bonneson, Sunkari, Pratt & Songchitruksa, 2011)**.

- Number of lanes provided for each movement;
- Length of turning bays;
- Presence of additional through lanes in the vicinity of the intersection;
- Detector locations;
- Use of left-turn phasing.

For single road section approaches, there are two lanes for through-going streams, one left-turning lane, and three receiving lanes. Auxiliary lanes for right-turning streams are implemented at the intersection, where the right-turning streams are canalized by four channelizing islands and pavement markings, which leave the right-turning traffic uncontrolled by signal controllers.

The north-south bound and east-west bound of the intersection are perpendicularly crossed. The pavement corner radius (Radius 38 m, length 60 m), and the channelization islands (24 m in length, with the area of 266 m^2 for each) are satisfied with the intersection geometric design requirements regulated by **AASHTO** (American Association of State Highway and Transportation Officials) and the intersection design guidebook from FHWA (Federal Highway Administration, United States). The layout for such an intersection is shown in Figure 9-1 & 9-2.

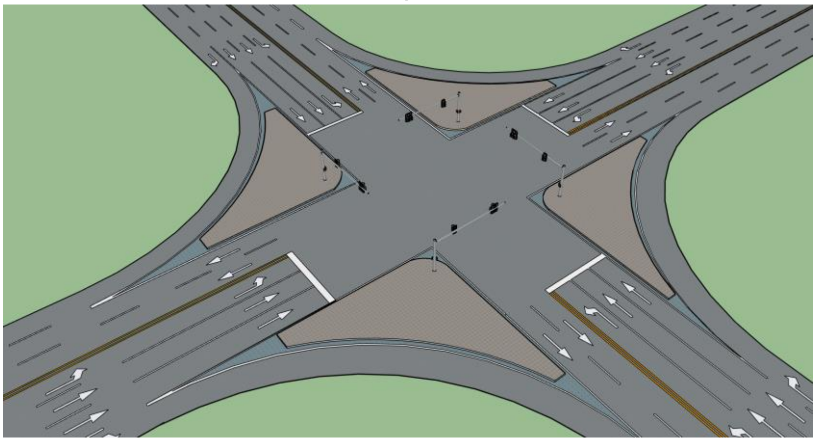


Figure 9-1. The layout of the simulating intersection (aerial view)

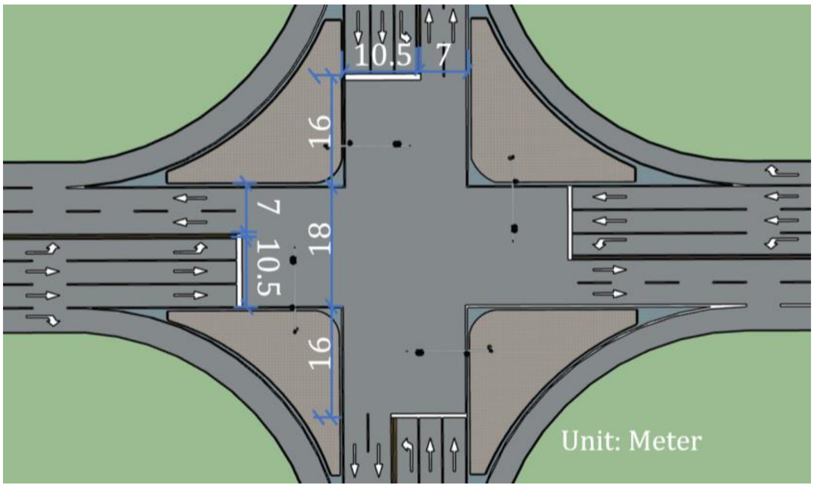


Figure 9-2. The layout of the simulating intersection (top view)

As the impacts brought by signal controllers on vehicle users passing through the intersection are more emphasized in this research, pedestrian and bicycle traffic involved at the intersection in this research are not considered to simplify the complexity, and the auxiliary lanes for canalized right-turning streams are assumed to be long enough to neglect the merging effects, therefore, the demand of right-turning streams can be taken as with no impact on other streams. Besides, the priority setting for certain transport mode (e.g. public transportation) is not involved, either.

5.2 Clearance Times

Clearance time, also called as "clearance interval" is defined as the period of time between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are given green" **(HCM 2000, p160)**. The clearance time often refers to a yellow change interval plus all-red interval.

The yellow interval is used to indicate that the related green movement is going to be terminated and a red phase is soon to be loaded **(ITE, 1994)**. The yellow interval can be determined by formulas described as ITE method:

$$\mathrm{y} = \mathrm{t}_r + \frac{v_{85appr}}{2a+2Gg} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(9)}$$

Where t_r is the driver perception/reaction time, v_{85appr} is 85th percentile speed of approaching vehicle, a is the deceleration rate, g is the acceleration rate due to gravity, and G is the grade of the approach.

The required yellow interval is provided to guarantee the approaching vehicle can either stop safely before the intersection or proceed through the intersection without accelerating.

The red clearance interval is used to guarantee the following traffic can safely enter the intersection without colliding with the last vehicle of previous traffic. Various methods existing to calculate the red clearance times. However, those methods are strongly differed in practice and there was no generally accepted method determining the interval length **(McGee & ITE, 2003)**. In respond to the identified need for its standardization for clearance times calculation from Dutch association of traffic control engineers (CVN) in 1992, **Muller, Dijker & Furth (2004)** developed a conflict zone method to calculate the red clearance interval. The new method is based on the driver behavior model that involves five parameters including the vehicle existing speed V_{exit}, drivers' reaction time t_r, acceleration rate with green signal a_{acc}, deceleration rate with red signal a_{dec}, and the maximum speed within the acceleration period V_{max}.

The resulting equations for calculating red clearance times developed by Muller et al are shown as below:

$$\begin{cases} t_{clearance} = t_{exit} - t_{entrance} \\ t_{exit} = S_{exit}/v_{exit} \\ t_{clearance} = t_r + \sqrt{\frac{2 \cdot s_{entrance}}{a_{acc}-a_{dec}}} & \text{if } s_{entrance} \leq s_{critical} \\ t_{entrance} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2\cdot(a_{acc}-a_{dec})} & \text{if } s_{entrance} \leq s_{critical} \\ s_{critical} = \frac{v^2{}_{max}}{2\cdot(a_{acc}-a_{dec})} \end{cases} \quad (10)$$

According to **Wilson & de Groot (2014)**, the reaction time is 1s, $v_{approach} = 14 \text{ m/s}$ (50 km/h) and $v_{exit} = 12 \text{ m/s}$ for through going vehicles, $v_{approach} = v_{exit} = 8 \text{ m/s}$ (30 km/h) for turning vehicles, and the vehicle length is 6 m. The parameter $(a_{acc} - a_{dec})$ is assumed a value of 2.5 m/s^2, and maximum speed V_{max} equals to the free-flow approaching speed according to **Muller et al. (2004)**.

1) **Conflict zones for through-going streams only** (conflict group 02-05/05-02, 05-08/08-05, 08-11/11-08, and 11-02/02-11).

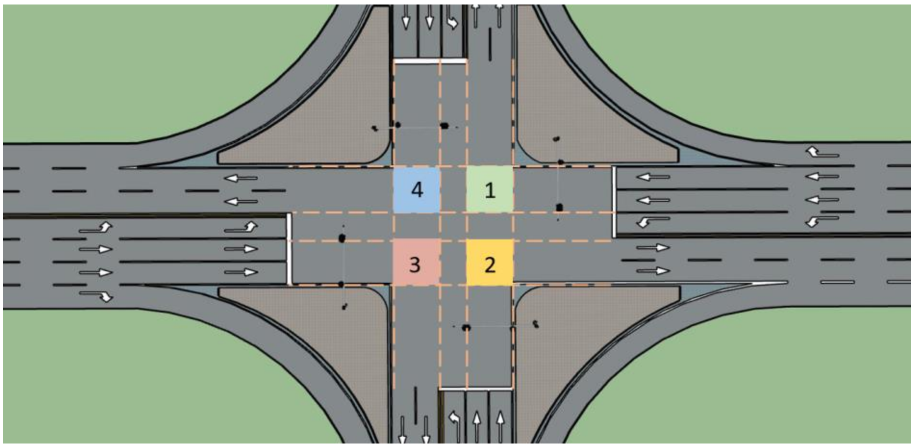


Figure 10. The conflict zones for through-going streams only

These conflict zones are shown in Figure 10 marked by "1", "2", "3", "4". The clearance times for a conflict pair of through-going streams that sharing the same conflict zones will be same with other conflict pairs as the layout of the intersection is geometrically symmetry, which can be calculated with the following equations:

$$s_{critical1} = \frac{v^2{}_{max}}{2\cdot(a_{acc}-a_{dec})} = \frac{14^2}{2\times 2.5} = 39.2 \text{ m}$$

$$s_{entrance1} = 16 \text{ m} < 39.2 \text{ m}$$

$$t_{entrance1} = t_r + \sqrt{\frac{2\cdot s_{entrance}}{a_{acc}-a_{dec}}} = 1 + \sqrt{\frac{2\times 16}{2.5}} = 4.58 \text{ s}$$

$$t_{exit1} = s_{exit1}/v_{exit1} = \frac{16+18+6}{12} = 3.33 \text{ s}$$

$$t_{\text{clearance1}} = t_{exit1} - t_{entrance1} = 3.33 - 4.58 = -1.25 \text{ s}$$

$$s_{entrance2} = 16 + 11 = 27 \text{ m} < 39.2 \text{ m}$$
$$t_{entrance2} = t_r + \sqrt{\frac{2 \cdot s_{entrance}}{a_{acc} - a_{dec}}} = 1 + \sqrt{\frac{2 \times 27}{2.5}} = 5.65 \text{ s}$$
$$t_{exit2} = s_{exit2} / v_{exit1} = \frac{16+7+6}{12} = 2.42 \text{ s}$$
$$t_{\text{clearance}2} = t_{exit2} - t_{entrance2} = 2.42 - 5.65 = -3.23 \text{ s}$$

2) **Conflict zones for left-turning streams only** (conflict group 03-06/06-03, 06-09/09-06, 09-12/12-09, and 12-02/02-12).

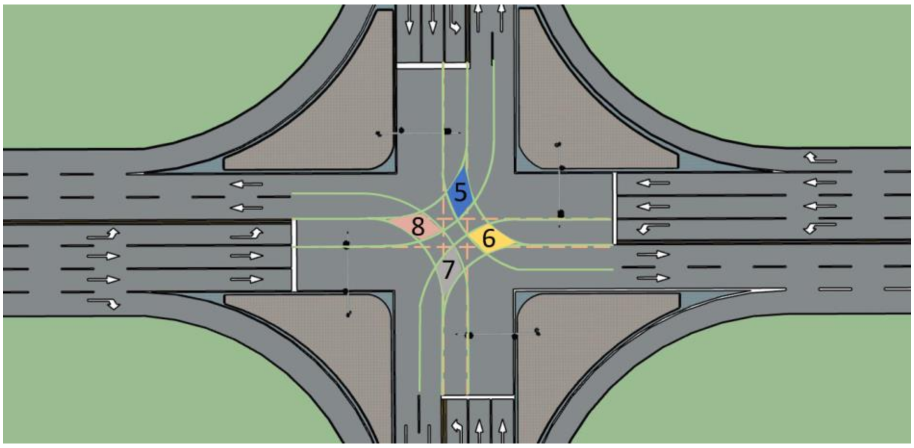


Figure 11. The conflict zones for left-turning streams only

These conflict zones are shown in Figure 11 marked by “5”, “6”, “7” and “8”. And the clearance times for a pair of left-turning conflict streams are calculated as below.

$$s_{critical2} = \frac{v^2{}_{max}}{2 \cdot (a_{acc} - a_{dec})} = \frac{8^2}{2 \times 2.5} = 12.8 \text{ m}$$
$$s_{entrance3} = 20 \text{ m} > 12.8 \text{ m}$$
$$t_{entrance3} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2 \cdot (a_{acc} - a_{dec})} = 1 + \frac{20}{8} + \frac{8}{2 \times 2.5} = 5.1 \text{ s}$$
$$t_{exit3} = s_{exit3} / v_{exit2} = \frac{32+6}{8} = 4.75 \text{ s}$$
$$t_{\text{clearance}3} = t_{exit3} - t_{entrance3} = 4.75 - 5.1 = -0.35 \text{ s}$$

$$s_{entrance4} = 28.3 \text{ m} > 12.8 \text{ m}$$
$$t_{entrance4} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2 \cdot (a_{acc} - a_{dec})} = 1 + \frac{28.3}{8} + \frac{8}{2 \times 2.5} = 6.14 \text{ s}$$
$$t_{exit4} = s_{exit4} / v_{exit2} = \frac{24+6}{8} = 3.75 \text{ s}$$
$$t_{\text{clearance}4} = t_{exit4} - t_{entrance4} = 3.75 - 6.14 = -2.39 \text{ s}$$

3) **The conflict zones for left-turning streams and through-going streams** (conflict group 03-05/05-03, 03-08/08-03, 03-11/11-03, 06-02/02-06, 06-08/08-06, 06-11/11-06, 09-02/02-09, 09-05/05-09, 09-11/11-09, 12-02/02-12, 12-05/05-12, 12-08/08-12)**.**

Those conflict zones are marked by “9”, “10”, “11”, “12”, “13”, “14”, “15” and “16” in Figure 12, 13 & 14. Three types of combinations between left-turning streams and through-going streams result in clearance times differ from case to case, and the three types of corresponding conflict zones are shown in Figure 6, 7 & 8 respectively.

For conflict groups 12-02/02-12, 03-05/05-03, 06-08/08-06, and 09-11/11-09 (with conflict zones marked by “9”, “11”, “13”, “15”), the corresponding calculated clearance times are:

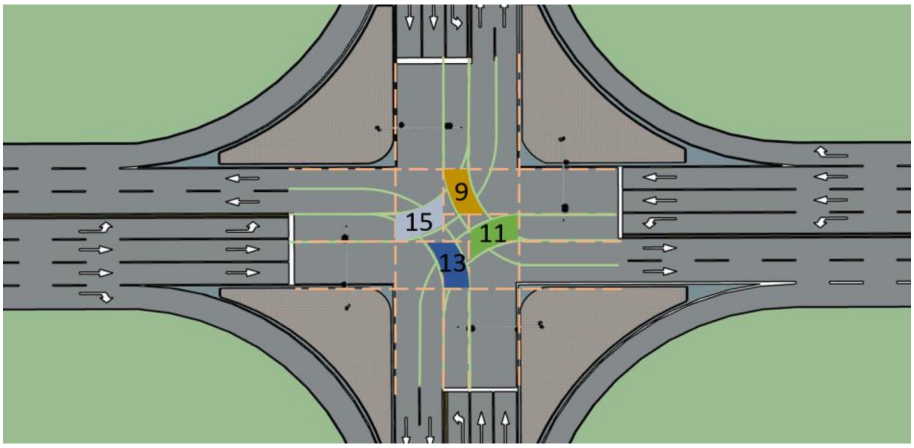


Figure 12. Type I conflict zones for through-going and left-turning streams

$$s_{critical2} = \frac{v^2{}_{max}}{2\cdot(a_{acc}-a_{dec})} = \frac{8^2}{2\times2.5} = 12.8 \text{ m}$$

$$s_{entrance5} = 16 \text{ m} > 12.8 \text{ m}$$

$$t_{entrance5} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2\cdot(a_{acc}-a_{dec})} = 1 + \frac{16}{8} + \frac{8}{2\times2.5} = 4.6 \text{ s}$$

$$t_{exit5} = s_{exit5}/v_{exit1} = \frac{27+6}{12} = 2.75 \text{ s}$$

$$t_{clearance5} = t_{exit5} - t_{entrance5} = 2.75 - 4.6 = -1.85 \text{ s}$$

$$s_{critical1} = \frac{v^2{}_{max}}{2\cdot(a_{acc}-a_{dec})} = \frac{14^2}{2\times2.5} = 39.2 \text{ m}$$

$$s_{entrance6} = 23\text{m} < 39.2 \text{ m}$$

$$t_{entrance6} = t_r + \sqrt{\frac{2\cdot s_{entrance}}{a_{acc}-a_{dec}}} = 1 + \sqrt{\frac{2\times23}{2.5}} = 5.29 \text{ s}$$

$$t_{exit6} = s_{exit6}/v_{exit2} = \frac{23+6}{8} = 3.53 \text{ s}$$

$$t_{clearance6} = t_{exit6} - t_{entrance6} = 3.53 - 5.29 = -1.76 \text{ s}$$

For conflict groups 09-05/05-09, 12-08/08-12, 03-11/11-03, and 06-02/02-06 (with conflict zones marked by "10", "12", "14", "16"), the corresponding calculated clearance times are:

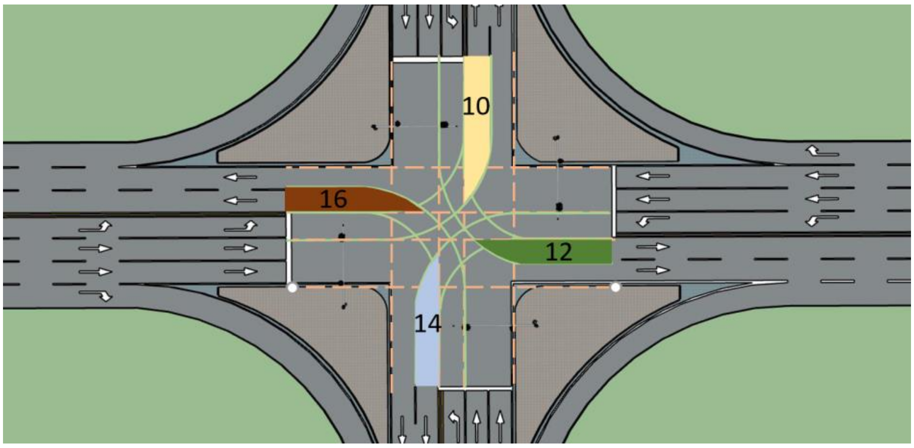


Figure 13. Type II conflict zones for through-going and left-turning streams

$$s_{critical1} = \frac{v^2{}_{max}}{2\cdot(a_{acc}-a_{dec})} = \frac{14^2}{2\times2.5} = 39.2 \text{ m}$$

$$s_{entrance7} = 37 \text{ m} < 39.2 \text{ m}$$

$$t_{entrance7} = t_r + \sqrt{\frac{2 \cdot s_{entrance}}{a_{acc} - a_{dec}}} = 1 + \sqrt{\frac{2 \times 37}{2.5}} = 6.44 \text{ s}$$

$$t_{exit7} = s_{exit7} / v_{exit2} = \frac{50+6}{8} = 7.00 \text{ s}$$

$$t_{\mathrm{clearance}7} = t_{exit7} - t_{entrance7} = 7.00 - 6.44 = 0.56 \text{ s}$$

$$s_{critical2} = \frac{v^2{}_{max}}{2 \cdot (a_{acc} - a_{dec})} = \frac{8^2}{2 \times 2.5} = 12.8 \text{ m}$$

$$s_{entrance8} = 35 \text{ m} > 12.8 \text{ m}$$

$$t_{entrance8} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2 \cdot (a_{acc} - a_{dec})} = 1 + \frac{35}{8} + \frac{8}{2 \times 2.5} = 6.97 \text{ s}$$

$$t_{exit8} = s_{exit8} / v_{exit1} = \frac{50+6}{12} = 4.67 \text{ s}$$

$$t_{\mathrm{clearance}8} = t_{exit8} - t_{entrance8} = 4.67 - 6.97 = -2.30 \text{ s}$$

For conflict groups 09-02/02-09, 12-05/05-12, 03-08/08-03, and 06-11/11-06 (with conflict zones marked by "17", "18", "19", "20"), the corresponding calculated clearance times are:

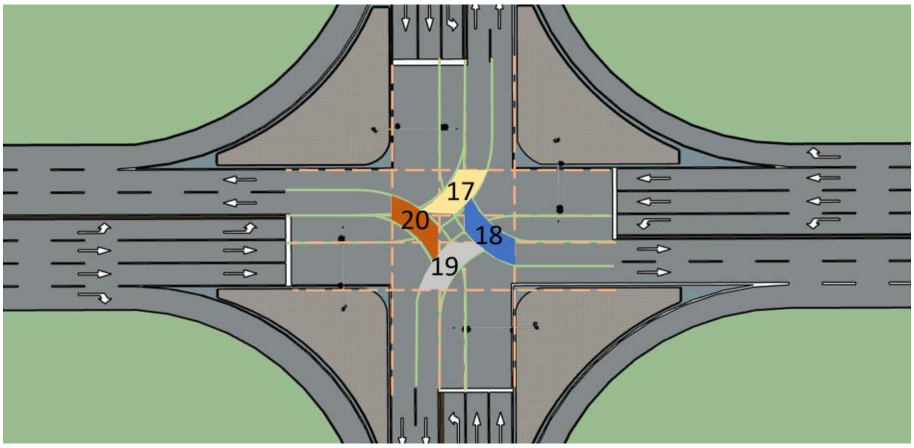


Figure 14. The conflict zones for through-going and left-turning streams

$$s_{critical1} = \frac{v^2{}_{max}}{2 \cdot (a_{acc} - a_{dec})} = \frac{14^2}{2 \times 2.5} = 39.2 \text{ m}$$

$$s_{entrance9} = 22.5\text{m} < 39.2 \text{ m}$$

$$t_{entrance9} = t_r + \sqrt{\frac{2 \cdot s_{entrance}}{a_{acc} - a_{dec}}} = 1 + \sqrt{\frac{2 \times 22.5}{2.5}} = 5.24 \text{ s}$$

$$t_{exit9} = s_{exit9} / v_{exit2} = \frac{34+6}{8} = 5.00 \text{ s}$$

$$t_{\mathrm{clearance}9} = t_{exit9} - t_{entrance9} = 5.00 - 5.24 = -0.24 \text{ s}$$

$$s_{critical2} = \frac{v^2{}_{max}}{2 \cdot (a_{acc} - a_{dec})} = \frac{8^2}{2 \times 2.5} = 12.8 \text{ m}$$

$$s_{entrance5} = 25 \text{ m} > 12.8 \text{ m}$$

$$t_{entrance5} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2 \cdot (a_{acc} - a_{dec})} = 1 + \frac{25}{8} + \frac{8}{2 \times 2.5} = 5.73 \text{ s}$$

$$t_{exit5} = s_{exit5} / v_{exit1} = \frac{27+6}{12} = 2.75 \text{ s}$$

$$t_{\mathrm{clearance}5} = t_{exit5} - t_{entrance5} = 2.75 - 5.73 = -2.98 \text{ s}$$

To summary, the corresponding calculated clearance times with equations (10) for all conflict pairs are listed in Table 2.

Table 2. Clearance times for conflict groups

	02	03	05	06	08	09	11	12
02			-3.23	-2.30		-2.98	-1.25	-1.85
03			-1.76	-2.39	-0.24		0.56	-2.39
05	-1.25	-1.85			-3.23	-2.30		-2.98
06	0.56	-0.35			-1.76	-2.39	-0.24	
08		-2.98	-1.25	-1.85			-3.23	-2.30
09	-0.24		0.56	-0.35			-1.76	-2.39
11	-3.23	-2.30		-2.98	-1.25	-1.85		
12	-1.76	-0.35	-0.24		0.56	-0.35		

Four conflict pairs are calculated with positive clearance times of 0.56 s, which means extra red time is required to clean the intersection before conflict movements are released, and these conflict pairs are marked by yellow in Table 3. For those clearance times calculated in negative, the values should be rounded up to zero second for safety reasons according to **Muller et al. (2004).**

5.3 Settings in VRIGEN

The intersection is implemented in VRIGEN to generate the control structures and timings with given demand profile. The estimated saturation flow for each link and the resulting clearance times obtained in previous section are modified to accommodate the characteristics of the simulated intersection.

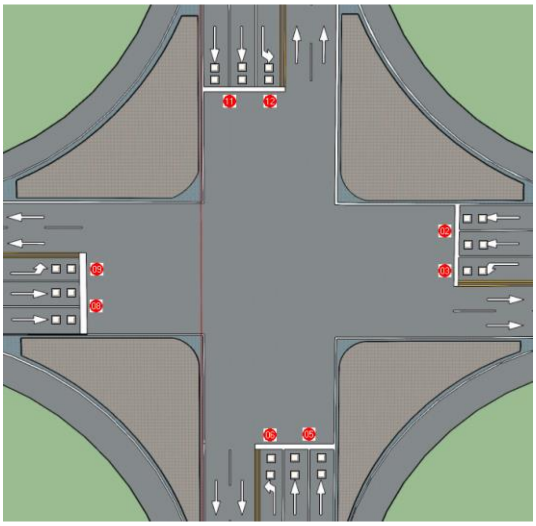


Figure 15. The generated control structures by VRIGEN

In VRIGEN, through-going streams and left-turning streams are selected, the numbers with standard coding for each stream are marked in red and shown in Figure 15. There are two white squares placed in each lane behind the stop line, which are the request detector (commonly located near the stop lines to detect the presence of vehicles for desired movements) and the extension detector (located upstream of stop lines in order to measure headways for gap acceptance logic, queue length and volume of approaching vehicles for added phase green time).

6. Intersection Simulation

In VISSIM, the lane width is set with a standard default value of 3.5 meters, the input vehicles are composed of 98% passenger vehicles and 2% HGV (Heavy Goods Vehicle). The default approaching speed for both types of vehicles is 50 km/h. The turning sections for left-turning streams are set with "Reduced Speed Area", where the turning vehicles are forced to be driven at a lower speed (defined as 30 km/h according to **CROW**). The crossing distance from the stop line to the edge of directing curbs is 50 meters for both through-going streams and left-turning streams.

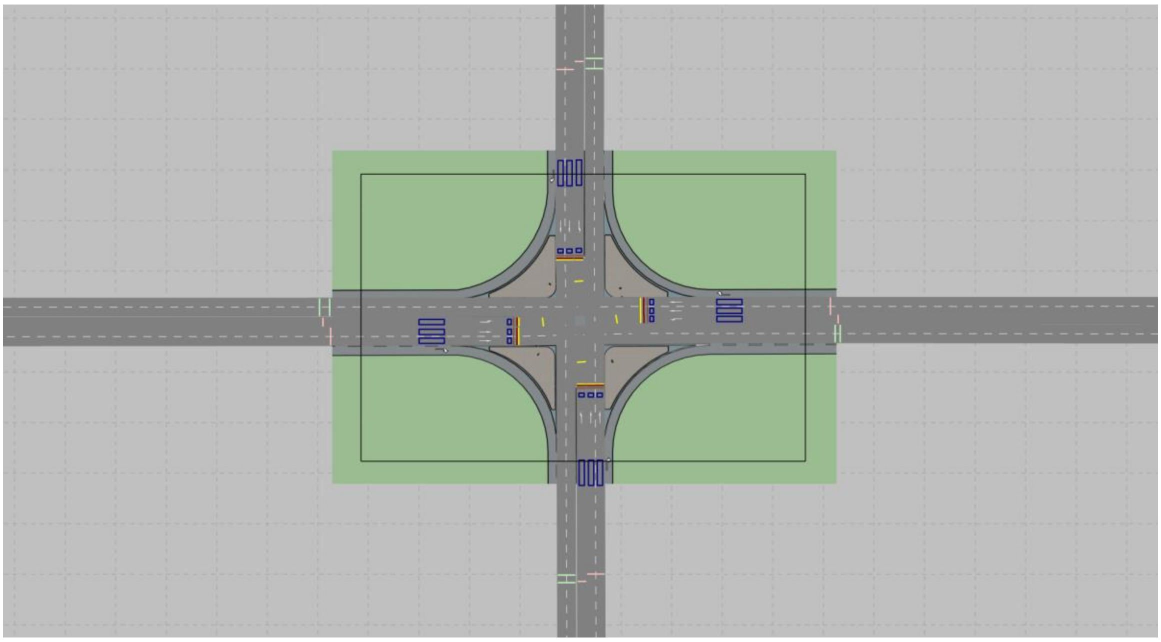


Figure 16. Intersection Modification in VISSIM

6.1 Saturation Flow Estimation

The saturation flow is estimated with a fixed vehicle input (9999 veh/h) setting for each stream, the saturation rates can be obtained by subtracting the number of vehicles that could not completely pass the intersection. The saturation estimation results are shown in Table 3.

Table 3. Estimation for the saturation flow

Stream	02	03	05	06	08	09	11	12
Saturation flow (veh/h)	2226*2	1804	2129*2	1900	2095*2	1872	2256*2	1915

To simplify the calculation and with the consideration of a relatively higher lane capacity in VISSIM simulations than in reality, it is assumed that all through-going streams and all left-turning streams are with the same demand inputs α and β, and the saturation flow rate is rounded to nearest hundred according to the minimum estimated values (reference value for through-going traffic and left-turning traffic is stream 08 and stream 03, respectively) listed in Table 3. Therefore, the saturation flow rate is taken as 4200 veh/h for straight-through streams, and 1800 veh/h for left-turning streams, these two values are used to determine the initial demand input (described in chapter 6.3). It is noted that the values listed in Table 3 are still used in VRIGEN for the determination of the control structures and timings.

6.2 Demand Scenarios

As the demand level at an intersection is a crucial factor that influence the signal timing and performance **(Li et al.,2013)**, a set of scenarios that consists of different demand patterns should be generated and evaluated. The scenarios can be divided into 9 categories according to the combination of volume-growing streams over the years:

1) The flow of one stream grows over the years.
By assuming the same initial demand for four approaches, which means that all four through-going streams (streams 02, 05, 08, 11 with standard coding) are with the same demand input α, and all four left-turning streams (streams 03, 06, 09, 12 with standard coding) are with the same demand input β, demand pattern with one-stream variations will have two conditions: the *through-going growth* and *left-turning* growth.

2) The flows of two streams grow simultaneously over the years.
Combinations of conflicting streams are considered. Stream 02 is conflict with streams 05, 06, 09, 11 and 12, stream 03 is conflict with streams 05, 06, 08, 11 and 12. As it is assumed that all four through-going streams and all four left-turning streams have same demands α and β. The combinations (03, 05), (03, 08), (03, 11) are same with (02, 12), (02, 09), (02, 06) respectively. Therefore, there are 7 unique categories with the combinations of two conflicting streams: (02, 05), (02, 06), (02, 09), (02, 11), (02, 12), (03, 06) and (03, 12).

According to the assumption and statements above, 9 scenarios are generated and defined according to the combinations of growing streams.

6.3 Initial Demand Determination

Before starting the growth for the demand patterns, an initial demand for the base scenario should be determined. Considering a case to be as generic as possible, a "*middle-level degree of saturation*" is chosen and defined as the initial demand in this research.

The determination of the initial demand is based on a four-stage fixed-time control with a maximum cycle time C_m of 120 seconds, with a 3-second yellow interval between green and red times. As in the initial demand is assumed to be same for any of the four approaches, the green times are supposed to be same in each control stage, therefore, the maximum phase green time for a four-stage control TG_m is 27 seconds ((120-3*4)/4). The full saturated flow rate for the signalized intersection (when the effective green time is 27s and no wasted green in each stage) is obtained via:

$$S_{full} = \frac{TG_m}{C_m} \cdot s \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots . (11)$$

According to the predefined saturation flow rates in chapter 6.1, the full saturated flow rate for the intersection with signalization is taken as 950 veh/h for through traffic and 400 veh/h for left-turning traffic. Therefore, the initial demand for a middle-level degree of saturation is set as half of the full-saturated flow rate (475 veh/h and 200 veh/h).

■ **Demand Growth Rate**
As illustrated previously, even a general demand growth can be expected over the years, how fast the demand will increase annually at one specific intersection is hard to be predicted. Therefore, instead

of assuming a fixed annual growth rate for the demand, the maximum increase of the demand within the predefined level-of-service is investigated, according to which the number of years can be determined once intensity studies at the intersection are available to be used for demand prediction.

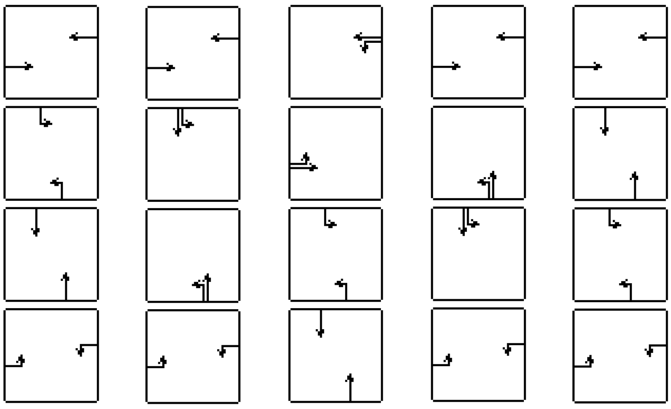
6.4 Phase Sequence Determination

The phase sequence of an intersection describes the predefined order for the allowed traffic movements with the giving signal display, which is also called as the structure of the controller. There are many variations possible for the phase sequences, and of which the selection should be based on the objective of providing the most efficient overall operation performance.

The control structure is determined by VRIGEN with the pre-determined initial demand, as well as the saturation rate per stream. The number of phases is always kept to a minimum as the increasing of the phase generally leads to higher delay in allowed traffic movements in other phases **(NCHRP, 2011).** In VRIGEN, all generated control structures are four phases (also called four stages), therefore, the criterion of phase numbers is not taken into consideration for the selection of the control structure.

With the predefined saturation flow rates and the given demand input of 475 veh/h for through-going streams and 200 veh/h for left-turning streams, 24 control structures in total are derived.

The control structure with the least minimum cycle time and the highest flexibility (the degree in which certain green phases can have earlier, longer or extra realizations when other conflicting green phases have no demand) should be selected as the optimal structure. As of a homogenous demand for all four approaches, there are more than one "optimum" structure available with the same minimum cycle times of 36 seconds and same flexibilities of 4, which are shown in in Figure 17. There is no strict difference among these six control structures, therefore, the first structure (a) is chosen in this study, which is used for both fixed-time and vehicle-actuated controllers. The cycle time of VA controller varies from 36 seconds to the maximum cycle time of 120 seconds, while the cycle time of the fixed-time control is constant at 41.09 seconds.



(a) (b) (c) (d) (e)

Figure 17. Six "optimal" control structures generated by VRIGEN

6.5 Number of Simulations

As the VISSIM simulation is a stochastic process, multiple simulations are required to derive reliable simulation results. Determining the number of simulations is important to derive reliable results with desired accuracy. It is designed in this study to obtain the average delay per vehicle with a reliability level of 95%.

Firstly, 12 pilot simulations with different random seeds setting were conducted to measure the vehicle delays. With the obtained vehicle delays, the average value and the accepted standard deviation can be used for determining the minimum required number of simulations N'.

$$\mathrm{N}' \geq t^2_{\frac{1}{2}\alpha,N-1}\left(1+\frac{1}{2}\xi^2\right)\frac{X_s^2}{X_d^2} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots. (12)$$

Where X_s is the sample standard deviation, X_d is the accepted deviation, α refers to the desired reliability, ξ is the abscissa or the normal distribution excess value, $t_{\frac{1}{2}\alpha,N-1}$ is the value which is obtained from the Student-t distribution.

Table 4. 12 pilot simulations for the measurement of control performance

Stream	**Estimation**	**Vehicle delay (VA)**	**Vehicle delay (FT)**
02	Average	20.12	19.18
	Standard Deviation	0.83	0.8
03	Average	22.66	19.92
	Standard Deviation	0.86	1.49
05	Average	20.2	18.42
	Standard Deviation	0.92	0.77
06	Average	21.98	20.85
	Standard Deviation	1.23	1.04
08	Average	20.22	18.12
	Standard Deviation	0.67	0.72
09	Average	22.47	20.37
	Standard Deviation	1.42	1.06
11	Average	20.09	19.82
	Standard Deviation	0.78	0.74
12	Average	22.17	20.72
	Standard Deviation	1.39	1.32

The minimum number of required simulations for the desired confidence level of 95% can be obtained based on the largest standard deviation value estimated in the pilot runs. As shown in Table 4, the largest standard deviation value is found for the stream 3 under the fixed-time controller, of which the average vehicle delay is 19.92 seconds. With the sample standard deviation 1.49s and an assumed accepted deviation of 1 second, the calculated minimum number of simulations required for fixed time controller is 11. Similarly, the standard deviation value is found to be largest for the stream 09 with the vehicle-actuated controller, and the minimum number of simulations required for an accepted deviation of 1s is 10.

Based on the calculated minimum number of simulation runs for the desired level of accuracy, 12 runs for a single simulation are conducted for both fixed-time and vehicle-actuated controllers, which is sufficient to derive reliable delay results.

6.6 Simulation Results

The delay results with the corresponding demand input, as well as the delay curves with the increasing demand for each scenario are summarized in this section.

The simulation is firstly conducted with the initial demand of 475 veh/h and 200 veh/h for through-going streams and left-turning streams, the delay results are summarized in Table 5.

Table 5: Delay results for the initial demand under fixed-time and vehicle-actuated controllers

Controller	Stream 02	Stream 03	Stream 05	Stream 06	Stream 08	Stream 09	Stream 11	Stream 12
VA	20.1s	22.7s	20.2s	22.0s	20.2s	22.5s	20.1s	22.2s
FT	19.2s	19.9s	18.4s	20.9s	18.1s	20.4s	19.8s	20.7s

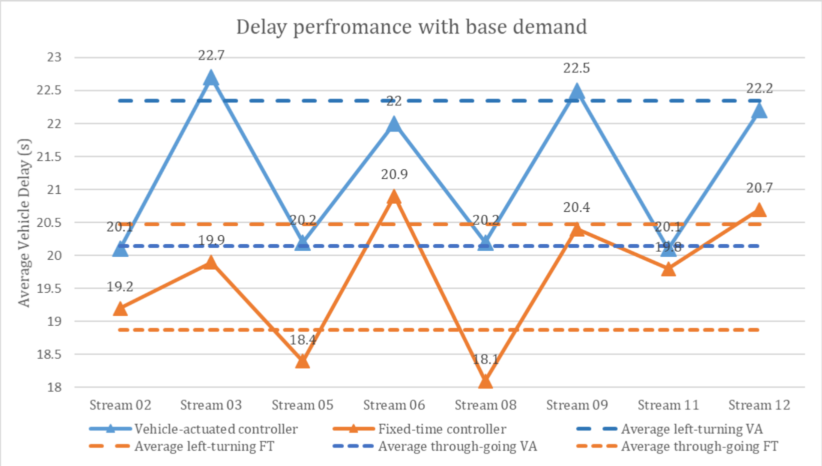


Figure 18. Delay performance with the base demand

It can be seen from Figure 18 that the delay results under vehicle-actuated controller is generally higher than the delays under fixed-time controller with the initial demand, and delays in left-turning streams are generally higher than that in through-going streams. The average vehicle delay under VA controller is 21.3s while the average delay under FT controller is 19.7s.

The delay results under VA controller are not as expected to be lower than the FT controller, it is caused by a longer wasted green time due to the extension activated by approaching vehicles far from the stop line, while the extended green can serve more vehicles than the actual number of vehicles. Shown in Appendix 1, the total green times during the simulation period (1 hour plus 10 minutes) is 5916.6 seconds for VA controller and 5767 seconds for FT controller, and the total actual green times allocated among four stages are calculated by summing up the dominating green times in each stage, which is 2973.6 seconds with VA control and 2949 seconds with FT control. The green times are distributed across 86 cycles under VA control and 101 cycles under FT control, which means that longer green times than its actual needs were loaded with VA controller in each cycle, and therefore more waiting times were required for other conflict streams at the same time.

It is expected that the delays will increase in all streams under the control of vehicle-actuated signal with the growing of the traffic, as the actual cycle time increases as well when longer green time is guaranteed to the growing streams, vehicles in other streams have to wait longer time to pass the intersection. For fixed-time controllers, the increasing delays are only expected to be observed in those volume-growing streams as the signal timing for other streams will not be affected. In this sense, changes in vehicle delay is more vulnerable for VA controllers when the demand is too small.

Take stream 02 and 03 as the reference through-going stream and left-turning stream, 9 scenarios in terms of stream and stream combinations are specified as:

- volume growth in stream (02);
- volume growth in stream (03);
- volume growth in stream (02,05);
- volume growth in stream (02,06);
- volume growth in stream (02,09);
- volume growth in stream (02,11);
- volume growth in stream (02,12);
- volume growth in stream (03,06);
- volume growth in stream (03,12).

1) Growth in one straight-through stream

Table 6a. Results of scenario 1 with VA controller (Growth in stream 02)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
26%	600	20.8	200	23.3	475	20.8	200	23.5	475	20.8	200	23.1	475	21.2	200	22.7
32%	625	20.8	200	23.4	475	20.6	200	23.0	475	20.9	200	22.9	475	21.0	200	23.1
37%	650	20.6	200	23.1	475	21.1	200	23.9	475	21.0	200	23.1	475	21.3	200	23.1
42%	675	20.8	200	23.9	475	21.2	200	23.6	475	20.9	200	23.1	475	21.4	200	23.8
57%	745	21.1	200	24.2	475	21.7	200	24.6	475	21.8	200	24.5	475	21.9	200	24.1
153%	1200	22.1	200	29.7	475	26.9	200	29.6	475	25.2	200	29.5	475	27.0	200	29.4
241%	1620	27.9	200	34.3	475	31.1	200	33.5	475	28.5	200	34.1	475	30.9	200	34.4
254%	1680	29.5	200	34.4	475	31.2	200	34.0	475	28.7	200	34.4	475	30.9	200	34.0
266%	1740	29.6	200	34.7	475	30.9	200	34.6	475	28.1	200	33.8	475	30.9	200	34.7
273%	1770	29.6	200	34.2	475	30.9	200	34.7	475	28.1	200	33.9	475	31.1	200	34.7
279%	1800	29.6	200	34.2	475	30.8	200	34.4	475	28.3	200	34.2	475	31.1	200	34.4

Table 6b. Results of scenario 1 with FT controller (Growth in stream 02)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
7%	510	20.1	200	20.4	475	18.4	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
14%	540	21.2	200	20.6	475	18.4	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
20%	570	22.0	200	20.6	475	18.4	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
26%	600	24.2	200	20.4	475	18.4	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
32%	625	26.0	200	20.1	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
37%	650	27.7	200	20.3	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
42%	675	32.9	200	19.8	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
43%	680	34.5	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
44%	685	35.3	200	20.1	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
47%	700	41.5	200	20.1	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
57%	745	59.4	200	20.1	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7

It can be seen from Table 6a that the delays under the control vehicle-actuated signal increase in all streams together with the growing in the volume of the stream 02. The average vehicle delays for left-turning streams remain overall 10% higher than the delays in through-going streams, and the delays stopped to increase when the demand growth reaches 254%, at which the highest delay was found to be 34.4s in stream 03, and the least delay was found to be 28.1s in stream 08. The stop in delay increment is caused by the fact that no more vehicles can be put into the intersection within the 70 minutes of simulation period, and there is no delay measurement for these extra vehicles.

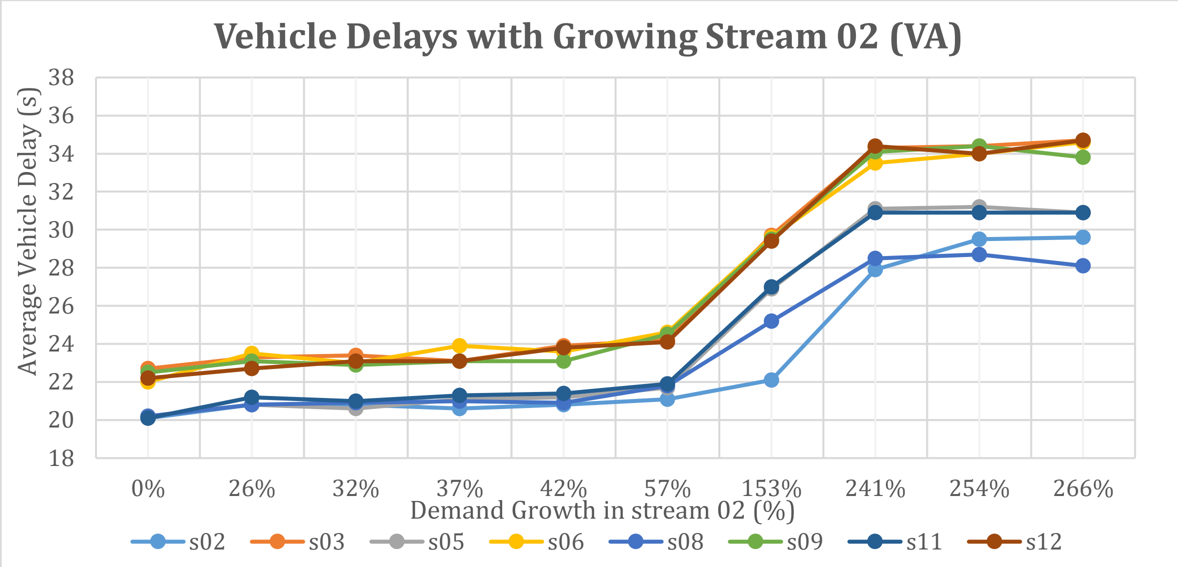


Figure 19-1. Vehicle delays with growing stream 02 (VA)

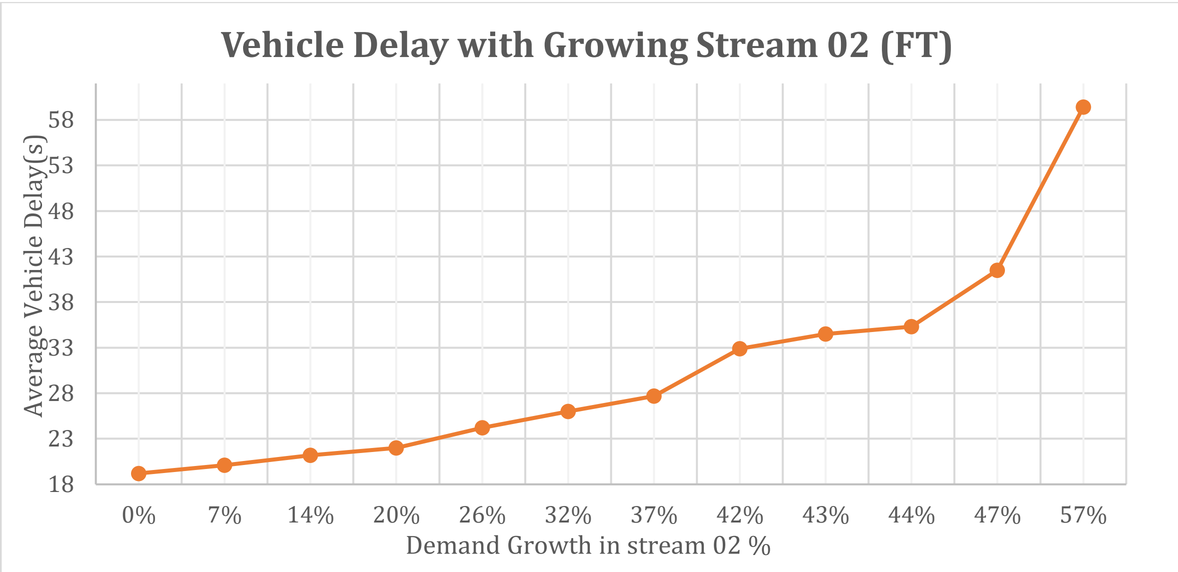


Figure 19-2. Vehicle delays with growing stream 02 (FT)

Overall, the delay results in left-turning streams are relatively consistent as shown in Figure 19-1, while the delays in through-going streams show some variations. The delays in stream 02 have been better maintained, which are generally lowest compared to other streams with the growing in demand. However, it is noticed that the delay in stream 08 becomes lowest when the demand growth exceeds 254%, which is reasonable considering the fact that stream 08 and stream 02 are in the same control stage, the extra green times actuated in order to accommodate the larger demand in stream 02 are allocated to stream 08 in the same time.

For the delays under the control of fixed-time signal, significant increase was only found in stream 02 as expected, and the delay growing pattern with FT control is shown in Figure 19-2.

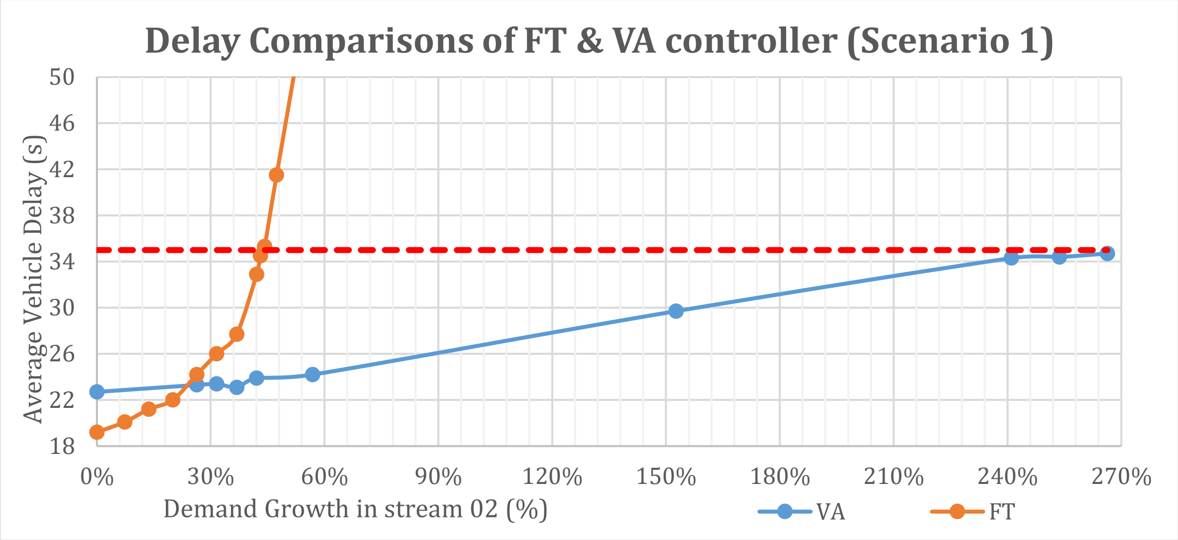


Figure 20. Delay comparisons between FT and VA controller (Scenario 1)

Shown in Figure 20 is the comparison between the delays in stream 02 under the fixed-time control and delays in stream 03 (which firstly reaches 35.0s with the increase of the demand) under the vehicle-actuated control. Considering the performance criterion in LoS rank D defined by HCM, for the same intersection, same given initial demand and with the same signal control structures, the vehicle-actuated controller can accommodate a 279% of volume increase in one through-going stream, while the fixed-time controller can only adapt to a 43% of demand growth. Therefore, the vehicle-actuated controller shows a better performance with 236% extra capacity in serving the volume growth in one through-going stream.

2) Growth in one left-turning stream

Table 7a. Results of scenario 2 with VA controller (Growth in stream 03)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
20%	475	20.5	240	23.2	475	20.7	200	23.0	475	20.4	200	23.0	475	20.9	200	22.4
50%	475	20.8	300	23.2	475	21.4	200	23.9	475	21.5	200	23.1	475	21.2	200	23.8
140%	475	22.0	480	24.5	475	23.9	200	26.2	475	25.0	200	26.1	475	23.8	200	26.2
200%	475	24.1	600	25.5	475	26.9	200	30.5	475	27.0	200	30.1	475	27.1	200	30.3
260%	475	27.5	720	29.1	475	30.0	200	33.7	475	30.6	200	33.2	475	29.7	200	32.7
290%	475	28.1	780	32.6	475	31.1	200	34.6	475	31.6	200	34.7	475	31.2	200	34.1
300%	475	28.1	800	33.2	475	31.8	200	34.6	475	31.9	200	34.5	475	31.5	200	34.7
305%	475	28.2	810	33.5	475	31.6	200	34.8	475	32.2	200	34.7	475	31.5	200	34.4
310%	475	28.1	820	33.5	475	31.7	200	34.1	475	32.5	200	34.4	475	31.5	200	34.6
315%	475	28.2	830	33.6	475	31.9	200	35.3	475	32.3	200	35.3	475	31.5	200	34.8

Table 7b. Results of scenario 2 with FT controller (Growth in stream 03)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
10%	475	19.1	220	21.7	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
20%	475	19.1	240	23.0	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
25%	475	19.2	250	24.4	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
35%	475	19.1	270	26.6	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
40%	475	19.2	280	26.8	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
45%	475	19.3	290	29.4	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
50%	475	19.2	300	31.0	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
60%	475	19.2	320	34.5	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
62%	475	19.1	324	36.6	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
65%	475	19.1	330	39.1	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
305%	475	19.1	810	90.5	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7

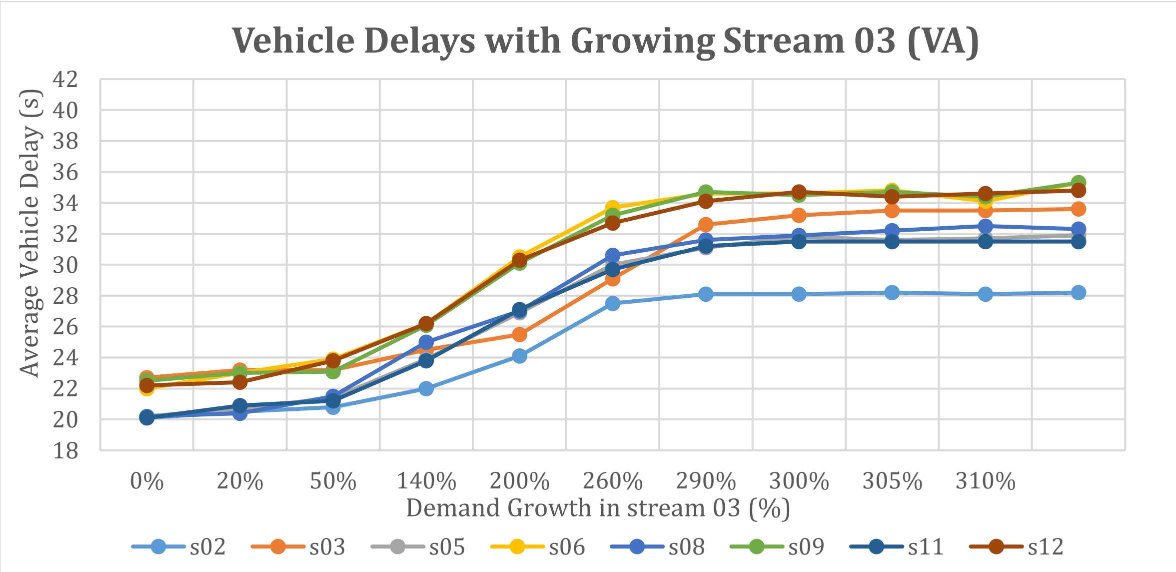


Figure 21-1. Vehicle delays with growing stream 03 (VA)

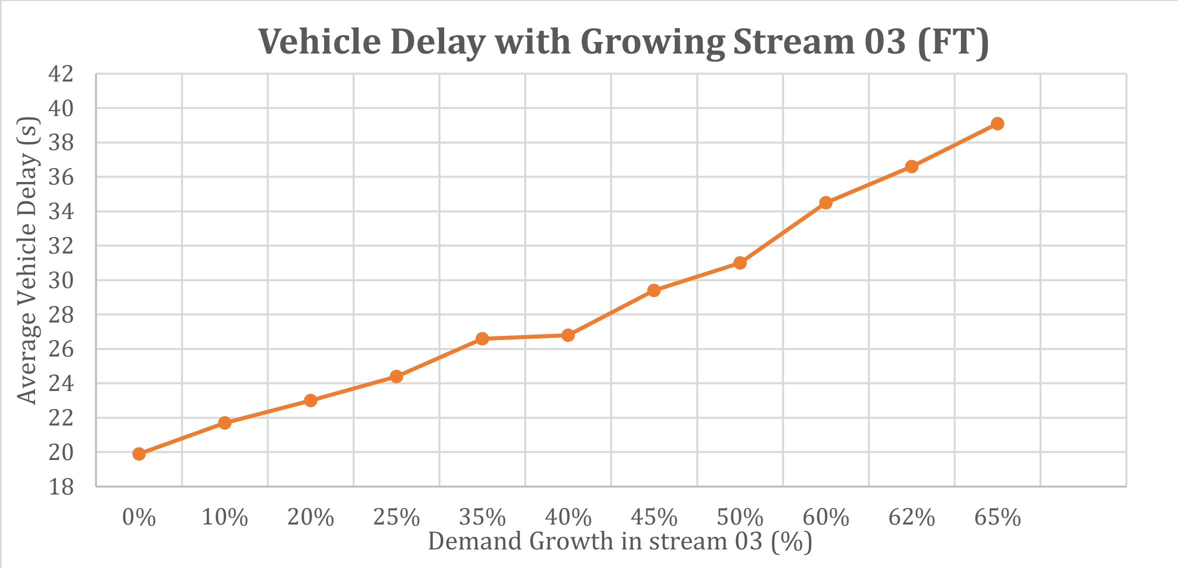


Figure 21-2. Vehicle delays with growing stream 03 (FT)

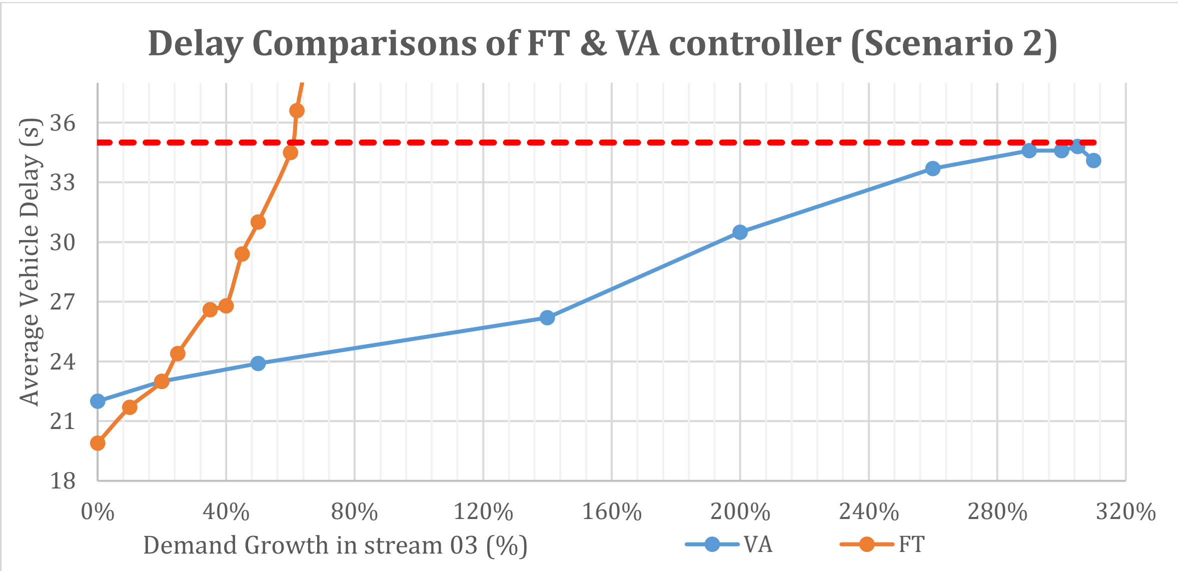


Figure 22. Delay comparisons between FT and VA controller (Scenario 2)

The highest delay was found in stream 06 under the control of vehicle-actuated controller by increasing the vehicle volume in stream 03, under the condition of which a 315% increase can be adapted within the performance criterion of 35s for the average vehicle delay. In comparison, the delay with fixed-time control reaches 35.0s when 61% of the volume growth was loaded on stream 03. Compared with scenario 1, both VA and FT show larger capabilities in demand growth adaption in scenario 2, where VA can provide 254% extra capacity within the criterion of 35s for the average vehicle delay compared with FT control.

3) Growth in two conflict streams (02, 05)

Table 8a. Results of scenario 3 with VA controller (Growth in stream 02 and stream 05)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
14%	540	21.2	200	22.3	540	20.5	200	22.2	475	20.8	200	23.4	475	20.8	200	23.3
20%	570	21.1	200	23.3	570	21.2	200	23.5	475	20.7	200	23.1	475	21.3	200	23.7
26%	600	21.4	200	23.7	600	21.5	200	23.9	475	20.8	200	23.5	475	21.3	200	23.9
50%	711	22.5	200	25.4	711	22.2	200	25.3	475	22.6	200	25.2	475	22.4	200	25.5
57%	745	22.8	200	26.6	745	22.6	200	26.5	475	23.4	200	25.9	475	23.5	200	26.1
89%	900	25.0	200	30.6	900	24.9	200	30.7	475	26.5	200	30.6	475	26.4	200	31.0
115%	1020	26.8	200	34.8	1020	27.5	200	34.5	475	30.5	200	34.6	475	29.5	200	34.8
121%	1050	27.7	200	36.4	1050	28.4	200	37.1	475	30.3	200	36.9	475	30.7	200	34.7
127%	1080	28.7	200	37.0	1080	28.7	200	36.6	475	31.5	200	36.9	475	30.9	200	37.0
153%	1200	32.3	200	40.4	1200	32.7	200	41.9	475	34.8	200	41.8	475	34.0	200	41.1
241%	1620	42.3	200	43.2	1620	43.2	200	43.1	475	35.4	200	42.8	475	36.6	200	42.7

Table 8b. Results of scenario 3 with FT controller (Growth in stream 02 and stream 05)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
7%	510	20.3	200	20.5	510	19.3	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
14%	540	21.0	200	20.5	540	20.0	200	21.0	475	18.1	200	20.4	475	19.8	200	20.7
20%	570	22.7	200	20.1	570	21.0	200	21.1	475	18.1	200	20.4	475	19.8	200	20.7
26%	600	25.6	200	20.3	600	22.0	200	21.3	475	18.1	200	20.4	475	19.8	200	20.7
32%	625	27.2	200	20.3	625	23.6	200	21.3	475	18.1	200	20.4	475	19.8	200	20.7
37%	650	30.9	200	20.1	650	25.5	200	20.7	475	18.1	200	20.4	475	19.8	200	20.7
43%	680	34.3	200	20.8	680	28.6	200	20.5	475	18.1	200	20.4	475	19.8	200	20.7
44%	684	34.7	200	20.2	684	29.2	200	20.8	475	18.1	200	20.4	475	19.8	200	20.7
45%	690	36.7	200	20.1	690	29.5	200	20.5	475	18.1	200	20.4	475	19.8	200	20.7
50%	711	40.8	200	20.2	711	32.8	200	20.6	475	18.1	200	20.4	475	19.8	200	20.7

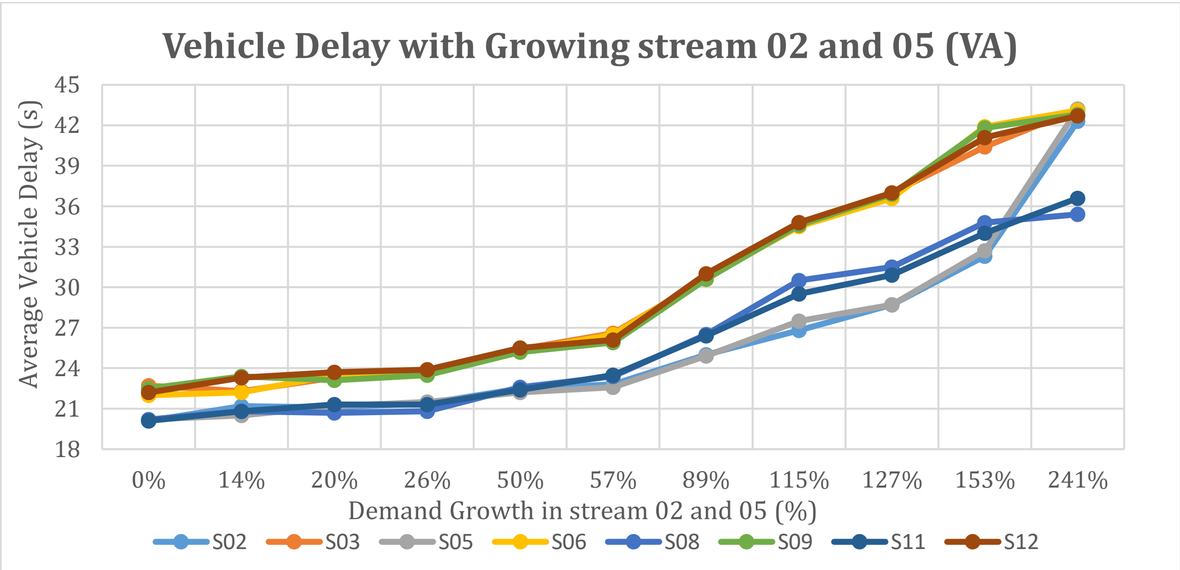


Figure 23-1. Vehicle delays with growing stream 02 and 05 (VA)

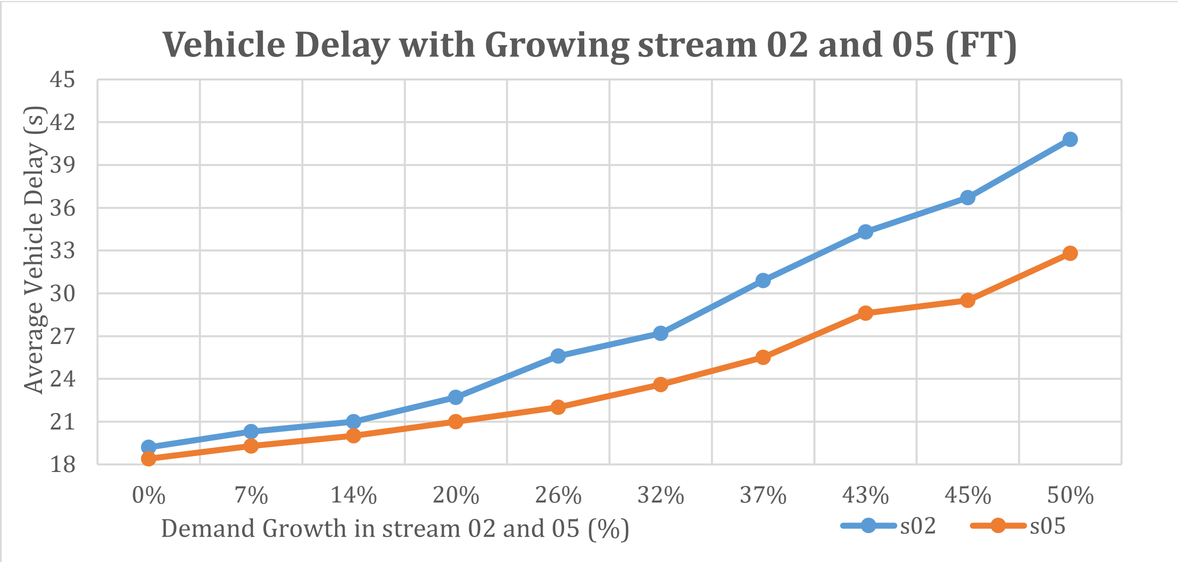


Figure 23-2. Vehicle delays with growing stream 02 and 05 (FT)

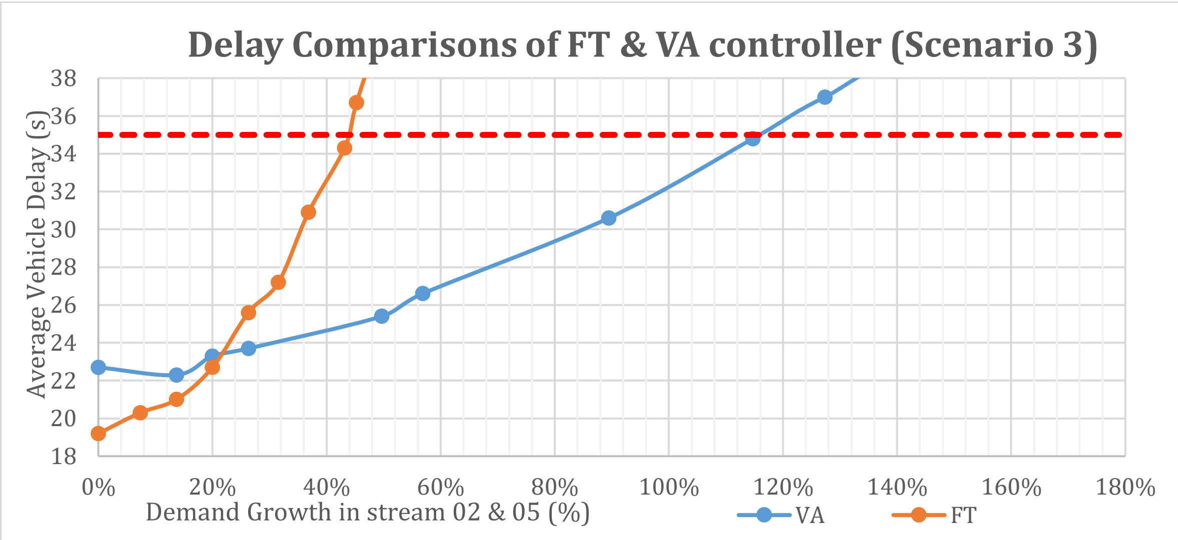


Figure 24. Delay comparisons between FT and VA controller (Scenario 3)

In the scenario of demand growth in stream 02 and stream 05 at the same time, the vehicle-actuated controller can serve 115% (in stream 12) demand increase, where the FT control can accommodate a 44% (in stream 02) of demand increase. The performance comparison in scenario 3 is a 71% difference for two types of signal controllers.

4) Growth in two conflict streams (02, 06)

Table 9a. Results of scenario 4 with VA controller (Growth in stream 02 and stream 06)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
14%	540	20.7	200	23.3	475	20.4	227	23.1	475	20.7	200	23.2	475	20.7	200	23.5
26%	600	21.5	200	23.7	475	21.1	253	23.4	475	21.0	200	23.7	475	21.3	200	23.0
33%	630	21.3	200	24.0	475	20.7	265	24.2	475	21.5	200	23.6	475	21.9	200	23.8
39%	660	21.6	200	24.5	475	21.2	278	24.7	475	21.4	200	24.2	475	22.5	200	24.5
57%	748	22.5	200	26.5	475	21.9	315	25.0	475	22.7	200	25.7	475	23.1	200	25.8
77%	840	22.9	200	27.8	475	23.3	354	26.7	475	24.7	200	27.4	475	25.2	200	27.5
100%	950	24.8	200	30.3	475	26.0	400	28.9	475	26.5	200	30.5	475	28.1	200	30.6
132%	1100	26.9	200	34.8	475	28.8	463	32.5	475	30.1	200	34.7	475	32.4	200	34.8
136%	1122	27.1	200	35.6	475	29.8	472	33.0	475	30.6	200	35.5	475	33.0	200	35.0
140%	1140	27.9	200	35.6	475	30.3	480	33.8	475	30.8	200	36.3	475	34.0	200	35.7
150%	1188	29.0	200	38.0	475	31.8	500	35.1	475	32.0	200	37.6	475	35.2	200	37.7

Table 9b. Results of scenario 4 with FT controller (Growth in stream 02 and stream 06)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
14%	540	20.9	200	20.7	475	18.4	227	22.4	475	18.1	200	20.4	475	19.8	200	20.7
20%	570	22.0	200	20.4	475	18.3	240	23.7	475	18.1	200	20.4	475	19.8	200	20.7
26%	600	24.2	200	20.1	475	18.3	253	24.4	475	18.1	200	20.4	475	19.8	200	20.7
33%	630	27.0	200	19.8	475	18.5	265	26.5	475	18.1	200	20.4	475	19.8	200	20.7
36%	645	29.4	200	20.4	475	18.4	272	26.4	475	18.1	200	20.4	475	19.8	200	20.7
39%	660	32.4	200	20.5	475	18.4	278	27.2	475	18.1	200	20.4	475	19.8	200	20.7
41%	670	34.2	200	20.0	475	18.4	282	28.2	475	18.1	200	20.4	475	19.8	200	20.7
43%	680	36.2	200	20.2	475	18.4	286	29.9	475	18.1	200	20.4	475	19.8	200	20.7
45%	690	39.0	200	20.1	475	18.4	291	30.8	475	18.1	200	20.4	475	19.8	200	20.7
52%	720	44.4	200	20.0	475	18.4	303	33.0	475	18.1	200	20.4	475	19.8	200	20.7
57%	748	57.2	200	20.4	475	18.4	315	34.5	475	18.1	200	20.4	475	19.8	200	20.7

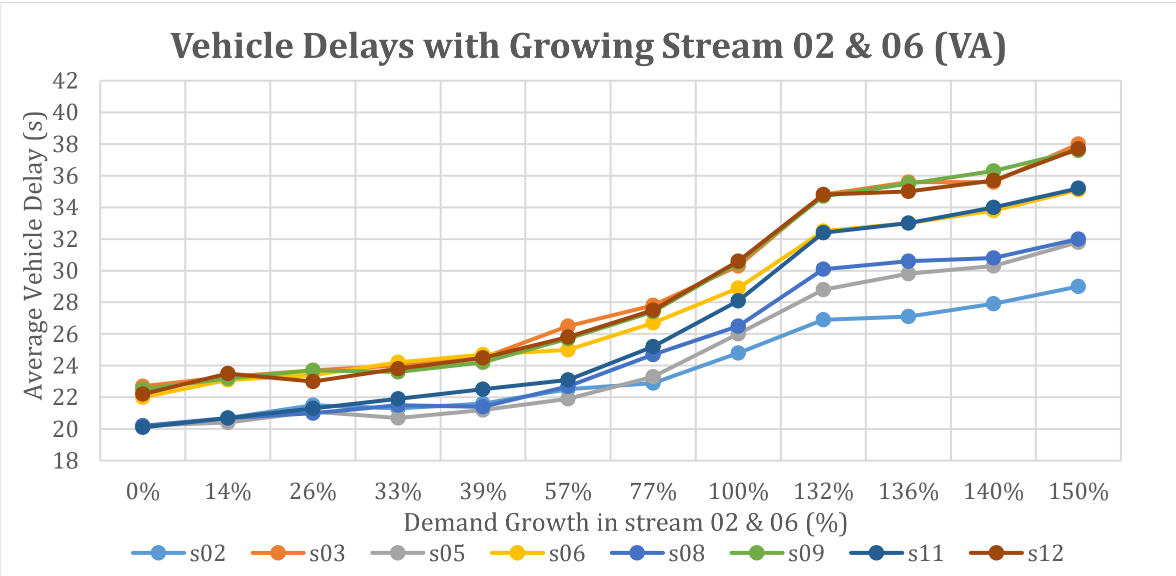


Figure 25-1. Vehicle delays with growing stream 02 and 06 (VA)

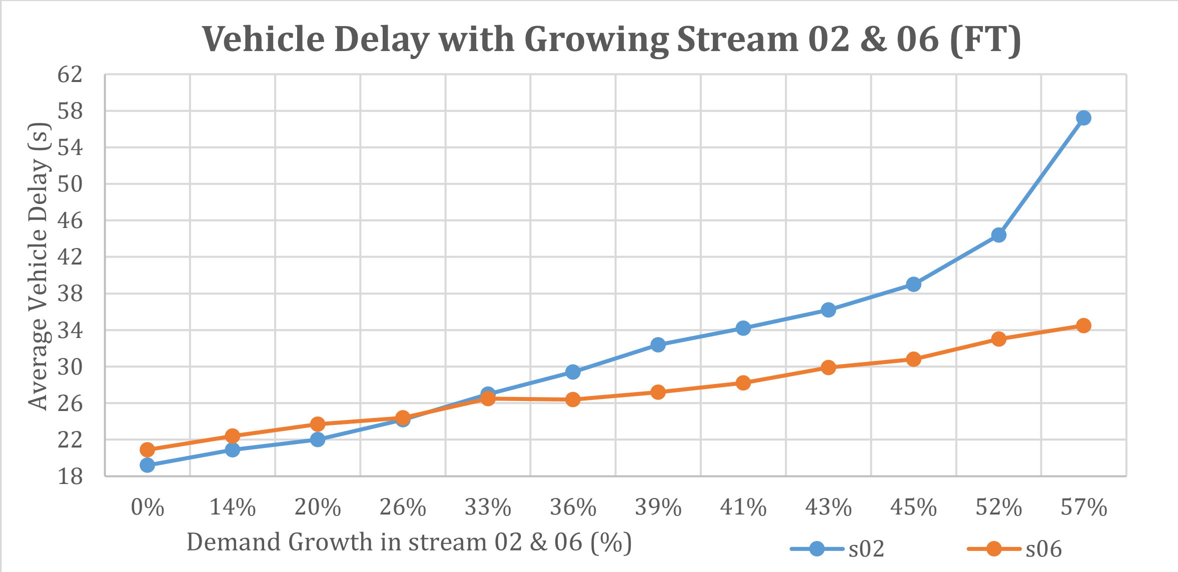


Figure 25-2. Vehicle delays with growing stream 02 and 06 (FT)

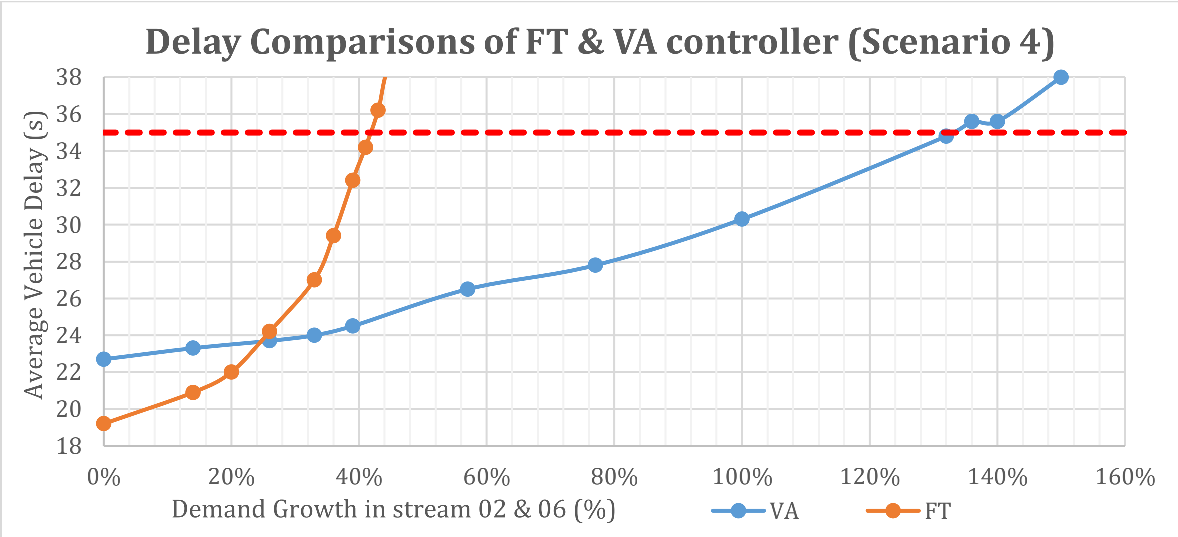


Figure 26. Delay comparisons between FT and VA controller (Scenario 4)

In scenario 4, 132% of demand growth (in stream 03) and 42% of demand growth (in stream 02) can be accommodated by the vehicle-actuated controller and fixed-time controller respectively. A 90% of volume growth in stream 02 and stream 06 can be served with the extra capacity provided by the vehicle-actuated controller.

5) Growth in two conflict streams (02, 09)

Table 10a. Results of scenario 5 with VA controller (Growth in stream 02 and stream 09)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
14%	540	21.0	200	23.1	475	20.6	200	22.6	475	20.3	227	23.1	475	21.0	200	22.7
33%	630	21.0	200	24.0	475	21.3	200	24.2	475	20.9	265	24.0	475	21.8	200	24.2
45%	690	21.9	200	24.2	475	22.7	200	24.8	475	21.0	291	24.1	475	22.7	200	25.2
57%	748	22.2	200	26.5	475	23.4	200	26.2	475	21.5	315	25.2	475	23.6	200	25.3
77%	840	23.4	200	28.1	475	25.3	200	28.2	475	21.9	354	27.2	475	25.4	200	27.8
100%	950	25.0	200	31.6	475	28.4	200	31.3	475	22.2	400	29.3	475	28.8	200	30.2
111%	1000	25.7	200	32.7	475	29.6	200	33.0	475	22.4	421	30.1	475	29.9	200	31.8
121%	1050	26.3	200	34.5	475	31.7	200	34.4	475	22.5	442	31.3	475	31.4	200	34.7
124%	1062	26.9	200	35.3	475	31.8	200	34.8	475	22.6	447	31.6	475	31.7	200	34.9
125%	1068	27.2	200	35.4	475	31.8	200	35.2	475	23.0	450	31.6	475	31.8	200	35.1
132%	1100	27.9	200	36.4	475	33.0	200	36.8	475	23.0	463	32.5	475	33.3	200	35.9
150%	1188	30.1	200	38.7	475	35.6	200	38.7	475	23.3	500	35.3	475	35.8	200	39.0

Table 10b. Results of scenario 5 with FT controller (Growth in stream 02 and stream 09)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
14%	540	20.9	200	20.7	475	18.3	200	20.7	475	18.2	227	22.2	475	19.8	200	20.7
20%	570	22.0	200	20.4	475	18.3	200	20.9	475	18.0	240	23.3	475	19.8	200	20.7
26%	600	24.2	200	20.0	475	18.3	200	21.2	475	18.1	253	24.3	475	19.8	200	20.7
33%	630	27.0	200	19.8	475	18.3	200	21.1	475	18.0	265	25.3	475	19.8	200	20.7
37%	650	30.5	200	20.1	475	18.2	200	20.8	475	17.9	274	26.5	475	19.8	200	20.7
39%	660	32.4	200	20.5	475	18.3	200	21.3	475	17.9	278	27.5	475	19.8	200	20.7
41%	670	34.2	200	20.0	475	18.4	200	20.6	475	18.1	282	27.8	475	19.8	200	20.7
43%	680	36.2	200	20.2	475	18.3	200	21.0	475	18.0	286	28.7	475	19.8	200	20.7
45%	690	39.0	200	20.1	475	18.2	200	21.3	475	17.9	291	29.0	475	19.8	200	20.7
52%	720	44.4	200	20.0	475	18.4	200	20.8	475	18.0	303	32.3	475	19.8	200	20.7
57%	748	57.2	200	20.4	475	18.3	200	21.2	475	18.1	315	34.0	475	19.8	200	20.7

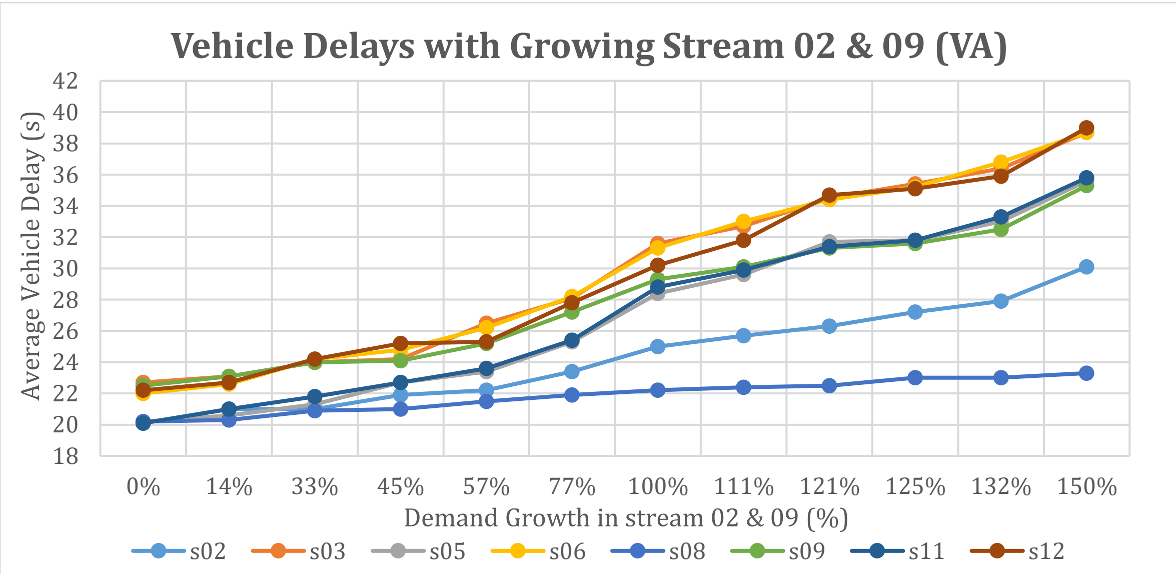


Figure 27-1. Vehicle delays with growing stream 02 and 09 (VA)

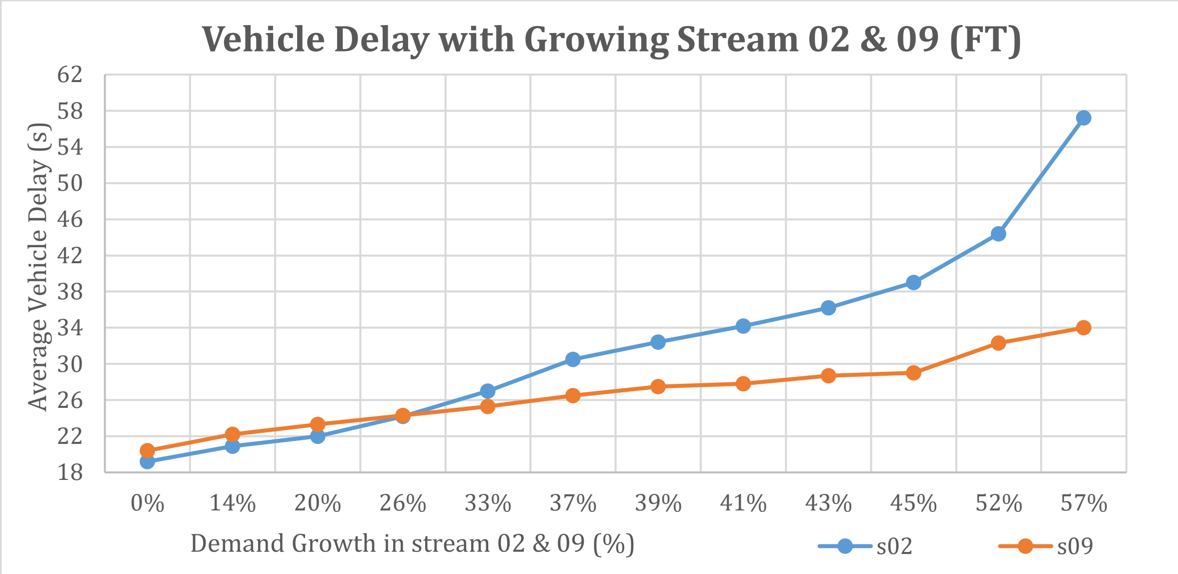


Figure 27-2. Vehicle delays with growing stream 02 and 09 (FT)

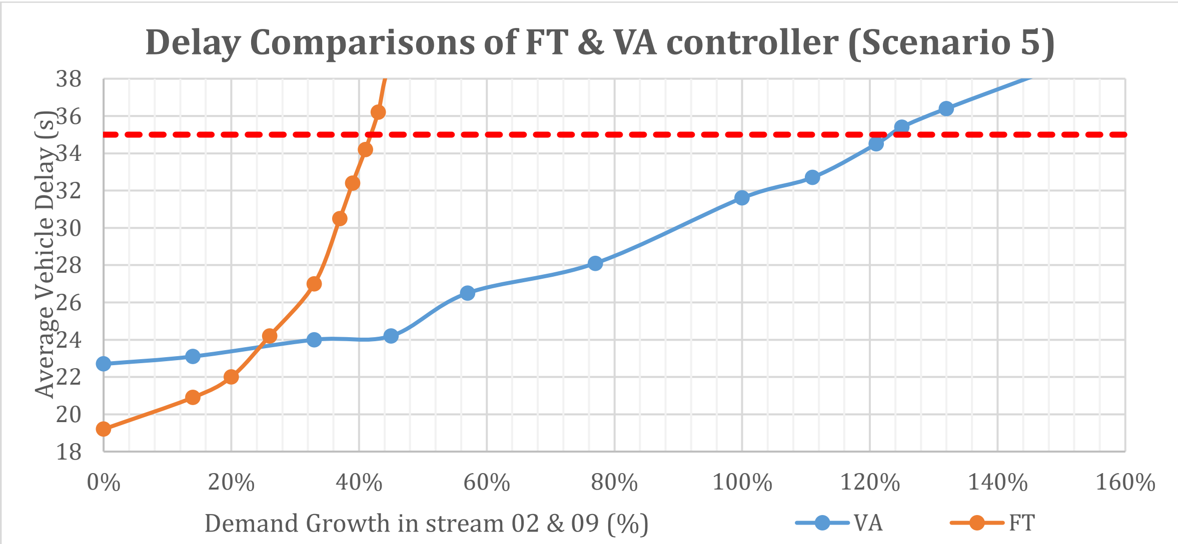


Figure 28. Delay comparisons between FT and VA controller (Scenario 5)

In scenario 5, 124% of demand growth (in stream 03) and 42% of demand growth (in stream 02) can be accommodated by vehicle-actuated controller and fixed-time controller respectively. An 82% of volume growth in stream 02 and 09 can be served by the extra capability in demand growth adaption with the control of vehicle-actuated signal.

6) Growth in two conflict streams (02, 11)

Table 11a. Results of scenario 6 with VA controller (Growth in stream 02 and stream 11)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
14%	540	20.8	200	23.0	475	20.7	200	23.4	475	20.5	200	23.0	540	20.6	200	23.2
20%	570	20.8	200	22.9	475	21.2	200	23.4	475	20.8	200	23.6	570	21.0	200	23.4
26%	600	21.3	200	23.5	475	21.3	200	23.9	475	21.1	200	23.5	600	21.3	200	24.2
50%	711	22.7	200	25.0	475	22.5	200	25.6	475	22.6	200	25.8	711	22.4	200	25.9
57%	745	23.1	200	25.9	475	23.3	200	26.8	475	23.3	200	25.9	745	23.2	200	25.6
71%	810	23.7	200	28.0	475	24.6	200	27.2	475	24.4	200	28.2	810	23.6	200	27.9
89%	900	25.7	200	31.0	475	26.3	200	31.4	475	27.1	200	30.0	900	25.6	200	31.3
102%	960	26.4	200	32.6	475	28.2	200	33.3	475	28.8	200	32.8	960	26.3	200	32.8
115%	1020	27.3	200	34.6	475	29.9	200	34.9	475	30.1	200	35.1	1020	27.2	200	34.7
121%	1050	27.8	200	35.1	475	30.2	200	35.6	475	30.5	200	36.1	1050	27.8	200	35.1
127%	1080	28.6	200	37.3	475	31.9	200	35.9	475	31.9	200	36.7	1080	28.3	200	35.8
153%	1200	32.5	200	41.1	475	34.3	200	41.1	475	34.1	200	41.0	1200	33.4	200	40.6

Table 11b. Results of scenario 6 with FT controller (Growth in stream 02 and stream 11)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
5%	500	20.1	200	20.9	475	18.6	200	20.9	475	18.2	200	20.5	500	20.7	200	21.0
14%	540	21.5	200	20.8	475	18.5	200	21.4	475	18.0	200	20.6	540	22.2	200	20.9
20%	570	23.7	200	20.2	475	18.3	200	21.3	475	18.0	200	21.1	570	23.3	200	21.1
23%	582	24.8	200	20.3	475	18.3	200	21.4	475	17.9	200	21.3	582	24.0	200	21.2
26%	600	26.8	200	20.5	475	18.5	200	21.7	475	18.3	200	21.0	600	26.2	200	21.2
33%	630	29.4	200	20.3	475	18.5	200	21.5	475	18.4	200	21.6	630	28.1	200	21.2
39%	660	34.5	200	20.2	475	18.7	200	21.1	475	18.5	200	21.2	660	33.0	200	20.7
40%	666	35.8	200	20.9	475	18.8	200	21.1	475	18.6	200	21.7	666	34.4	200	20.8
41%	672	37.4	200	20.8	475	18.7	200	21.1	475	18.3	200	21.4	672	36.1	200	21.0
50%	711	50.8	200	20.4	475	18.5	200	20.8	475	18.4	200	21.9	711	49.8	200	21.2
57%	745	71.6	200	20.4	475	18.3	200	20.8	475	18.6	200	20.8	745	77.5	200	20.9

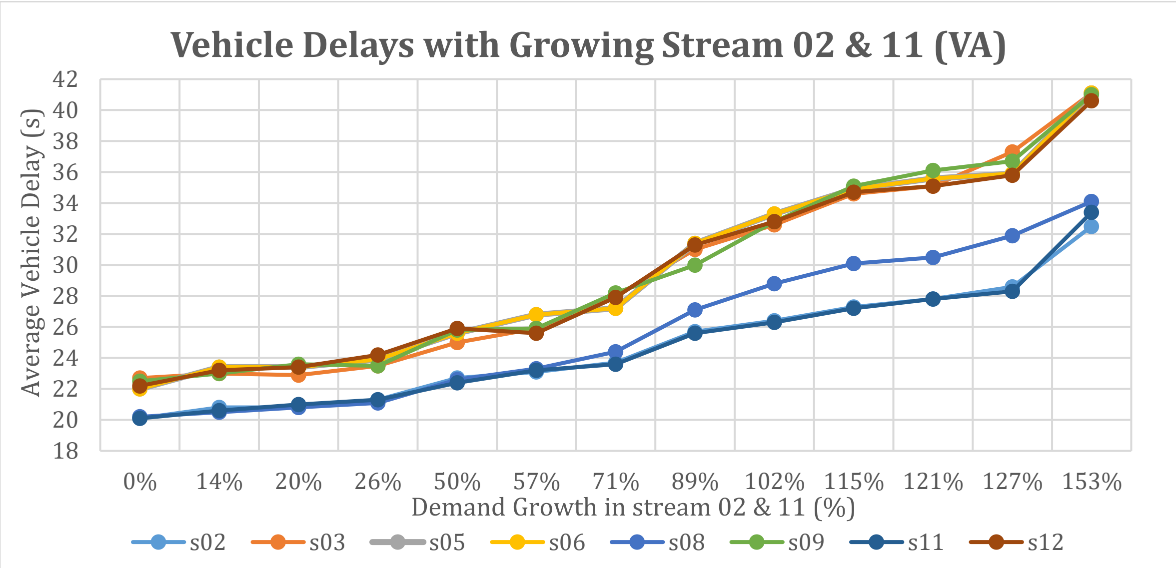


Figure 29-1. Vehicle delays with growing stream 02 and 11 (VA)

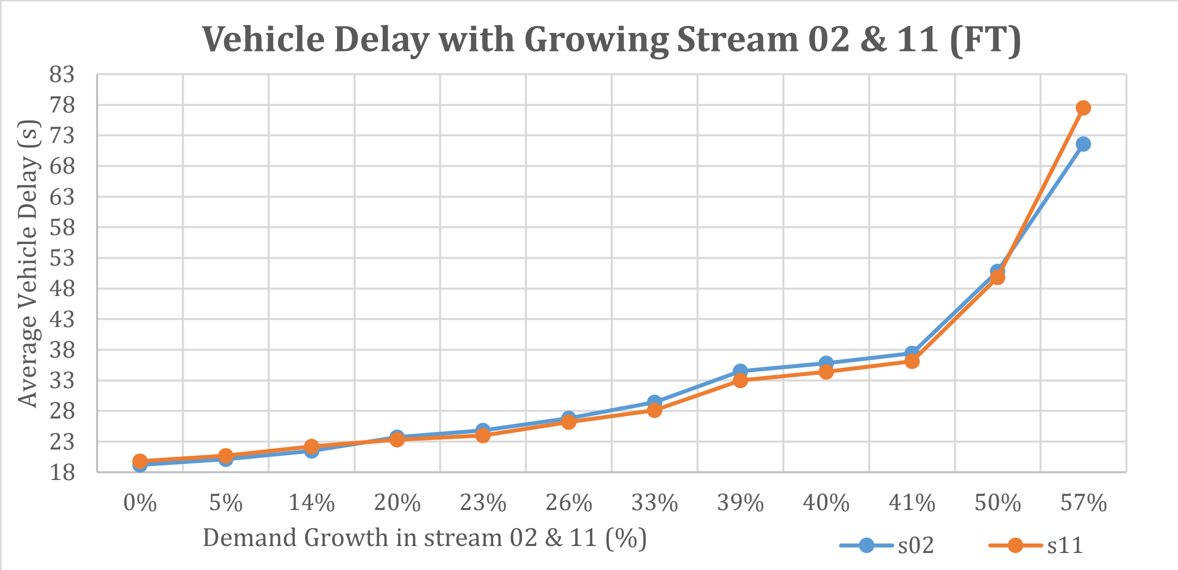


Figure 29-2. Vehicle delays with growing stream 02 and 11 (FT)

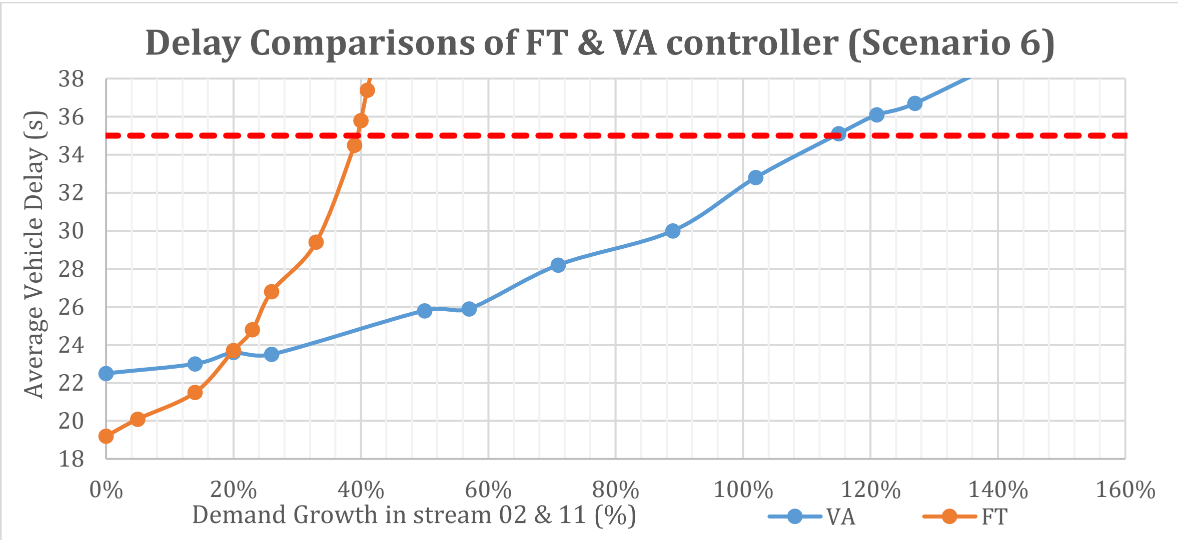


Figure 30. Delay comparisons between FT and VA controller (Scenario 6)

In scenario 6, 115% demand growth (in stream 09) and 40% demand growth (in stream 02) can be accommodated by the vehicle-actuated controller and the fixed-time controller respectively. A 75% of the demand growth in stream 02 and stream 11 can be served by the extra capacity provided by the vehicle-actuated controller.

7) Growth in two conflict streams (02, 12)

Table 12a. Results of scenario 7 with VA controller (Growth in stream 02 and stream 12)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
14%	540	20.6	200	23.1	475	20.7	200	23.1	475	20.4	200	23.6	475	20.7	227	22.9
26%	600	21.0	200	23.6	475	21.3	200	23.6	475	20.9	200	24.0	475	21.2	253	23.8
39%	660	21.6	200	24.0	475	21.9	200	23.8	475	21.8	200	24.3	475	21.6	278	24.3
50%	711	21.7	200	23.9	475	22.9	200	24.8	475	22.4	200	25.0	475	21.9	299	24.6
57%	748	22.4	200	25.7	475	23.3	200	26.1	475	22.4	200	25.7	475	22.5	315	25.2
71%	810	22.8	200	27.2	475	24.9	200	26.8	475	23.9	200	26.6	475	23.2	341	25.9
100%	950	24.1	200	30.2	475	28.1	200	30.2	475	26.4	200	30.4	475	26.2	400	28.1
111%	1000	25.0	200	32.3	475	29.3	200	32.2	475	27.7	200	31.6	475	27.1	421	29.4
121%	1050	25.4	200	33.6	475	31.2	200	34.1	475	28.6	200	33.2	475	27.9	442	30.6
127%	1080	26.2	200	34.2	475	31.9	200	34.7	475	29.8	200	34.6	475	29.1	455	31.7
129%	1086	26.6	200	35.3	475	32.5	200	34.9	475	30.0	200	35.3	475	28.8	457	31.8
132%	1100	26.6	200	35.3	475	32.5	200	34.7	475	30.2	200	34.7	475	29.6	463	32.0
150%	1188	29.0	200	38.0	475	35.9	200	38.7	475	32.5	200	37.8	475	31.3	500	34.6

Table 12b. Results of scenario 7 with FT controller (Growth in stream 02 and stream 12)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
14%	540	20.9	200	20.7	475	18.3	200	20.7	475	18.1	200	20.1	475	19.9	227	23.1
20%	570	22.0	200	20.4	475	18.3	200	20.9	475	17.9	200	20.5	475	19.9	240	24.3
26%	600	24.2	200	20.1	475	18.3	200	21.2	475	18.1	200	20.5	475	19.9	253	26.0
33%	630	27.0	200	19.8	475	18.3	200	21.1	475	18.0	200	20.8	475	19.9	265	27.5
39%	660	32.4	200	20.5	475	18.3	200	21.3	475	17.9	200	20.9	475	19.9	278	28.2
40%	666	32.9	200	20.1	475	18.3	200	20.8	475	18.0	200	20.6	475	19.8	280	29.1
41%	672	33.7	200	20.5	475	18.5	200	20.7	475	18.1	200	20.6	475	19.8	283	30.9
43%	678	35.8	200	20.4	475	18.2	200	21.0	475	18.1	200	20.7	475	19.9	285	30.4
44%	684	37.7	200	20.1	475	18.3	200	21.2	475	18.0	200	20.8	475	19.7	288	32.2
45%	690	39.0	200	20.1	475	18.2	200	21.3	475	17.9	200	20.9	475	19.8	291	32.9
50%	711	46.1	200	19.9	475	18.3	200	20.9	475	18.1	200	20.7	475	19.9	299	33.8

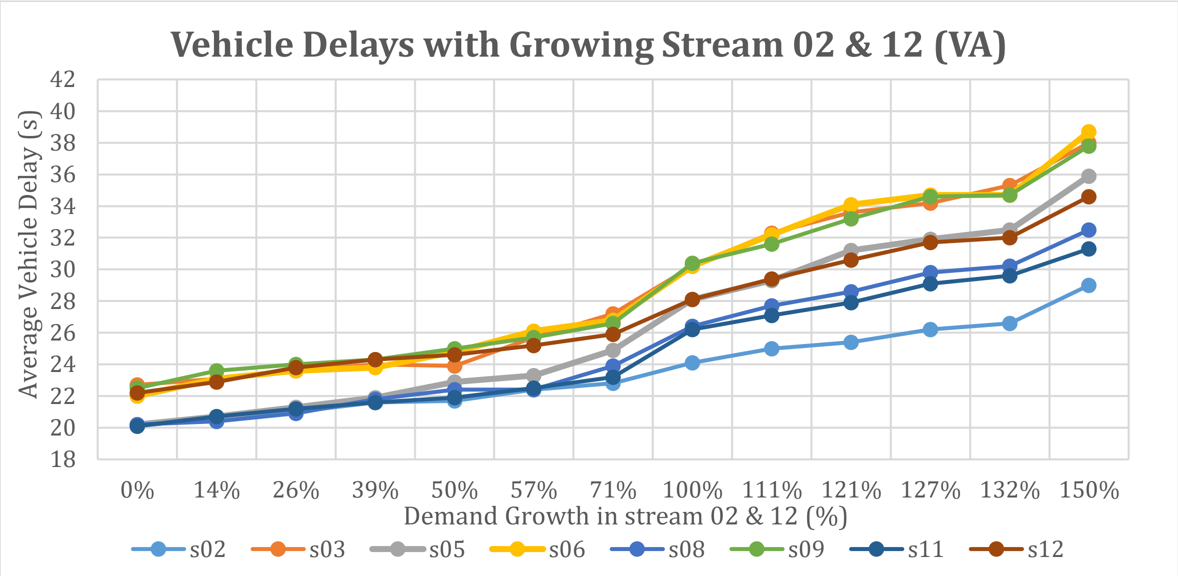


Figure 31-1. Vehicle delays with growing stream 02 and 12 (VA)

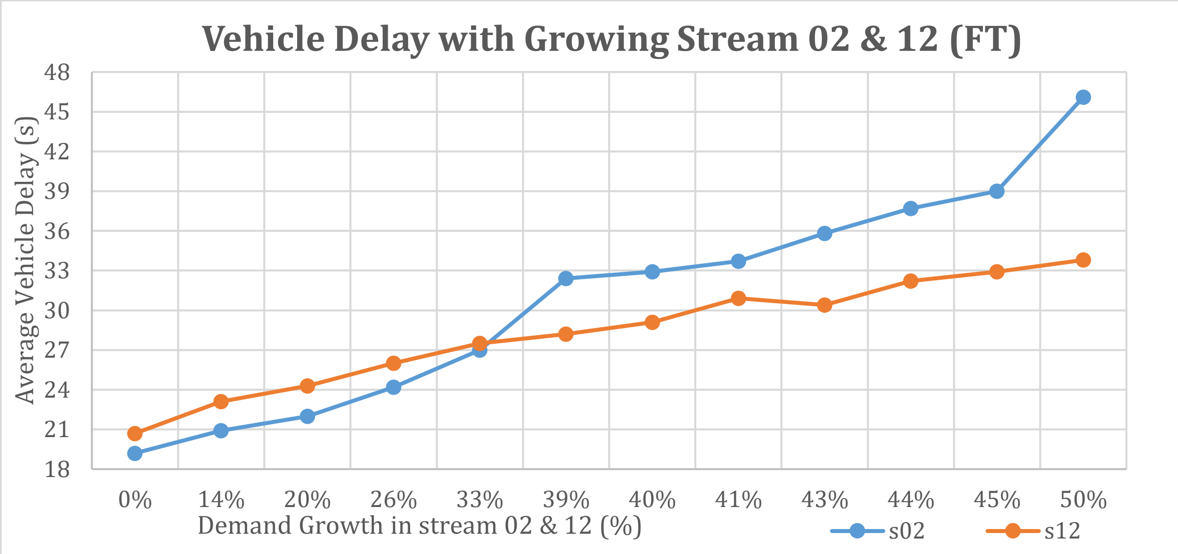


Figure 31-2. Vehicle delays with growing stream 02 and 12 (FT)

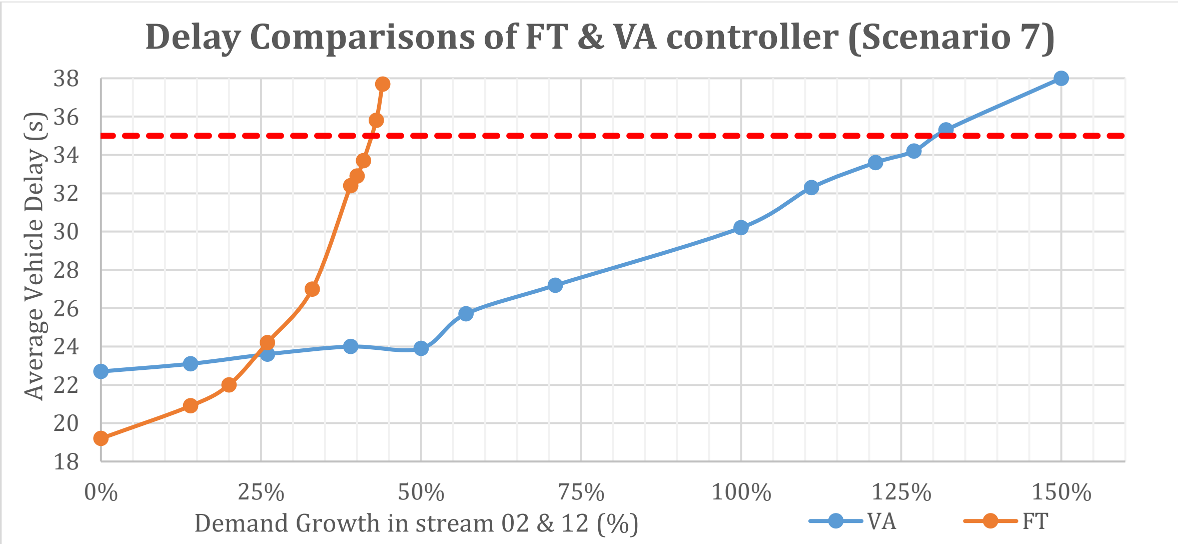


Figure 32. Delay comparisons between FT and VA controller (Scenario 7)

In scenario 7, 128% demand growth (in stream 03) and 42% demand growth (in stream 02) can be accommodated by vehicle-actuated controller and fixed-time controller respectively. An 86% of the demand growth in stream 02 and stream 12 can be served by the extra capacity of the vehicle-actuated controller.

8) Growth in two conflict streams (03, 06)

Table 13a. Results of scenario 8 with VA controller (Growth in stream 03 and stream 06)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
35%	475	20.6	270	23.6	475	20.5	270	23.8	475	21.3	200	23.7	475	21.5	200	23.9
50%	475	21.6	300	24.7	475	21.4	300	24.2	475	22.5	200	24.7	475	22.7	200	24.0
80%	475	22.9	360	26.0	475	23.0	360	26.4	475	24.4	200	27.0	475	24.5	200	27.5
140%	475	28.4	480	29.2	475	28.1	480	30.7	475	31.1	200	33.2	475	31.2	200	32.8
155%	475	29.6	510	32.2	475	29.8	510	32.9	475	33.2	200	36.4	475	33.1	200	35.3
170%	475	32.0	540	33.6	475	31.6	540	36.1	475	35.0	200	38.8	475	35.3	200	37.8
185%	475	33.2	570	37.0	475	33.2	570	39.4	475	37.5	200	39.9	475	37.4	200	40.0
200%	475	34.6	600	40.6	475	34.8	600	44.5	475	39.7	200	42.9	475	40.3	200	42.2
230%	475	36.3	660	46.7	475	36.5	660	50.8	475	40.2	200	43.6	475	41.0	200	42.9
260%	475	36.4	720	47.3	475	36.2	720	51.3	475	41.3	200	43.2	475	40.6	200	42.9
300%	475	36.5	800	47.6	475	35.8	800	51.4	475	41.1	200	43.8	475	40.6	200	44.1

Table 13b. Results of scenario 8 with FT controller (Growth in stream 03 and stream 06)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
5%	475	19.2	210	20.9	475	18.5	210	21.8	475	18.1	200	20.4	475	19.8	200	20.7
20%	475	19.0	240	23.0	475	18.3	240	23.7	475	18.1	200	20.4	475	19.8	200	20.7
35%	475	19.5	270	25.2	475	18.4	270	27.0	475	18.1	200	20.4	475	19.8	200	20.7
50%	475	19.5	300	29.0	475	18.4	300	31.5	475	18.1	200	20.4	475	19.8	200	20.7
56%	475	19.3	312	31.5	475	18.4	312	34.4	475	18.1	200	20.4	475	19.8	200	20.7
58%	475	19.3	315	32.3	475	18.4	315	34.5	475	18.1	200	20.4	475	19.8	200	20.7
59%	475	19.4	318	33.6	475	18.4	318	37.2	475	18.1	200	20.4	475	19.8	200	20.7
65%	475	19.4	330	36.5	475	18.4	330	39.7	475	18.1	200	20.4	475	19.8	200	20.7
80%	475	19.6	360	50.4	475	18.4	360	54.6	475	18.1	200	20.4	475	19.8	200	20.7
95%	475	19.7	390	73.6	475	18.4	390	79.7	475	18.1	200	20.4	475	19.8	200	20.7

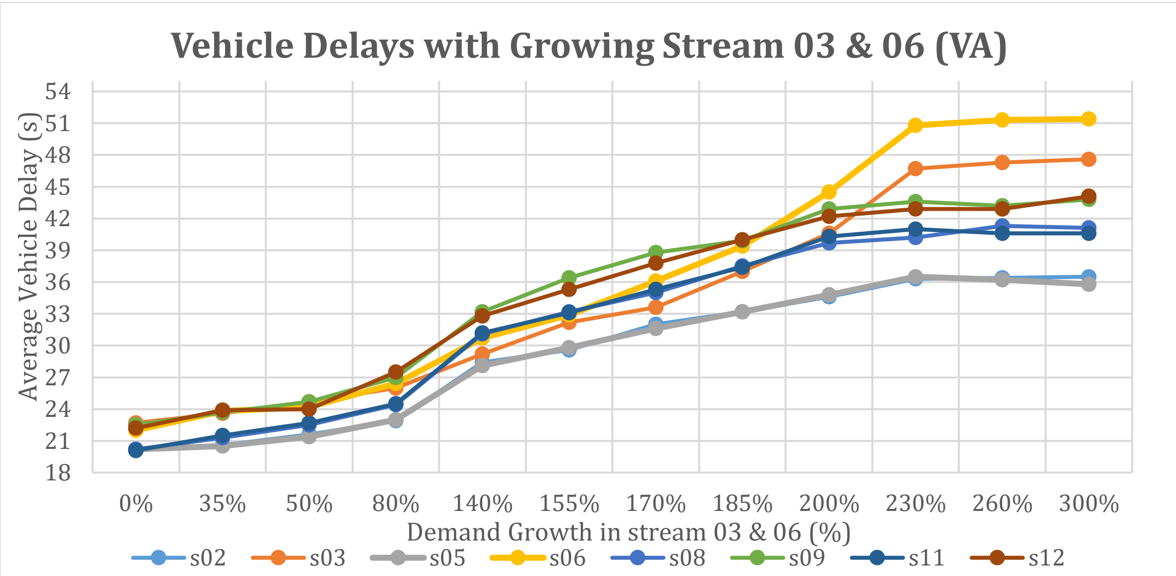


Figure 33-1. Vehicle delays with growing stream 03 and 06 (VA)

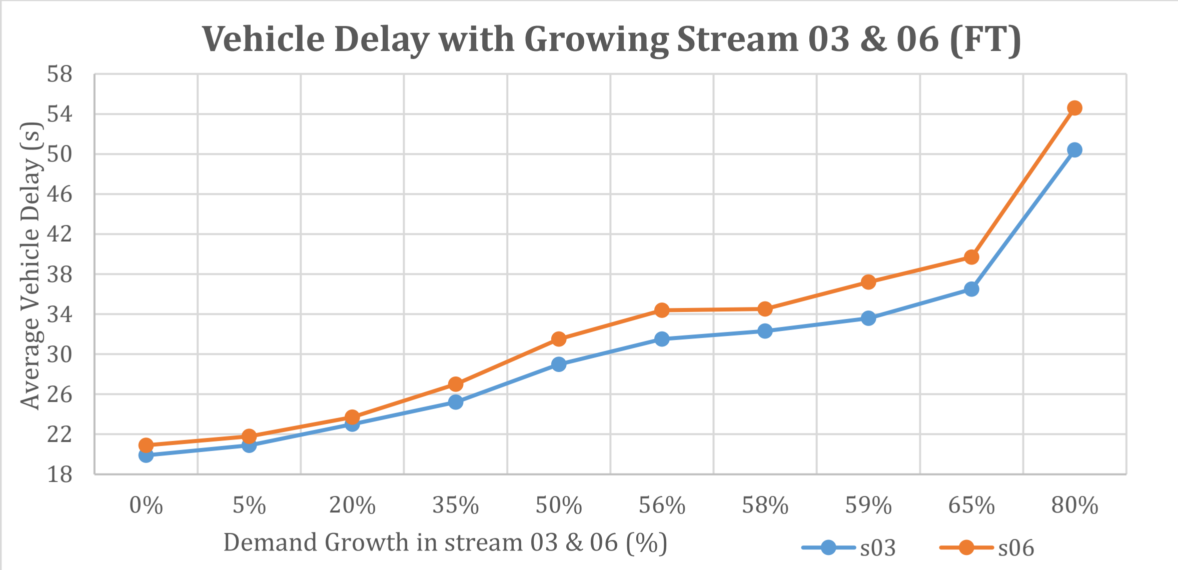


Figure 33-2. Vehicle delays with growing stream 03 and 06 (FT)

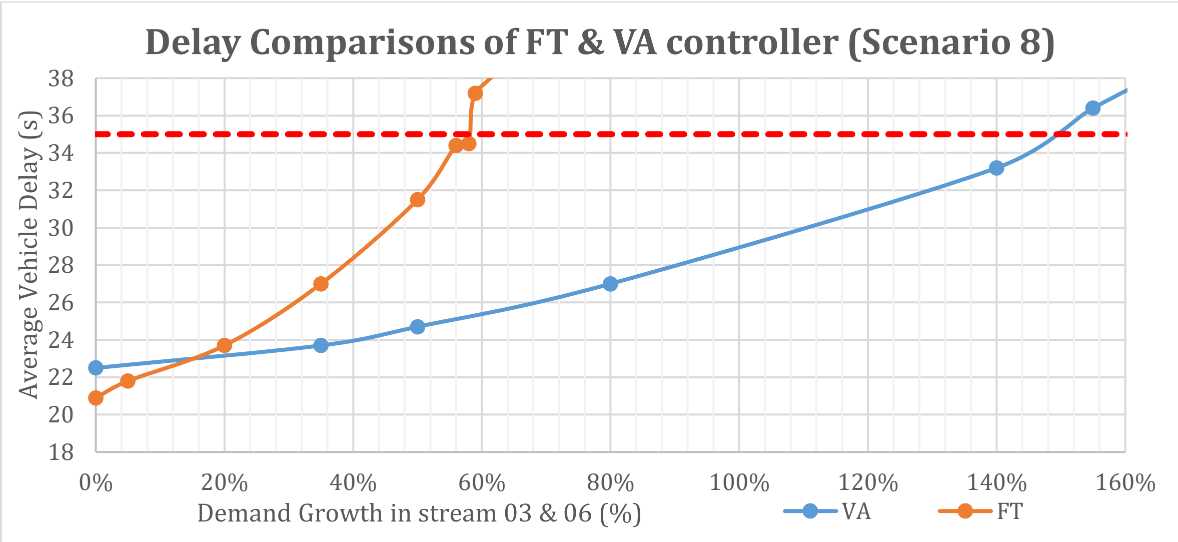


Figure 34. Delay comparisons between FT and VA controller (Scenario 8)

In scenario 8, a 155% of demand growth (in stream 12) and a 58% of demand growth (in stream 06) can be accommodated by the control of vehicle-actuated signal and fixed-time signal respectively. A 97% of the extra demand growth in stream 03 and in stream 06 can be served by the vehicle-actuated controller within the delay performance criterion of 35s.

9) Growth in two conflict streams (03, 12)

Table 14a. Results of scenario 9 with VA controller (Growth in stream 03 and stream 12)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	20.1	200	22.7	475	20.2	200	22.0	475	20.2	200	22.5	475	20.1	200	22.2
25%	475	20.8	250	23.5	475	20.8	200	23.4	475	20.8	200	23.3	475	20.8	250	23.5
50%	475	21.4	300	24.2	475	22.3	200	24.4	475	22.1	200	24.7	475	21.6	300	24.4
80%	475	23.0	360	25.6	475	24.2	200	27.2	475	24.6	200	25.9	475	22.8	360	25.6
95%	475	24.1	390	27.3	475	26.0	200	27.8	475	26.0	200	28.0	475	23.8	390	26.8
110%	475	24.9	420	28.2	475	26.8	200	29.9	475	27.3	200	29.4	475	25.2	420	27.9
140%	475	27.9	480	30.5	475	31.1	200	33.8	475	31.2	200	33.9	475	28.1	480	29.9
146%	475	28.1	492	31.1	475	32.1	200	34.3	475	31.5	200	34.5	475	28.6	492	31.4
149%	475	29.2	498	31.2	475	32.7	200	35.3	475	32.5	200	34.8	475	28.6	498	31.3
155%	475	29.8	510	32.2	475	33.1	200	36.1	475	33.0	200	36.9	475	29.8	510	33.0
170%	475	31.7	540	33.8	475	35.8	200	38.3	475	35.6	200	38.2	475	32.1	540	35.3
185%	475	33.6	570	36.7	475	37.3	200	40.4	475	37.6	200	40.7	475	33.7	570	39.4

Table 14b. Results of scenario 9 with FT controller (Growth in stream 03 and stream 12)

Growth	Stream 02		Stream 03		Stream 05		Stream 06		Stream 08		Stream 09		Stream 11		Stream 12	
	In	D	In	D	In	D	In	D	In	D	In	D	In	D	In	D
0%	475	19.2	200	19.9	475	18.4	200	20.9	475	18.1	200	20.4	475	19.8	200	20.7
5%	475	19.4	210	21.1	475	18.5	200	21.4	475	18.2	200	21.0	475	20.2	210	21.7
20%	475	19.4	240	23.8	475	18.6	200	21.5	475	18.1	200	20.8	475	20.1	240	25.3
29%	475	19.6	258	26.3	475	18.3	200	21.7	475	18.0	200	21.3	475	20.0	258	27.6
35%	475	19.8	270	26.4	475	18.3	200	21.4	475	17.9	200	21.1	475	20.0	270	29.0
40%	475	19.5	280	28.4	475	18.4	200	21.3	475	18.1	200	21.1	475	20.1	280	30.3
45%	475	19.4	290	29.2	475	18.5	200	21.3	475	18.0	200	20.9	475	20.1	290	33.7
47%	475	19.6	293	29.9	475	18.4	200	21.2	475	17.9	200	21.2	475	20.2	293	34.5
48%	475	19.7	295	29.9	475	18.3	200	21.4	475	17.9	200	21.0	475	20.2	295	35.6
50%	475	19.7	300	31.1	475	18.4	200	21.4	475	18.0	200	21.1	475	20.1	300	35.8
65%	475	19.6	330	39.5	475	18.4	200	21.4	475	18.3	200	20.9	475	20.1	330	53.3

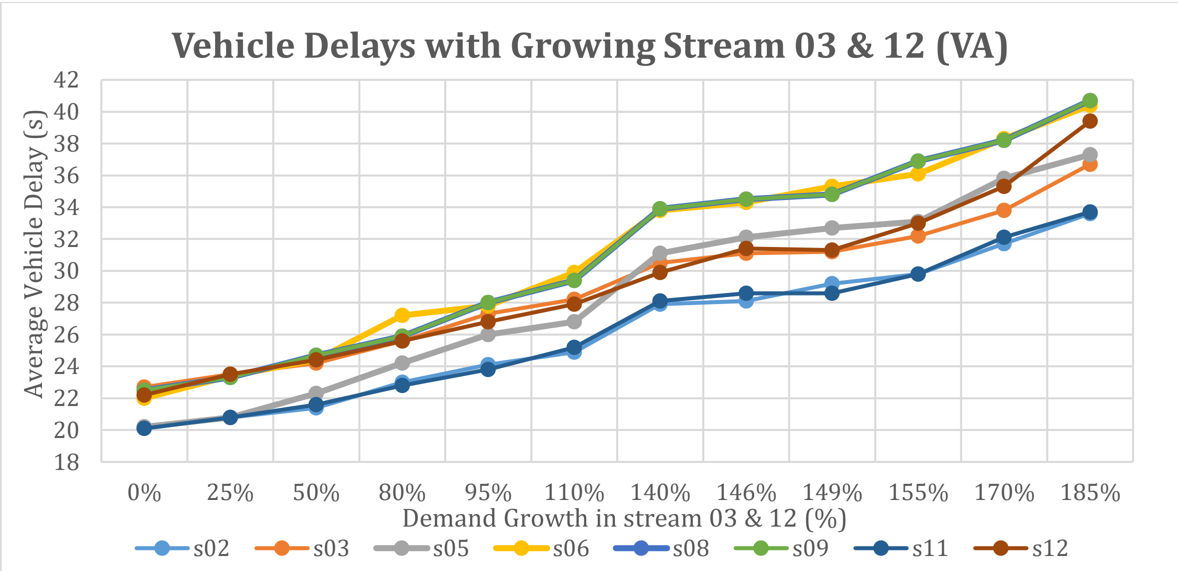


Figure 35-1. Vehicle delays with growing stream 03 and 12 (VA)

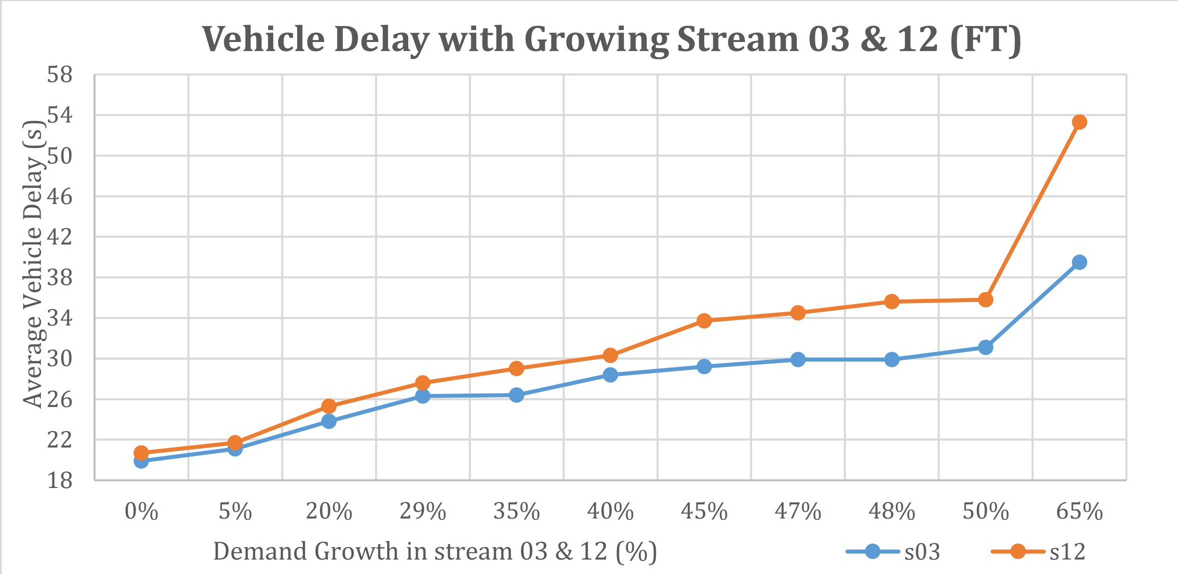


Figure 35-2. Vehicle delays with growing stream 03 and 12 (FT)

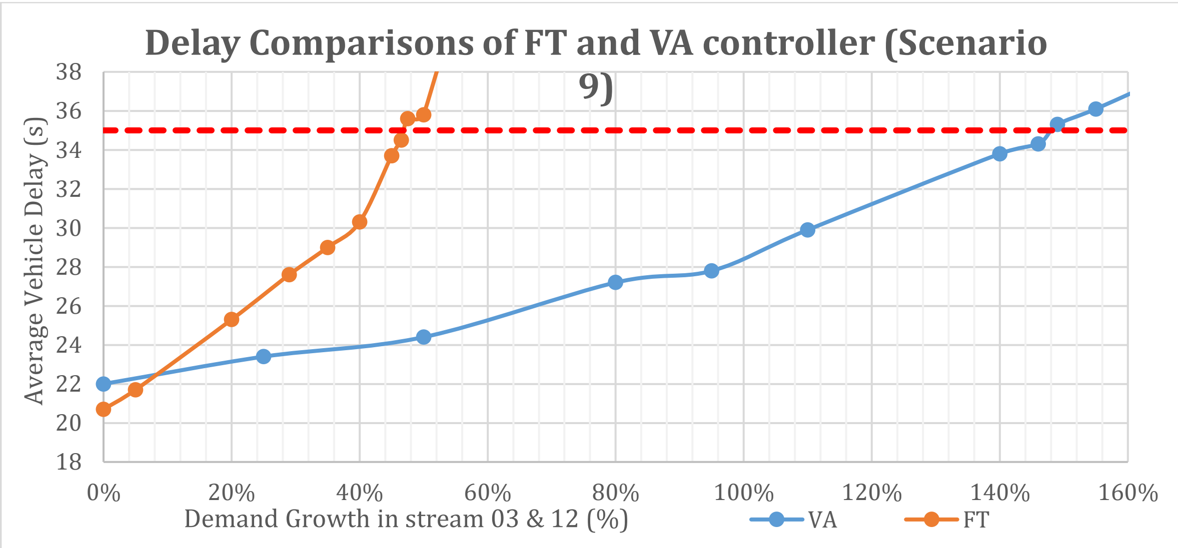


Figure 36. Delay comparisons between FT and VA controller (Scenario 9)

In scenario 9, a 149% of demand growth (in stream 06) and a 47% of demand growth (in stream 12) can be accommodated by vehicle-actuated controller and fixed-time controller respectively. A 102% of the extra volume growth in stream 03 and in stream 12 can be served by the vehicle-actuated controller within the delay criterion threshold of 35s.

6.7 Result Analysis and Scenario Comparison

According to the simulation results and delay curves illustrated in section 6.6, findings based on the comparison between the fixed-time control and the vehicle-actuated control in different scenarios are summarized in this section.

All four left-turning delay curves are generally higher than the four through-going curves with the control of vehicle-actuated signal, and the delay results for left-turning streams are more consistent excluding the left-turning stream(s) which is (are) with growing demand.

In comparison, the delay results for through-going traffic show larger variations among four streams. Though the delays in all streams show a growing pattern, the average vehicle delay is better maintained for the volume-increasing stream(s) since more green times are allocated to that or those growing stream(s). The delay increment in those streams is always slower than other streams, which makes it the lowest among all 8 streams. At the same time, the delay growth of parallel streams at the same control stage is also well maintained with more allocated green times.

Table 15. VA and FT performance (capability in accommodating demand increase) comparisons

Scenario	VA	VA(VRIGEN)	VA(Net)	FT	Comparison	Comparison(Net)
1(02)	279%	57%	222%	43%	236%	179%
2(03)	315%	50%	265%	61%	254%	204%
3(02,05)	115%	50%	65%	44%	71%	21%
4(02,06)	132%	57%	75%	42%	90%	33%
5(02,09)	124%	56%	68%	42%	82%	26%
6(02,11)	115%	56%	59%	40%	75%	19%
7(02,12)	128%	56%	72%	42%	86%	30%
8(03,06)	155%	50%	105%	58%	97%	47%
9(03,12)	149%	50%	99%	47%	102%	52%

Shown in Table 15 is the performance comparison among 9 scenarios, instead of direct comparisons for the control delays under the control of vehicle-actuated and fixed-time signals described in section 6.6, a concept of net comparison is come up and illustrated.

As a matter of fact, the cycle length of the predefined vehicle-actuated controller varies between the minimum cycle time (36.0 seconds) and the maximum cycle time (120 seconds), a certain degree of demand increase will not change the optimality of the existing control structure. The *critical demand sets* for each scenario were tested in VRIGEN, with which the calculated optimal signal timings or the control structure by VRIGEN are updated. These critical demand sets are around 50% to 57% higher (shown in the third column of Table 15) than the initial demand, which indicates that the vehicle-actuated controller itself has the capability in adapting a certain degree of demand fluctuations.

In comparison, any volume increase of the initial demand will cause the timing or structure change of the fixed-time control determined by VRIGEN. As the signal control structure can only be improved when the demand exceeds the critical demand sets for VA controllers, the net capability of the demand growth adaptation is a better performance indicator for a fair comparison between fixed-time and vehicle-actuated controllers.

Therefore, the net extra capability provided by VA controller in maintaining the average vehicle delay at the intersection within the LoS is compared with FT controller, summarized in the last column of Table 15. For the volume increase in a single stream, more demand increase in left-turning streams can be accommodated both under the VA control and the FT control compared with the demand growth in through-going streams. For the comparison between two types of signal controllers, the vehicle-actuated controller can serve 179% and 204% more demand increase than fixed-time controller in scenario 1 and scenario 2, respectively.

In summary, around 20% of the net demand increase can be adapted under the scenarios (scenario 3 and scenario 6) of two though-going stream combinations, and approximately 50% of demand increase can be served under the scenarios (scenario 8 and scenario 9) of two left-turning stream combinations. With regarding to the net capability comparisons in scenarios of one though-going stream and one left-turning stream combinations, the capabilities in demand adaptions are in between other two types of stream combinations, in which around 26% to 33% higher demand adaptions are observed under the control vehicle-actuated signals.

6.8 Discussions of the Results

Though the delay curves for two types of the controllers in each scenario are composed of 10 to 12 demand growth inputs, the data points are still too few to represent the actual shape of the delay curves. Due to the time constraint for the simulation records, the delay results were not recorded with a fixed small growing interval (for instance every 1%, 2% or 5% of the demand growth), the critical demand input above which the delay will exceed 35s is determined following a binary (logarithmic) search algorithm. Therefore, data collection points are mainly concentrated near the critical demand input, and only the delay curves for the controller comparison shown in section 6.6 represent their actual shapes, while these data points are not uniformly distributed.

Besides, due to the random nature of vehicle arrival patterns and the existence of standard deviations for each set of delay record, fluctuations of the delay results can not be avoided, which can cause the deviation of actual value when determining the critical demand and the critical demand growth rate. As a matter of fact, instead of determining a unique critical demand, there should be a demand range within which the vehicle delays will have significant chance to exceed 35s. Therefore, a lower value of the demand compared to the critical demand determined in this research could also lead to an average vehicle delay larger than 35s, in other word, the critical demand or demand growth should be lower than the values determined in this research and therefore the results are overestimated.

Furthermore, the critical demand growth and the comparison is calculated on a basis of the predefined initial demand, the critical growth rate determined for each controller will be changed when with other initial demands. Though the concept of net capability in demand growth adaptation is introduced for the VA controller, which can help to compensate the impacts on the critical demand growth rate introduced by different initial demand inputs to some extent. It caused the unfair comparison between FT and VA controllers, which totally neglects the self-ability in demand adaptation of VA controllers.

The delay results can also differ when the geometric design of the intersection is changed. The conclusions in this research are only valid for the intersection designed in this research, any change of the intersection can lead to different delay performance results for both types of the signal controllers.

Moreover, there are still difference among each stream. Even the intersection is designed as generic as possible, which is ideally an absolute symmetry intersection with four identical approaches, all through-going streams and all left-turning streams should be totally same. However, as evaluated in section 6.1, the saturation flow rates of each stream are different, thus the four through-going streams and four left-turning streams could not be simply represented by one stream 02 and one stream 03, which also lead to inaccurate delay results and the performance comparisons.

7. Conclusions & Recommendations

Conclusions of the research are given in this section. In sub section 7.1, answers for both sub research questions and the main research question are summarized. In sections 7.2 & 7.3, insights for practical applications of our research findings and recommendations for further research are illustrated.

7.1 Answers for the Research Questions

1) *What is the level of performance for the signal control at an intersection? And at which level improvements on signal timings or structures should be considered?*
   There are many aspects of the performance to justify the signal control at an intersection, indicators of the performance include the throughput flow, vehicle delays, number of stops, total travel times, fuel consumption and emissions **(Bonneson et al., 2011)**. The average vehicle delay is taken as the performance indicator in this research, which is defined as the difference between the actual travel time experienced by passing the intersection with and without signal control **(NCHRP, 2001)**. A criterion of 35s per vehicle is taken as the performance threshold according to the level-of-service rank defined in HCM manual, above which the signal controllers should be improved.

2) *How fixed-time and vehicle-actuated controllers should be designed to accommodate the given demand optimally?*
   The basic principle of signal timing improvement is to allocate more green times to dominating streams. And the control structure in terms of phase sequence & splitting can also get changed when there is significant volume difference in the streams at the current same control stage. For the fixed-time controller, any demand change can reduce its optimality, the increasing demand can be accommodated by a longer cycle time for the fixed-time control.

   For the vehicle-actuated signal controller, its optimality can be kept with a certain degree of demand variation without changing existing control structure and timings. When the demand keeps growing, longer minimum green times for corresponding streams become necessary to accommodate the increasing demand inputs. When the demand is too high and saturated, the vehicle-actuated controller will work in a "fixed-time" way since the maximum green splits have been reached. When the maximum cycle length could not be extended anymore due to local regulations, then the operation performance at the intersection could not be improved by simply redesigning the controller. Advanced technologies and physical measures might be necessary to fit such kind of high demand.

3) *How much of the demand growth can be served by the fixed-time controller before it needs to be redesigned?*
   Depending on the volume of which stream or streams showing with a growing trend, the volume growth that can be served by fixed-time controllers within the delay criterion of 35s varies from 40% to 61% for the case modified in this research.

4) *How much of the demand growth can be served by the vehicle-actuated controller before it needs to be redesigned?*
   Similar with the fixed-time controller, the capability in demand growth adaptation of vehicle-actuated controllers also varies according to the stream and stream combinations. In conditions

with one-stream growth, the vehicle-actuated control can accommodate a 279% of volume growth in through-going stream and a 315% of volume growth in left-turning stream. In conditions of two through-going streams, a 115% of demand growth can be served. In conditions of two left-turning streams, a 149% to 155% of demand growth can be served. And in conditions of one through-going stream and one left-turning stream, the capability in volume growth adaptation varies from 124% to 132%.

- **Main Research Question:**

  "How frequent should the fixed-time controllers and vehicle-actuated controllers be redesigned to maintain a comparable level of performance at an isolated intersection?"
  A comparable performance is defined in this research as the average control delay per vehicle no higher than 35 seconds, and the redesign frequency for the signal controllers is varied under different growing patterns.

  Take a demand condition modified in scenario 7 (the flows in stream 02 and stream 12 increase together) as an example, the vehicle-actuated controller can serve at least 30% (net comparison) more demand growth than fixed-time controller. When an average annual volume growth rate "α" is predictable to be used, then a "$30\%/\alpha$" times higher redesign frequency is required for the fixed-time controllers to maintain a comparable performance level with vehicle-actuated controllers.

7.2 Insights for Practical Applications

As illustrated in previous section, the traffic signal controllers should be periodically checked and updated when necessary if substantial demand increase were observed. The operators should have an inventory of these equipment, together with their capabilities to be identified. Based on recorded capabilities and current demand profiles at the intersection, the necessity for the improvement of the signal controllers can be evaluated.

This research quantified the capabilities in demand growth adaptation of both fixed-time and vehicle-actuated controllers under different demand situations. The results of this research can help to guide the planning for the signal maintenance strategy, which should be preventive before the operation performance of the signal becomes unacceptable.

It can be expected that responsive controllers generally work better (in terms of average vehicle delay) in any demand condition compared with the fixed-time controllers at an isolated intersection, while it is not realistic to replace all fixed-time controllers to responsive strategies. According to a survey conducted among road managers in the Netherlands by **Wilson, Middleham and Vermeul (2000)**, 86% of the total 5250 signalized intersection controllers are installed as vehicle-actuated, and 60% of the respondents with a fixed-time controller argued that lack of time and money is the key reason for not upgrading to fully responsive control programs. With this quantified comparison between fixed-time controllers and vehicle-actuated controllers, the delay savings introduced by the implementation of vehicle-actuated controllers, the initial investment costs and the ongoing maintenance costs can be compared via a cost-benefit analysis, by which local authorities can decide whether it is profitable to upgrade the fixed-time controllers.

7.3 Recommendations for Further Research

With regarding to possible step-further research on the redesigning frequency comparison between different types of signal controllers, there are some aspects and scopes of the research that can be improved and further investigated.

Right-turning streams. The right-turning streams are totally neglected in this research to simplify the complexity of demand settings. In reality, the downstream saturation flow rate will be lowered due to the merging effects introduced by the right-turning streams, therefore, the consideration of right-turning streams can help to improve the quality of the simulation results.

Pedestrians and cyclists. The pedestrian and cyclist are also not considered in this research, and their interactions with motorists are not accommodated by the signal controllers. In practice, the communications between pedestrians, cyclists and motorists at a signalized intersection are usually involved of priority problems, which is aimed at improving the equality among different user types. The introduction of pedestrians and cyclists traffic can help to provide more realistic insights for the comparison of signal performance.

Stream combinations. Nine scenarios in total are classified according to the combination of conflict streams, and only conditions of two maximum involved streams are investigated in this research. As a matter of fact, there are much more possibilities with combinations of more involved streams, which could also be investigated for more complex demand conditions in further studies. Moreover, stream combinations but with different growing speeds between streams can also be an interesting scope in following researches.

Acknowledgements

This work is well supported and supervised by Dr. Andreas Hegyi and Dr. Maria Salomons from the Department of Transport & Planning in the Delft University of Technology. We together witnessed this research grows from an immature idea to a completed scientific study, I learned and benefited quite a lot by getting myself dedicated into this interesting topic, which trained me with critical thinking and scientific skills to better solve problems. Many thanks for their patience and great help throughout the whole research process.

Reference

AASHTO, 2001. “A Policy on Geometric Design of Highways and Streets (Green Book). 6th Edition” American Association of State Highway and Transportation Officials, Washington, D.C. Retrieved from: https://nacto.org/wp-content/uploads/2015/04/GDHS-6_ToC.pdf

Akcelik, R. (1994). Estimation of Green Times and Cycle Time for Vehicle-Actuated Signals. Transportation Research Record No. 1457. Part 2, Traffic Flow and Capacity. pp 63-72. Retrieved from: https://trid.trb.org/view/425344

Bell, M. G. H., & Brookes, D. (1993). Discrete time-adaptive traffic signal control: The calculation of expected delays and stops. Transportation Research Part C: Emerging Technologies, 1(1), 43–55. Retrieved from: https://doi.org/10.1016/0968-090x(93)90019-c

Bonneson, J., Sunkari, S. R., Pratt, M., Songchitruksa, P., & Texas Transportation Institute (2011). Traffic Signal Operations Handbook: Second Edition. Retrieved from: https://rosap.ntl.bts.gov/view/dot/23504

Bullen, A. (1989). Effects of Actuated Signal Settings and Detector Placement on Vehicle Delay. *Transportation Research Record No. 1244*, Traffic and Grade Crossing Control Devices. Retrieved from: http://onlinepubs.trb.org/Onlinepubs/trr/1989/1244/1244-005.pdf

Calvert, S., Minderhoud, M., Taale, H., Wilmink, I. & Knoop, V. (2016). Traffic Assignment and Simulation Models. TrafficQuest Report. Retrieved from: https://doi.org/10.13140/RG.2.1.3784.45

Clark, J. (2007). Assessing the Sensibility of Signal Timing Split Optimization in Addressing Congestion. ITE (Institute of Transportation Engineers) Journal, 78(8), 24–29. Retrieved from: http://citeseerx.ist.psu.edu/viewdoc/download?doi=10.1.1.639.8514&rep=rep1&type=pdf

Cronje, W. B. (1983). Optimization Model for Isolated Signalized Traffic Intersections. Transportation Research Record, 80–83(905). Retrieved from: https://trid.trb.org/view/195824

Day, C. M., T. M. Brennan, H. Premachandra, A. M. Hainen, S. M. Remias, J. R. Sturdevant, G. Richards, J. S. Wasson, and D. M. Bullock. (2010). Quantifying benefits of traffic signal retiming. Publication FHWA/IN/JTRP-2010/22. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana. Retrieved from https://doi.org/10.5703/1288284314250

Dion, F., & Hellinga, B. (2002). A rule-based real-time traffic responsive signal control system with transit priority: application to an isolated intersection. Transportation Research Part B: Methodological, 36(4), 325–343. Retrieved from: https://doi.org/10.1016/S0191-2615(01)00006-6

Dion, F., Rakha, H., & Kang, Y.-S. (2004). Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections. Transportation Research Part B: Methodological, 38(2), 99–122. Retrieved from: https://doi.org/10.1016/S0191-2615(03)00003-1

Fitzpatrick, K., Wooldridge, M. D., & Blaschke, J. D. (2005). Urban Intersection Design Guide: Volume 1 - Guideline 6. Performing Organization Code Project 0-4365 13. Type of Report and Period Covered Product 12. Sponsoring Agency Name Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration. Project Title: Urban Intersection Design Guidance. Retrieved from: https://static.tti.tamu.edu/tti.tamu.edu/documents/0-4365-P2.pdf

Furth, P., & Muller, T. (1999). TRAFCOD: A Method for Stream-Based Control of Actuated Traffic Signals. Presented at 78th Annual meeting of the Transportation Research Board, National Research Council, Washington, D.C., USA, 1999. Retrieved from: https://www.semanticscholar.org/paper/TRAFCOD-%3A-A-Method-for-Stream-Based-Control-of-Furth/8d88bba09ab9c29be73448f65d51174b3d71ddd8#citing-papers

Gao, Y. (2008). Calibration and Comparison of the VISSIM and INTEGRATION Microscopic Traffic Simulation Models. *Master Thesis*. *Virginia Polytechnic Institute and State University*. Retrieved from: https://doi.org/etd-09102008-135101

Gershenson, C. (2004). Self-Organizing Traffic Lights. Retrieved from: http://arxiv.org/abs/nlin/0411066

Heydecker, B. G. (1996). A decomposition approach for signal optimization in road networks. Transportation Research Part B: Methodological, 30(2), 99–114. https://doi.org/10.1016/0191-2615(95)00025-9

Highway Capacity Manual. Transportation Research Board (TRB), National Research Council, Washington, D.C., 2000. Retrieved from: https://sjnavarro.files.wordpress.com/2008/08/highway_capacital_manual.pdf

Hu, H., Gao, Y., & Yang, X. (2010). Multi-objective Optimization Method of Fixed-Time Signal Control of Isolated Intersections. 2010 International Conference on Computational and Information Sciences. Retrieved from: https://doi.org/10.1109/iccis.2010.316

ITE. 1994. Determining Vehicle Signal Change and Clearance Intervals. Publication IR-073, Institute of Transportation Engineers (ITE), Technical Council Task Force 4TF-1, Washington, D.C., 1994. Retrieved from: https://trove.nla.gov.au/work/13855527

Li, Y., Yu, L., Tao, S., & Chen, K. (2013). Multi-Objective Optimization of Traffic Signal Timing for Oversaturated Intersection. Mathematical Problems in Engineering 1–9. https://doi.org/10.1155/2013/182643

McGee, H. W., & Institute of Transportation Engineers. (2003). Making Intersections Safer: A Toolbox of Engineering Countermeasures to Reduce Red-Light Running Red-Light Running: An Informational Report. Federal Highway Administration. Publication IR-115. Washington, D.C. *ISBN: 0-935403-76-0*. Retrieved from https://safety.fhwa.dot.gov/intersection/conventional/signalized/rlr/rlr_toolbox/rlrbook.pdf

Muller, T. H. J., & de Leeuw, M. (2006). New Method to Design Traffic Control Programs. Transportation Research Record: Journal of the Transportation Research Board, 1978(1), 68–75. Retrieved from: https://doi.org/10.1177/0361198106197800110

Muller, T. H. J., Dijker, T., & Furth, P. G. (2004). Red Clearance Intervals: Theory and Practice. Transportation Research Record, 1867(1), 132–143. Retrieved from: https://doi.org/10.3141/1867-16

NACTO, 2015. Fixed versus Actuated Signalization. National Association of City Transportation Officials. Retrieved from: https://nacto.org/publication/urban-street-design-guide/intersection-design-elements/traffic-signals/fixed-vs-actuated-signalization/

NCHRP, 2001. Evaluating Intersection Improvements: An Engineering Study Guide. National Cooperative Highway Research Program (NCHRP) Report 457. *ISBN 0-309-06705-7*, Transportation Research Board, 2001. Retrieved from: http://onlinepubs.trb.org/onlinepubs/nchrp/esg/esg.pdf

Papageorgiou, M., Ben-Akiva, M., Bottom, J., Bovy, P. H. L., Hoogendoorn, S. P., Hounsell, N. B., … McDonald, M. (2007). Chapter 11 ITS and Traffic Management. Handbooks in Operations Research and Management Science, 14, 715–774. Retrieved from: https://doi.org/10.1016/S0927-0507(06)14011-6

Papageorgiou, M. (2004). Overview of Road Traffic Control Strategies. IFAC Proceedings Volumes, 37(19), 29–40. Retrieved from: https://doi.org/10.1016/s1474-6670(17)30657-2

TAMU Mobility. (2011). Traffic Management – System Efficiency: Signal Operation & Management. Texas A&M Transportation Institute, Texas. Retrieved from: https://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Signal-Operations-and-Management-4-Pg.pdf

Taale, H. (2002). Comparing methods to optimize vehicle actuated signal control. Eleventh International Conference on Road Transport Information and Control, 2002, 114–119. https://doi.org/10.1049/cp:20020216

Tang, K., & Nakamura, H. (2011). Investigating operational benefits of group-based signal control in Japan through a stochastic approach. International Journal of Intelligent Transportation Systems Research, 9(2), 71–81. Retrieved from: https://doi.org/10.1007/s13177-011-0028-y

Webster, F.V. (1958). Traffic Signal Settings. Technical report. Road Research Technical Paper No. 39. Department of Scientific and Industrial Research, Road Research Laboratory, HMSO, London, U.K. Retrieved from: https://trid.trb.org/view/113579

Wilson, A., & de Groot, R. (2014). Manual Traffic Light Regulations 2014 (In Dutch: Handboek verkeerslichtenregelingen). CROW: The National Knowledge Platform for Infrastructure, Traffic, Transport and Public Space. ISBN: 978 90 6628 643 6. Retrieved from: https://www.crow.nl/publicaties/handboek-verkeerslichtenregelingen-2014

Wilson, A., Middelham, F. and Vermeul, J. (2000). "Traffic Control in Urban Areas: A Survey among Road Managers", Proceedings of the 10th International Conference on Road Transport Information and Control, IEE conference publication 472, London, pp. 172-176. Retrieved from: https://digital-library.theiet.org/content/conferences/10.1049/cp_20000127

Wong, S. C., Wong, W. T., Leung, C. M., & Tong, C. O. (2002). Group-based optimization of a time-dependent TRANSYT traffic model for area traffic control. Transportation Research Part B: Methodological, 36(4), 291–312. Retrieved from: https://doi.org/10.1016/s0191-2615(01)00004-2

Salomons, A. M. (2008). Optimising cycle times of controlled intersections with VRIGen. In Colloquium Vervoersplanologisch Speurwerk (CVS) Congress. https://www.cvs-congres.nl/cvspdfdocs/cvs08_14.pdf

Sunkari, S. (2004). The Benefits of Retiming Traffic Signals. Institute of Transportation Engineers Journal, 74(4). Retrieved from: https://trid.trb.org/view/700327

Yulianto, B., Setiono (2012). Traffic Signal Controller for Mixed Traffic Conditions. IOSR Journal of Mechanical and Civil Engineering (IOSRJMCE) ISSN: 2278-1684 Volume 4, Issue 1 (Nov-Dec. 2012), pp. 18-26. Retrieved from: http://iosrjournals.org/iosr-jmce/papers/vol4-issue1/D0411826.pdf?id=2540

Zakariya, A. Y., & Rabia, S. I. (2016). Estimating the minimum delay optimal cycle length based on a time-dependent delay formula. Alexandria Engineering Journal, 55(3), 2509–2514. Retrieved from: https://doi.org/10.1016/j.aej.2016.07.029

Zhang, L., Yin, Y., & Lou, Y. (2010). Robust Signal Timing for Arterials under Day-to-Day Demand Variations. Transportation Research Record: Journal of the Transportation Research Board, 2192(1), 156–166. Retrieved from: https://doi.org/10.3141/2192-15

Zhou, Z., & Cai, M. (2014). Intersection signal control multi-objective optimization based on genetic algorithm. Journal of Traffic and Transportation Engineering (English Edition), 1(2), 153–158. Retrieved from: https://doi.org/10.1016/S2095-7564(15)30100-8

2030 Committee. (2011). Traffic Management System Efficiency: Signal Operation & Management. The Transportation Action Program. Texas A&M Transportation Institute. Retrieved from https://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Signal-Operations-and-Management-4-Pg.pdf

Appendix 1. Green Times Allocation for Base Demand

Table appdendix-1. Green times for base demand (475 veh/h and 200 veh/h) with vehicle-actuated control

Cycles	02	03	05	06	08	09	11	12
1	6.00	6.00	6.00	6.00	10.80	6.00	6.00	6.00
2	8.80	7.00	8.60	7.80	10.80	7.00	6.00	8.60
3	15.80	6.80	7.20	8.60	10.40	8.60	8.00	6.00
4	8.60	7.00	8.60	8.60	11.40	6.60	10.60	8.40
5	9.00	8.60	11.40	7.00	10.60	6.00	6.80	7.20
6	13.00	6.00	8.60	6.00	8.80	7.00	12.20	7.00
7	8.80	8.60	6.80	8.60	8.80	7.00	12.00	6.00
8	7.00	10.20	8.60	8.60	8.20	11.80	7.60	7.80
9	6.00	7.00	7.40	6.00	7.00	7.60	10.40	10.00
10	7.20	8.60	6.80	8.40	7.00	6.00	6.80	6.80
11	11.40	8.40	8.40	6.80	8.60	10.80	8.60	6.80
12	8.60	8.60	10.40	6.80	10.00	9.40	10.80	8.60
13	6.80	6.00	10.60	6.00	8.00	6.00	8.40	6.00
14	6.00	6.60	6.60	6.00	8.80	7.00	7.20	7.00
15	10.80	11.20	15.00	14.00	10.20	8.60	12.00	7.00
16	8.60	10.40	9.60	8.60	9.40	10.40	8.60	8.60
17	8.60	9.00	6.80	8.60	8.60	8.60	8.60	8.60
18	10.40	6.00	9.00	6.00	13.40	9.20	12.00	12.60
19	8.40	6.00	9.40	9.00	8.40	6.80	8.60	10.20
20	7.80	8.60	12.60	6.80	8.60	8.60	6.00	7.00
21	13.00	8.60	8.40	8.60	10.20	7.20	9.60	6.40
22	6.80	9.40	10.20	7.20	7.60	11.80	8.80	8.60
23	8.60	6.00	6.80	6.00	8.00	7.00	7.00	6.00
24	8.40	10.00	7.00	6.00	8.60	8.40	8.40	6.00
25	8.60	8.40	9.00	6.00	8.40	8.80	6.80	6.00
26	8.20	8.60	9.80	6.00	7.20	7.00	12.80	10.20
27	6.80	6.80	6.00	8.60	9.80	9.80	7.00	6.80
28	12.40	7.80	11.20	7.60	6.60	9.00	6.80	6.00
29	11.60	10.20	6.20	8.40	8.60	6.00	8.40	6.80
30	6.00	6.80	7.00	6.00	8.60	6.00	8.60	6.00
31	7.00	6.00	8.60	9.00	8.40	6.00	6.00	8.60
32	7.00	9.00	12.80	6.00	12.20	6.60	15.80	9.80
33	8.40	6.60	10.00	8.40	8.20	7.00	10.00	7.80
34	8.60	8.60	8.60	6.00	10.80	6.80	8.60	6.80
35	13.80	7.00	8.60	9.00	13.00	8.20	11.60	12.00
36	11.00	8.60	7.20	8.60	7.60	8.60	9.80	6.80
37	11.20	8.60	9.60	7.40	13.00	8.60	6.80	8.60
38	8.60	10.80	10.00	8.60	8.60	8.00	12.60	8.60
39	11.40	6.00	9.20	8.60	6.00	6.40	8.60	6.80
40	8.60	7.00	13.40	11.60	11.60	12.00	7.00	10.00
41	7.40	8.60	7.80	10.20	7.60	7.00	10.40	8.60
42	7.00	6.00	8.40	8.20	10.00	6.00	7.20	6.00

43	7.40	8.60	7.80	8.60	10.60	7.60	7.80	6.80
44	10.40	14.00	8.60	7.40	7.00	6.00	11.60	11.80
45	8.60	8.60	12.60	12.00	8.60	10.00	9.60	8.40
46	14.80	7.60	9.60	10.20	9.60	8.60	12.40	8.60
47	12.80	9.80	18.80	13.00	17.80	12.40	15.80	7.00
48	10.60	7.00	6.60	6.00	15.00	6.00	11.60	8.40
49	11.60	11.60	6.00	6.00	8.60	6.00	8.60	8.80
50	8.60	8.60	6.60	8.60	10.20	8.80	8.80	7.00
51	8.60	6.00	8.60	8.60	8.40	8.60	7.40	8.60
52	11.20	13.20	11.60	11.20	9.40	11.00	8.60	6.00
53	10.00	6.00	10.00	8.60	8.40	6.00	8.60	7.00
54	11.80	6.00	10.40	11.20	11.40	8.40	7.00	8.40
55	11.80	6.60	11.60	11.80	14.80	12.00	8.60	8.80
56	11.60	12.20	8.60	6.00	8.60	7.80	11.60	11.00
57	6.80	6.00	9.00	10.40	9.80	9.60	6.00	7.00
58	7.00	6.00	10.80	6.00	7.80	6.00	8.60	6.00
59	10.40	8.40	9.20	6.60	8.60	6.60	12.20	12.00
60	16.20	7.20	7.80	9.80	8.60	6.00	6.00	9.20
61	8.60	6.00	11.60	6.00	9.80	6.00	11.00	8.60
62	10.80	6.00	15.80	10.20	7.60	8.20	12.80	6.00
63	8.60	7.20	14.20	8.60	12.80	6.80	11.40	10.80
64	8.40	6.80	11.60	10.40	6.60	6.80	8.60	6.60
65	15.00	9.20	9.40	8.80	7.00	7.60	13.60	10.80
66	6.80	8.60	11.60	14.00	8.40	9.60	8.60	8.00
67	8.60	6.80	9.20	6.00	12.00	6.00	8.60	7.00
68	9.60	6.00	8.60	6.00	8.40	8.60	11.40	8.60
69	9.80	8.60	8.60	6.80	8.60	8.60	6.60	6.00
70	8.40	6.80	12.80	7.40	8.40	6.60	8.40	6.40
71	9.40	6.00	6.80	8.00	15.80	6.00	8.40	7.40
72	7.20	12.40	6.00	8.60	8.60	11.00	8.60	7.00
73	8.60	6.80	7.00	7.40	6.00	7.00	10.00	10.60
74	11.40	8.40	7.20	8.60	14.40	11.80	14.60	8.60
75	8.60	6.00	8.60	7.20	11.60	9.60	11.60	11.60
76	6.00	8.60	8.40	6.00	8.60	8.40	9.60	6.00
77	8.60	8.80	6.60	6.80	8.40	7.60	8.60	6.00
78	8.40	9.60	10.20	6.80	6.60	8.60	13.20	9.80
79	8.40	6.80	9.40	8.80	8.40	6.00	8.60	8.60
80	6.80	6.00	8.60	8.40	6.60	7.00	6.60	6.00
81	8.60	6.80	8.60	8.40	7.20	6.60	8.40	6.00
82	8.60	6.80	8.40	7.40	11.60	14.20	8.40	6.00
83	11.60	10.20	9.20	6.00	8.60	6.00	12.20	10.00
84	6.00	8.60	8.40	8.60	9.60	6.80	8.80	7.80
85	12.00	6.80	8.60	6.00	9.00	6.00	6.40	8.60
86	0.00	0.00	8.40	8.60	0.00	0.00	9.80	6.80
SUM	793.00	676.00	792.20	694.00	802.20	676.00	801.40	681.80

Table appendix-2. Green times for base demand (475 veh/h and 200 veh/h) with fixed-time control

Cycles	02	03	05	06	08	09	11	12
1	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
2	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
3	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
4	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
5	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
6	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
7	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
8	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
9	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
10	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
11	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
12	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
13	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
14	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
15	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
16	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
17	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
18	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
19	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
20	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
21	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
22	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
23	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
24	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
25	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
26	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
27	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
28	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
29	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
30	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
31	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
32	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
33	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
34	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
35	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
36	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
37	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
38	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
39	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
40	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
41	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
42	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
43	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
44	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80

45	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
46	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
47	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
48	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
49	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
50	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
51	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
52	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
53	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
54	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
55	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
56	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
57	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
58	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
59	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
60	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
61	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
62	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
63	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
64	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
65	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
66	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
67	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
68	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
69	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
70	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
71	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
72	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
73	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
74	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
75	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
76	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
77	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
78	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
79	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
80	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
81	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
82	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
83	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
84	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
85	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
86	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
87	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
88	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
89	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
90	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
91	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00

92	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
93	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
94	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
95	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
96	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
97	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
98	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
99	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
100	7.00	7.40	7.40	7.00	7.60	7.00	7.00	6.80
101	7.00	7.20	7.40	7.00	7.40	7.00	7.00	7.00
SUM	707.00	737.20	747.40	707.00	757.40	707.00	707.00	697.00

Table appendix-3. Total green times comparison for the base demand (475 veh/h and 200 veh/h)

	02	**03**	**05**	**06**	**08**	**09**	**11**	**12**	**Total**
SUM(VA)	793.00	676.00	792.20	694.00	802.20	676.00	801.40	681.80	5916.60
SUM(FT)	707.00	737.20	747.40	707.00	757.40	707.00	707.00	697.00	5767.00
Comparison	86.00	-61.20	44.80	-13.00	44.80	-31.00	94.40	-15.20	149.60

Table appendix-4. Total green times stage allocation for the base demand (475 veh/h and 200 veh/h)

	Stage 1		**Stage 2**		**Stage 3**		**Stage 4**		**Total**
stream	**02**	**08**	**06**	**12**	**05**	**11**	**03**	**09**	
SUM(VA)	802.20		694.00		801.40		676.00		2973.60
SUM(FT)	757.40		707.00		747.40		737.20		2949.00
Comparison	44.80		-13.00		54.00		-61.20		24.60

Note1: The critical streams in each control stage are marked in green in Table appendix-4.
Note2: The total simulation time per demand input is set as 70 minutes, including 10 minutes of warming up time used for filling vehicles into the intersection. The delay results summarized in Section 6 are only measured for the formal 1 hour of simulation time (from 600s to 4200s of the simulation run); The green time changes in Appendix 1 are recorded for the whole simulation time (from 0s to 4200s of the simulation run).