Traffic Signal Design with an Increasing Queue Discharge Rate

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Abstract: The traditional method of traffic signal design is based on the assumption of a constant queue discharge rate, which is also termed as saturation flow rate. However some field studies conducted in Taiwan and USA, contradicts with such assumption as they reported a marginally increasing trend observed for the queue discharge rate towards the back of queue. This paper reports on field study conducted in Auckland at six signalized intersections. The field observations confirm the findings of previous researchers that the queue discharge rate increases towards the back of queue. Two empirical models are proposed to reflect the observed queue discharge behavior. One of them is implemented to calculate the capacity and signal timings. The proposed model shows prospect to overcome the shortcomings of the traditional method for practical applications.

Keywords: *signalized intersections, capacity, signal timing*

1. INTRODUCTION

A convenient assumption of steady saturation flow rate over a saturated green time makes it considerably easy to calculate the lane group capacity which is equal to saturation flow rate multiplied by green to cycle time ratio (HCM, 2010). Several factors are identified in literature that can influence saturation flow rate; however the basic concept remains the same that when a signal changes to green, the flow across the stop line increases rapidly to saturation flow rate, which remains constant until either the queue is exhausted or the green period ends (Akcelik, 1981, HCM, 2010). Figure 1 illustrates this traditional method of traffic signal design for signalized intersections.

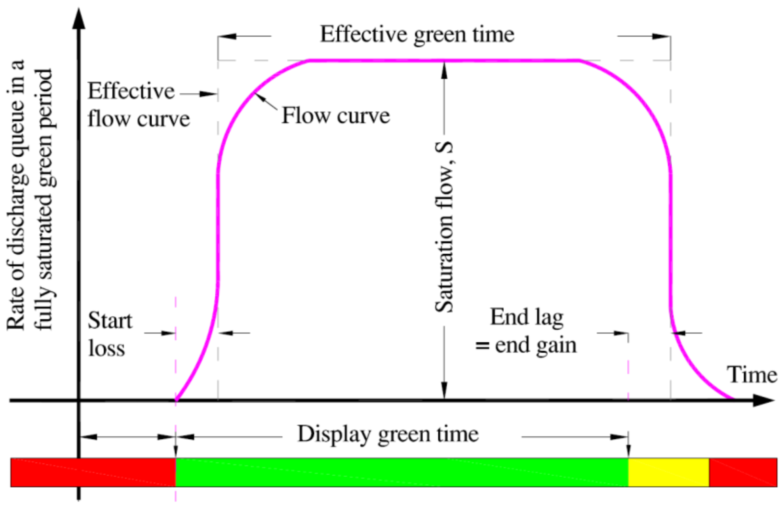


Figure 1 Traditional method of traffic signal design

A number of field studies conducted in the past have revealed significant variations in the queue discharge rate as shown in Figure 2. Jin et al. (1979) reported that the departure headway decreases sequentially with queue position. Teply (1983) conducted a study in Canada and noted that saturation flow rate depends not only on site-specific conditions but also on the duration of green period and type of community. Kimber and Hollis (1979) and Dion et al. (2004) proposed some modifications in the traditional method to incorporate the variable nature of queue discharge rate observed in the field. However, their methods cannot be implemented in practice as they are unable to cater for varying traffic conditions at signalized intersections.

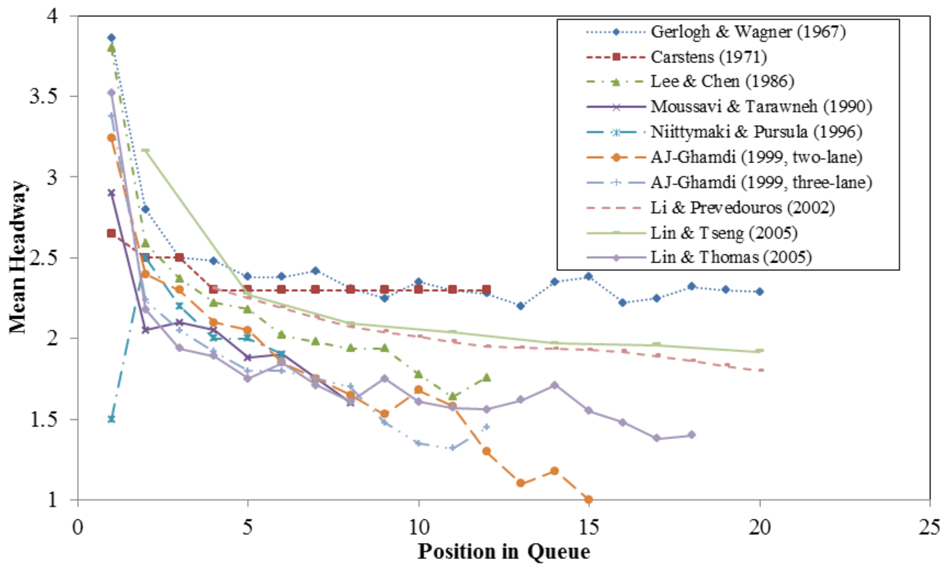


Figure 2 Departure headways with respect to queue position

A series of studies conducted in Taiwan and USA revealed a more consistent increase in queue discharge rate with green time (Li and Prevedouros, 2002, Lin et al., 2007, Lin and Thomas, 2005, Lin et al., 2004). Li and Prevedouros (2002) conducted a study in Hawaii that revealed a rather complex relationship between queue discharge rate and queue position. For through and left turning movements, the minimum headway could not be reached until the 9^{th} to 12^{th} vehicle in the queue crosses the stop line. Lin et al. (2004) reported that queue discharge rate often does not confirm to the notion of a quick rise to a steady state. They quantified the extent of errors by conducting statistical analysis on 38 urban lanes in Taiwan. They noted that queue discharge rates increase at an average rate of 24% for through movement and 16% for protected left turn movement when compared to those determined using HCM (2000) method. They also noted that there is a 40% chance that the lost time will differ from the correct value by 2 sec and 50% chance that the estimated capacity will deviate from the actual capacity by 5%, if the average lost time and saturation flow rate of a group of similar lanes are used as estimates for each lane in the study group. They conducted similar study at three signalized intersections in Long Island, New York (Lin et al., 2007), where all of them exhibit a general trend of gradual compression of headways as green time passes.

These strong evidences of variations in queue discharge rate discussed in the previous paragraphs motivated us the conduct similar study in Auckland to verify those observations and if that is the case then propose a new model based on the field observations to address the deficiencies of the traditional method. The rest of this paper is organized as follows. A review of related literatures is presented in the following section. Section 3 provides the details of data collected from six signalized intersection in Auckland and verifies an increasing trend for queue discharge rate that was observed from the field data. In section 4, we proposed new models to depict queue discharge behavior that was observed in the field. Section 5 presents the characteristics of the proposed models. In section 6, we implement the model for capacity and signal timing calculations. Finally, some concluding remarks are drawn in the last section.

2. LITERATURE REVIEW

2.1 Traditional Method

Saturation Flow Rate

A great deal of study conducted in the past to quantify the maximum queue discharge rate at signalized intersections. Clayton (1941) stated that the saturation flow rate at signalized intersections typically ranges from 1200 to 1800 pc/h/l. Greenshield et al. (1946) observed a queue discharge rate of 1714 pc/h/l after 6 vehicles passing through the stop line. Highway Capacity Manual (1950) suggested a value of 1500 pc/h/l for saturation flow rate, which was later revised to 1800 pc/h/l in HCM (1965). In 1997, this value was revised again to 1900 pc/h/l, which remained the same for all subsequent editions of the manual (HCM, 1998, HCM, 2000, HCM, 2010).

Capacity and Degree of Saturation

The capacity of a lane group is defined as the number of vehicles that can be discharged through the intersection per hour during the allocated green time. Mathematically, it can be expressed as:

$$c = s\frac{g}{C} \tag{1}$$

where, c is the capacity of a lane group, s is saturation flow rate, g is green time and C is cycle time.

The degree of saturation (DOS) also termed as v/c ratio; is an important variable in traffic signal design, which considered as an indicator of the adequacy of intersection capacity. It can be expressed as follows:

$$X = \frac{v}{c} = \frac{v.C}{s.g} \tag{2}$$

where v is arrival flow rate for a lane group and c is the capacity of the lane group. The DOS is calculated based on the assumption that the maximum discharge rate will prevail during the allocated green times. While this indicator has shown reasonable results in the highway sections where no complete stop and go scenario exist, Gilbert (1977) criticized the application of v/c ratio based on the full saturation flow rate for signalized intersection. In the latter versions of HCM, an effective green time is used instead of total green time to address this problem.

Cycle Time Formulation

Cycle time can influence capacity as well as operational characteristics of signalized intersections. Webster (1958) proposed the following cycle time formulation based on mathematical simulation of a range of traffic conditions to minimize delay at signalized intersection.

$$c_0 = \frac{1.5L + 5}{1 - Y} \tag{3}$$

where, c_0 denotes the optimal cycle time, L is total lost time within the cycle, and Y is the sum of critical phase flow ratios.

Later, Akcelik (1981) proposed a modified formulation to cater for different variables to be optimized in addition to delay:

$$c_0 = \frac{(1.4 + k)L + 6}{1 - Y} \tag{4}$$

where, k is a stop penalty parameter, which can have a value of 0.4, 0.2 or 0 to achieve minimum fuel consumption, minimum cost, or minimum delay respectively. In HCM (2010), the cycle time is calculated based on equalization of the volume to capacity ratio as follows:

$$c = \frac{L.X}{X - \sum y_i} \tag{5}$$

where X is the critical volume to capacity ratio.

2.2 Models based on Variable Queue Discharge Rate

Briggs (1977) was first to propose a deterministic model to incorporate an increasing trend in queue discharge rate. His model assumes a constant acceleration for queued vehicles and takes the following form;

$$h_n = T_r + \sqrt{\frac{2.d_i.n}{A}} - \sqrt{\frac{2.d_i.(n-1)}{A}} \tag{6}$$

For $d.n < d_{i_{max}}$, otherwise

$$h_n = T_r + \frac{d_i}{V_q} \tag{7}$$

where;

h_n	is the headway of the nth vehicle
n	is the queue position
V_q	is the desired speed of queued traffic
d_i	is the distance between vehicles in a stopped queue
$d_{i_{max}}$	is the distance travelled to reach speed V_q
T_r	is driver starting response time
A	is constant acceleration of queued vehicles

This model has two parts. The first part works when vehicles try to achieve a desired speed V_q at the beginning of green phase. While the second part consists of a state when vehicles have already achieved the desired speed and they are traveling at a constant speed. The first part gives a good approximation of queue discharge behavior at the beginning of green phase; however, the second part gives a constant headway. The values used in this model suggest that the minimum headway is achieved after 6 vehicles.

Bonneson (1992) reported that headway continues to decrease until 8^{th} or 9^{th} vehicle in the queue crosses the stop line after which a constant level is attained. He presented his model based on driver reaction time, vehicle speed and acceleration. Unlike Briggs' model, Bonneson's model was devised on a non-constant acceleration rate. The headway of n^{th} vehicle according to this model is;

$$h_n = \tau N_1 + T_r + \frac{d_i}{V_{max}} + \frac{V_{sl(n)} - V_{sl(n-1)}}{A_{max}} \tag{8}$$

where τ is the additional response time of the first queued vehicle, N_1 is 1 if n=1, or 0 if n>1, T_r the driver response time, di the distance between vehicles in a stopped queue, $V_{sl(n)}$ the stop line speed of the nth queued vehicles, V_{max} the maximum speed, A_{max} is the maximum acceleration. For the calculation of V_{sl}, an equation is presented.

$$V_{sl(n)} = V_{max}.(1 - e^{-n.k}) \tag{9}$$

where k is β/V_{max}, and β is an empirical calibration constant.

Long (2007) presented a driver behavior model of queue discharge rate in terms of discharge times of each vehicles in queue. His model considers the distances of the vehicles in queue to stop line, acceleration, and average start up lost time taking the following form:

$$td_n = w_n + ta_n = \tau + n.T_r + t_a(sa_n, \alpha, \beta) \tag{10}$$

where $td_n$is the average discharge time at the stop line of the nth vehicle in queue, w_n is the start-up waiting time for the nth vehicle in a queue, $ta_n$is the average acceleration time, τ is excess start-up time of the lead vehicle in a queue, T_r is uniform or average start-up response time of each driver, $sa_n$is average distance of acceleration of nth vehicle from start of motion to time ta_n, α is the average initial rate of acceleration and β is average rate of decrease in acceleration with increasing speed. It is difficult to implement this method for practical applications as equation (10) cannot be solved easily and needs root-search method to be applied.

Based on the discussions in this section and the previous section, it is clear that the concept of a constant saturation flow rate is not realistic. Although there are a few models proposed in the literatures to approximate variations in queue discharge rate, which were calibrated and validated against field observations data with a reasonable accuracy, they could not get much attention for practical applications mainly due to the complexity of the models and the number of assumptions made in those models. It would be wise to develop a model which is relatively simple with a minimum number of variables used to capture the observed queue discharge behavior. Such model shall be easy to implement in practice as well.

3. FIELD OBSERVATIONS IN AUCKLAND

Data Collection

The data were collected using video recording technique under normal driving conditions (with no unusual traffic and sunny weather conditions) from six signalized intersections in Auckland that include St. Lukes Road – New North Road intersection, Balmoral Road–Sandringham Road intersection, Dominion Road – Balmoral Road intersection, Manukau Road – Greenlane East Road intersection, Pah Road – Mount Albert Road intersection, and Great South Road – South Eastern Highway intersection. Figure 3 shows locations and layout of these intersections. Two hours of data were collected from each intersection except one during evening peak periods.

These intersections were selected for this study based on the selection criteria in the literature (Le et al., 2000). These criteria are helpful to locate relatively ideal intersections to minimize the need to adjust for prevailing conditions. The three main selection criteria include: presence of heavy traffic with at least one exclusive through lane; close to ideal geometric and roadway conditions (with 3.6m lane widths, a level approach grade, minimal or no pedestrian movements, no curb parking, no bus stop in the vicinity); and acceptable distance from adjacent intersection.

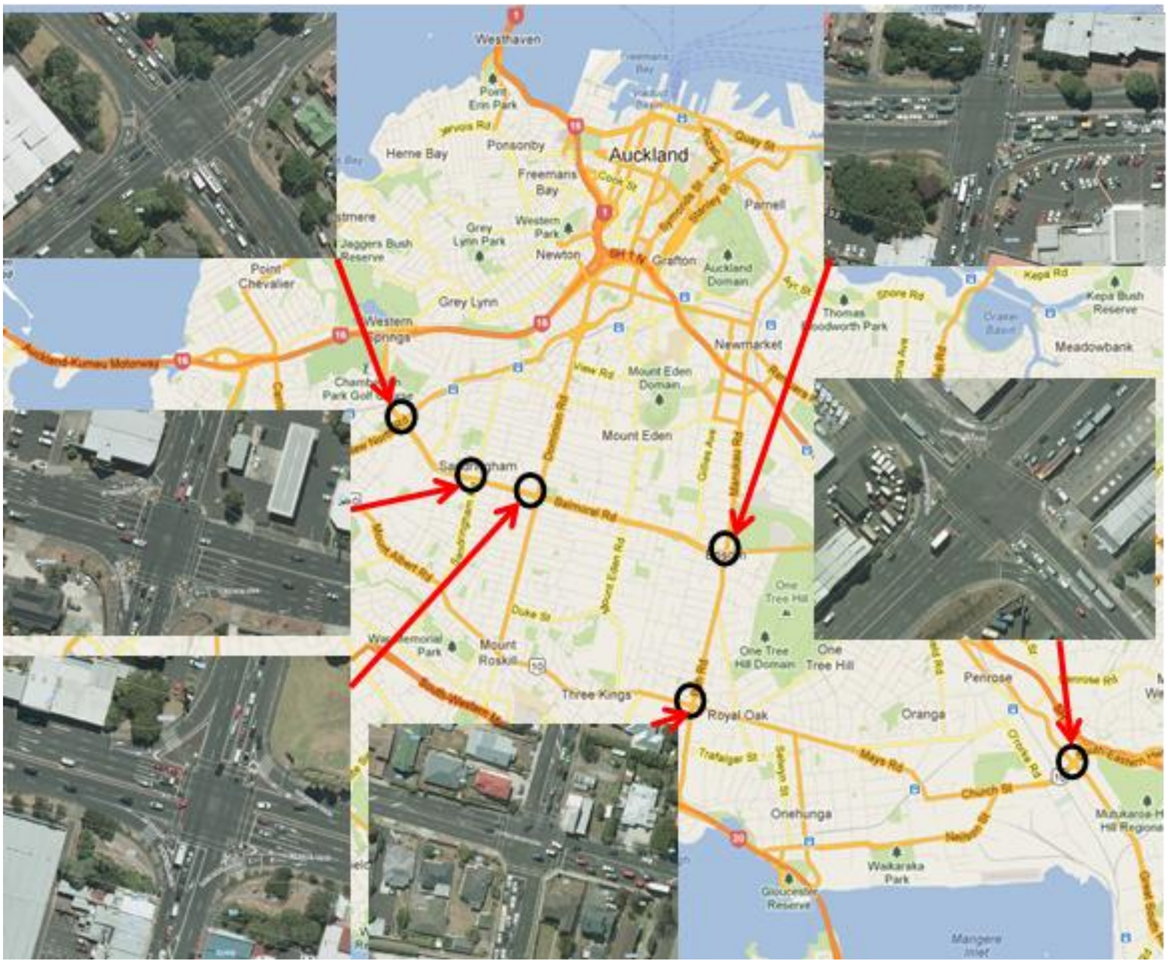


Figure 3 Selected Sites at Auckland

Data Analysis

Individual headway data is processed from the recorded video data for each vehicle in the queue. Table 1 presents the results of data analysis that shows some variations in the driver's reaction time from 1 to 1.2 seconds for the first five intersections that are located in the close proximity. While the last intersection that is Great South Road – South Eastern Highway intersection shows a different trend with an average reaction time of 2.05 seconds. The start-up lost time were recorded within the range of 1.18 to 3.08 seconds. The high start-up delay for St. Lukes Road – New North Road intersection could be due to downstream approach grade with a sharp curve resulting an increase in the start-up lost time.

Table 1 Relationship between queue discharge rate and green time

Intersection	Number of Phases	Reaction Time (Start of green to movement of first vehicle)		Start-up Lost Time			Queue Discharge Rate	
		R_t	Std	Avg	Med	Std	Equation	R^2
Balmoral - Sandringham Rd	58	1.02	0.79	1.51	1.78	1.72	0.0099 t + 0.3496	0.83
Balmoral - Dominion Rd	59	1.20	0.57	1.18	1.03	1.26	0.004 t + 0.4853	0.44
GT South Rd - SE Highway	22	2.05	0.48	2.64	2.34	1.22	0.0098 t + 0.4113	0.54
Manukau - Greelane East Road	58	1.15	0.45	1.24	0.94	1.12	0.0027 t + 0.4711	0.57
Pah - Mt Albert Road	60	1.18	0.41	1.73	2.14	1.41	0.0045 t + 0.4618	0.67
St Lukes - New North Road	54	1.00	0.47	3.08	3.16	0.96	0.0098 t + 0.3842	0.60

A large variation is noted for the stopping position of the first vehicles from the stop line. In calculation of start-up lost time, this distance has a significant influence as the stop line is taken as a reference line for headway calculations. Figure 4 presents queue discharge rate versus green time plots for each intersection. An increasing trend can be observed for all the intersections with a varying slope and R^2 values ranging from 0.44 to 0.83.

The results presented in this section are in line with the findings of previous researchers that queue discharge rate increases towards the back of queue. A possible explanation to increasing queue discharge behavior observed in the field would be the fact that the drivers located at the back of the queue are generally more likely to be stopped by change of signal to red than those located at the beginning of the queue. This consideration possibly influences driving behavior as a result those located at the back of queue try to maintain minimal headway with their preceding vehicle.

4. MODEL DEVELOPMENT

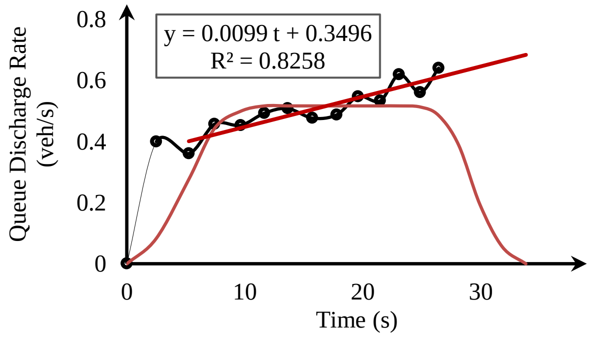
With an increasing trend, the maximum queue discharge rate will depend on the allocated green time. It has been reported in the literatures that queue discharge rate continue to increase till the 50 second of green time. Most of them reported that the queue discharge rate increases sharply at the beginning of green period, and then it becomes relatively flatter towards the end of queue. The field observations presented in this paper also confirms these patterns. Based on the field observations presented in the previous section, here we propose two types of models to approximate the observed queue discharge behavior namely linear and non-linear models.

Linear Model

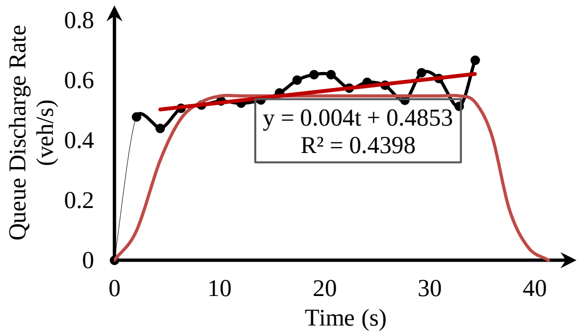
A simple linear model can be proposed based on the curve fitting method.

$$Q = \propto t + \beta \tag{11}$$

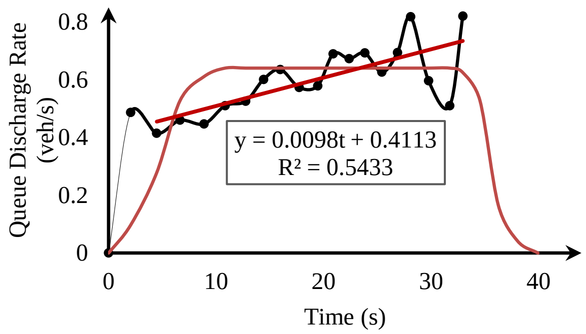
Here the values of $\propto$ and C can be computed from field observations. For six intersections in Auckland, we computed a value of 3.23 and 1788 for $\propto$ and β respectively. The proposed linear can be a simple way to reduce the errors caused by a constant saturation flow rate. However, it can create a deception of an infinite increase of queue discharge rate, which is not realistic and can lead to erroneous calculations.



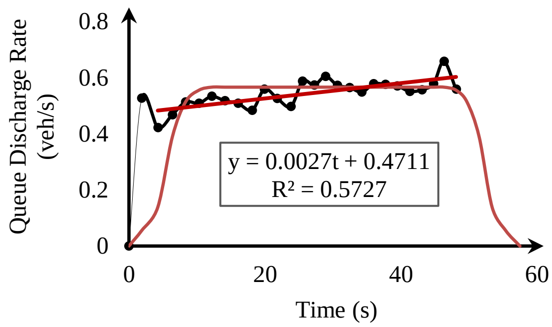
a) Balmoral - Sandringham Rd Intersection)



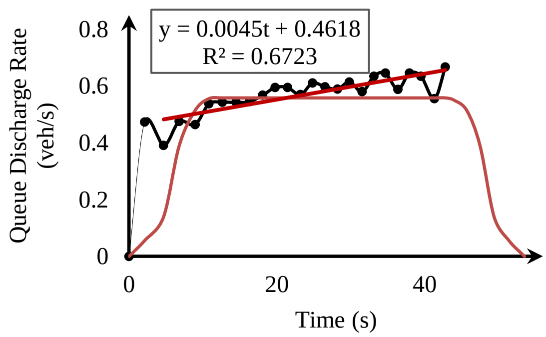
b) Balmoral - Dominion Road Intersection



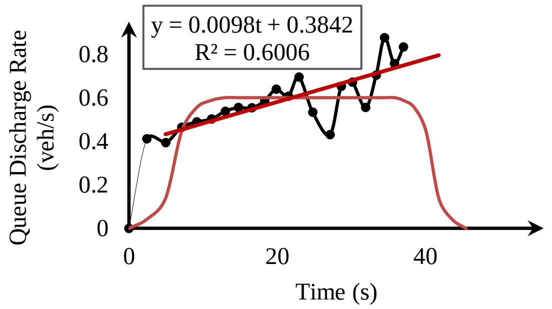
c) Great South Road - South East highway Intersection



d) Manukau - Greenlane East Intersection



e) Pah Road - Mount Albert Road Intersection



f) St. Lukes - New North Road Intersection

Figure 4 Queue discharge rate versus green time plot

Nonlinear model

The following nonlinear model can capture a relatively sharp transition from a low to high queue discharge rate followed by a marginal increase throughout the green time.

$$Q = \frac{t - l_s}{1 + h_m(t - \propto)} \tag{12}$$

where;

Q is the discharge rate after t seconds of the movement (in veh/ sec)
t is the time in seconds
l_s is the initial lost time in seconds
h_m is minimum average headway recorded after a number of cycle
$\propto$ is a correction factor

The analogy of the maximum queue discharge rate is not used here, as it is highly variable at the initial stages. Instead a queue discharge rate that is achievable in time t is used, which is dependent on the initial lost time and a signal management parameter. The initial lost time parameter incorporates the initial reaction time of the drivers after having seen the green light and started to respond. This initial lost time could vary with vehicle type and drivers' aggressiveness. h_m is the minimum average headway recorded after a number of cycles. Field observation for h_m is a better way to calculate more accurate discharge flow rate.

5. MODEL CHARACTERISTICS

The proposed models have two main characteristics. First, queue discharge rate varies with as green time passes. Second, it increases sharply initially and then it tends to increase marginally towards the end of green time. To encapsulate these characteristics, various factors are introduced in the model as discussed in the following paragraphs.

Difference between l_s and Start-up lost time: The lost time notation used here is different from the traditional start-up lost time. The traditional model makes another assumption here that the portion other than saturation flow rate is start up lost time. This start-up lost time is measured by adding additional time consumed by the first few vehicles in a queue. With the decreasing headway situation, this assumption cannot work as studies indicated that headway cannot sustain after few vehicles and continue to reduce till 7^{th} or 8^{th} vehicles and sometimes even later. ls denotes the time when green period start to the time when first vehicle start moving so it considers the driver's reaction and perception time.

Difference between h_m and h_s: The h_m (minimum average headway) used in this model is different than h_s (average saturation headway) as the former is dealing with the minimum achievable headway for the intersection while the latter is dealing with the average headway after first 4 or 5 vehicles. The observed data indicated that the flow rate at the signalized intersections grows gradually. For example, at Balmoral – Dominion Road Intersection, the headway at the 15 seconds is 2.05 seconds and at 50 seconds, the headway is 1.9 seconds, the average saturation headway (h_s) for the intersection is 2 seconds, but h_m is 1.9 seconds for this case.

Description of $\propto$: This factor is related to a general behavior of traffic stream at signalized intersections. The traffic behavior at different parts of the world behaves differently when they see the signal is turned green and they are allowed to move. In one scenario, the traffic stream will push the accelerator as soon as they see the signal turns green, as in the case of the countries with the high volume of traffic. On the other hand, the traffic stream will take a smooth acceleration pattern. In the first instance, the value of $\propto$ will be higher. For the later scenario, the $\propto$ value will be lower. A value of 0.2 to 0.4 gives a good fit in the model for the New Zealand conditions.

6. MODEL PERFORMANCE

Figure 5 presents a comparison between the proposed model and other existing models including the traditional model, Bonneson's model and Brigg's model along with the field measurements. The proposed model gives the best estimate of queue discharge rate when compared with the observed data. The traditional model and Briggs model comes to a steady state after initial increase in queue discharge rate. In Brigg's model, queue discharge rate increases sharply till the speed reaches its maximum value after which it becomes flat. In the case of traditional model, after first few vehicles it becomes flat for the rest of green time. Whereas Bonneson's model gives a similar shape to that of the proposed model except that it overestimates the queue discharge rate towards the back of queue.

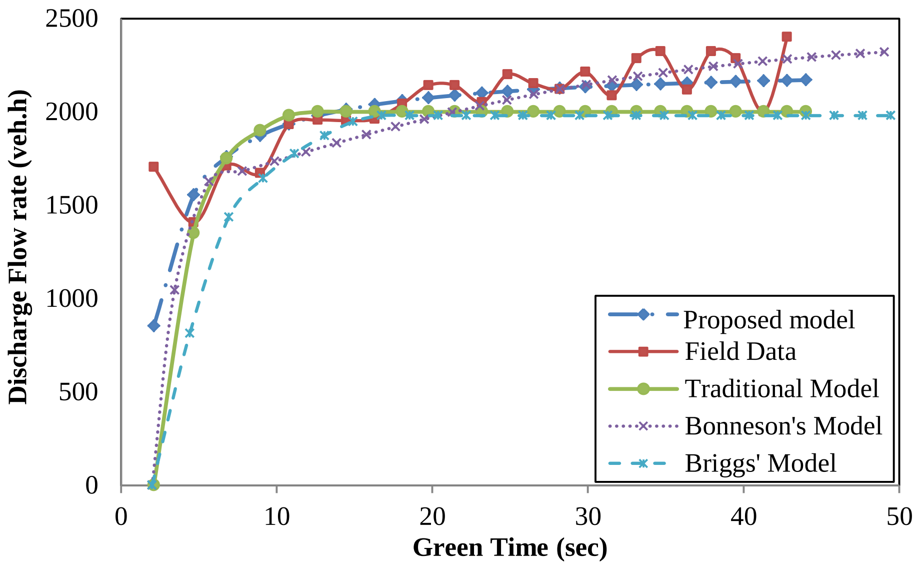


Figure 5 Comparison between different models

7. MODEL IMPLEMENTATION

Impact of the proposed model on capacity calculations: In the scenario, where the queue discharge rate is not achieving a constant high value after few vehicles, instead a steady increase of queue discharge rate is present; Eq. (1)) can be modified by replacing S with Eq. (12) and the new form of the equation will be;

$$c = \frac{g(g - l_s)}{C(1 + h_m(g - \propto))} \tag{13}$$

Eq. (13) gives the capacity calculation that is dependent on the green time allocated to the phase. The model clearly depicts that more green time means more capacity is added in the system. This impact can be seen in two states as shown in Figure 6a. In first state, vehicles start moving on the onset of green, the capacity predicted by nonlinear model is less than the traditional model. This prediction is justifiable as traditional model calculates capacity with the maximum discharge rate and does not incorporate the initial transition period in which vehicles are attaining maximum flow rate. In the second state, the nonlinear model predicts a gradual increase and the difference reaches at about 5% after 45 seconds of green time when comparing it with traditional capacity models.

To simplify this equation, a value of initial lost time and correction term of $\propto$ as zero, the equation will become;

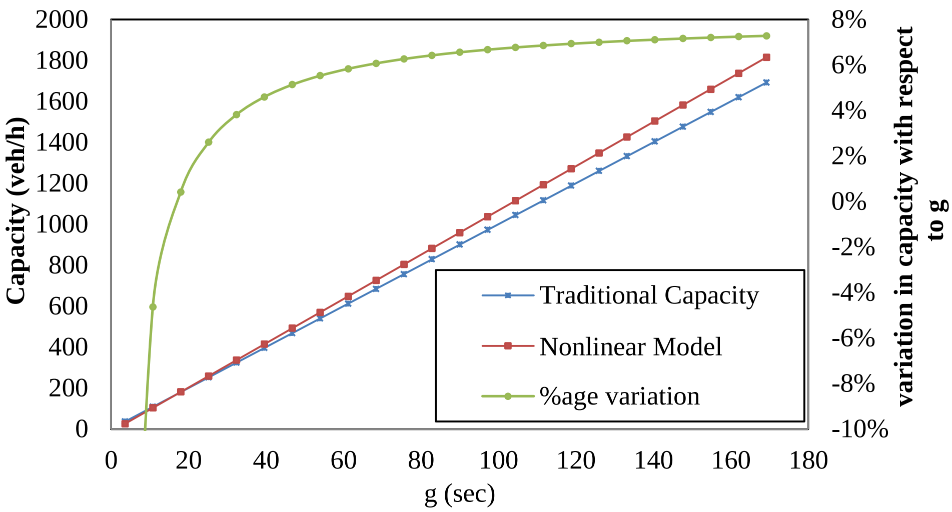
$$c = \frac{g^2}{C(1 + h_m.(g))} \tag{14}$$

The comparison of the Eq. (14) with the traditional capacity model shows that a 6.9 % increase can be gained in the 45 seconds of green time as shown in the Figure 6b. The Figure 6a and Figure 6b are showing the increase in capacity; however the difference is in the shape of percentage gain curve which is increasing sharply in start and then continues increasing gradually for later equation case.

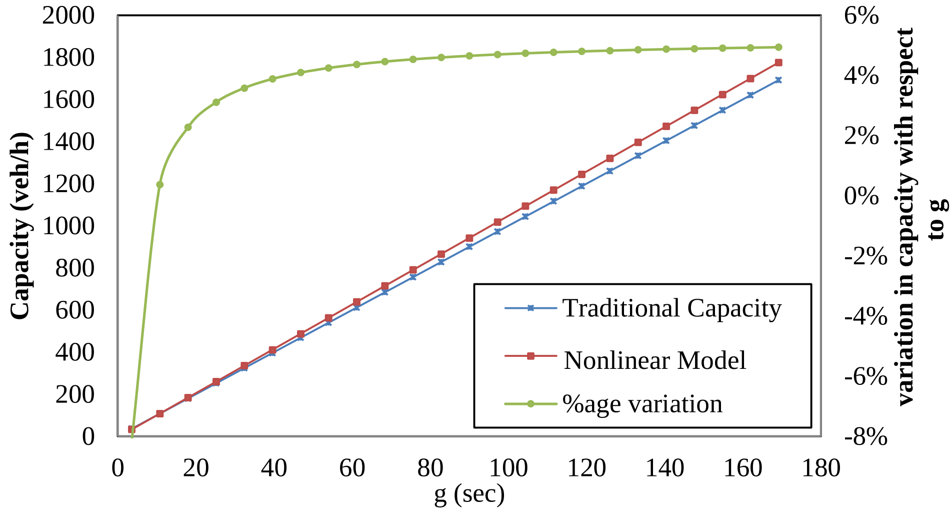
The comparison between two figures shows that the difference of percentage increase is mainly in the shape of the curve, and overall difference remains in between 5 to 7%. The existing capacity formula that account only green time ratio (g/c), does not account into the green time variation that affect the increase in queue discharge rate. For example, a 0.4 ratio for the 100 second cycle time will allow a 40 seconds green time for that particular approach. The same ratio, with 30 seconds cycle time will allow a 12 seconds green time. The traditional approach does not differentiate between these two cycle times and the capacity of that approach will be 720 pc/h/l. However, the modified model indicated that a 3.9% increase in capacity is possible at 40 seconds green time with a capacity of 748 pc/h/l instead of 720 pc/h/l.

Impact of the proposed model on signal timing calculation: The effect of the green time on queue discharge rate makes it difficult to calculate flow ratios directly as flow ratios will be changed if the green time is changed. For example, the flow ratios that give a green time of 20 seconds and 45 seconds based on a fixed saturation flow rate might not be correct, as the queue discharge rate may be different at these two points. Therefore, instead of introducing a flow ratio concept, a green time demand concept is required that allocate the green time based on the demand flow.

The green time demand calculated with this model indicates that relatively more green time demand is required for queues formed due to low traffic volumes. This fact is self-explanatory, as in low volumes the queue may not be long enough to reach the traditional maximum discharge flow rate. Traditional model is silent in this condition and flow ratios calculated are based on maximum discharge rate. In high volumes, the green time demand decreases. Figure 7 explain this phenomenon in terms of traffic volume and green time demand. The basis of green time ratio in traditional models considers a single value of saturation flow rate and it doesn't matter if the green time ratio is calculated for 4 vehicles in a cycle or 40 vehicles. However, the proposed model is capable to handle this short coming and proposes a demand as per the time required to clear queue in varying flow conditions.



a) Variation in capacity using $\mathbf{l_s}$ and $\propto$



b) Variation in capacity with $\mathbf{l_s} = 0$ and $\propto = 0$

Figure 6 Impact on capacity calculations

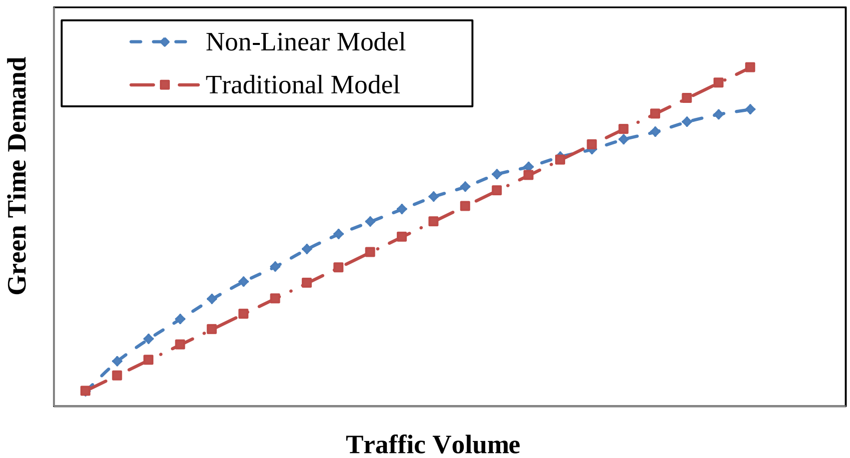


Figure 7 Comparison between traditional and the proposed non-linear model for green time demand (a conceptual illustration)

The green time demand for all approaches can help in determining critical movements and sum of critical movements can be utilized for the cycle time calculations. For the pre-timed signalized intersection, this demand concept does not change, and the only thing that can limit is when maximum signal timing has a restraint. In that case, the green times can be adjusted based on priorities and effect of the over saturation can be measured. The formula for the minimum cycle time can be derived by equating sum of green time calculations with total lost time. Mathematically it can be described as

$$C\left(1-\sum g_r\right)=L \tag{15}$$

where g_r is green time demand (unit less). From this equation, cycle time can be derived as;

$$C = \frac{L}{1-\sum g_r} \tag{16}$$

Multiplying both sides of Eq. (16) with $X_c$will make this equation to somewhat same as Equation 31-68 described in chapter 31 of HCM2010. However, with this formulation, $X_c$value that is arbitrarily assumed is not necessary and cycle time can be derived directly.

The modified form of Webster cycle time formula, based on the green time demand is;

$$C = \frac{\alpha L+\beta}{1-X\sum g_r} \tag{17}$$

where X is degree of saturation and α and β can be used as per Webster (α =1.5 and β=5) or as per ARR 123(α=1.4 and β=6).

The integral of the equation gives the number of vehicles that can pass the intersection in the time t. Assuming the simplified case, with replacing *t* with green time *g*; $\int \frac{g}{1+h_m.g}.dg$.

Total number of vehicles passing through intersection

$$= \frac{h_m.g-\log(1+h_m.g)}{{h_m}^2} \tag{18}$$

This integration is useful as it can describe how much vehicles can cross the intersection in a given g time.

Example: A case used in the Webster's paper is verified with modified traffic volumes for the methodology described above. A four legged signalized intersection that allow through movements only is having an hourly flow rates of 900 pc/h/l for North and South approach and 600 pc/h/l for East and West Approach. A saturation flow rate of 1800 pc/h/l gives flow ratios for North and South approach is 0.50 and for East and West approach is 0.33.

Traditional approach with fixed saturation flow rate and with an assumption of a two phase signal, the critical sum of flow ratios is 0.83, which gives the optimum cycle time of 102 seconds from Webster formula with total lost time of 8 seconds and 108 seconds for practical cycle time. Assuming a $X_c$value of 0.9, the cycle time from HCM will be same as practical cycle time. This result does not dependent on green time allocation on approaches. The model indicated that a more green time allocated will increase capacity, hence reduces the overall cycle time. The results for the practical cycle time calculated based on variable queue discharge rate as shown in Figure 9(a). For optimum cycle time, the model predicted the same pattern with increase in queue discharge rate reduces the cycle time.

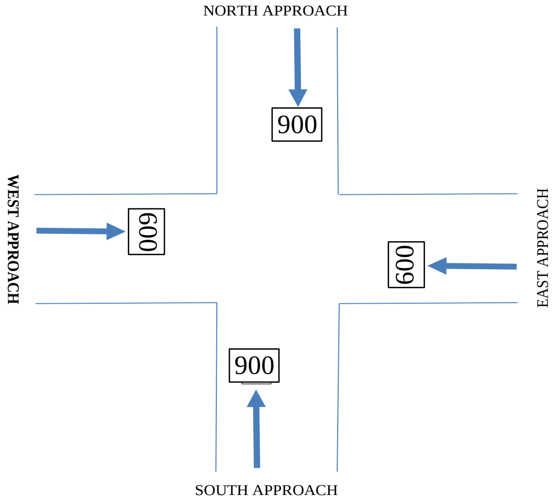
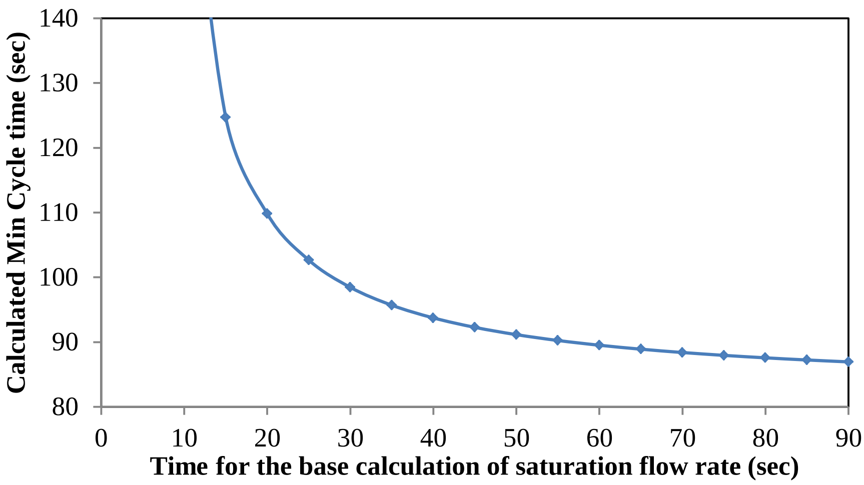
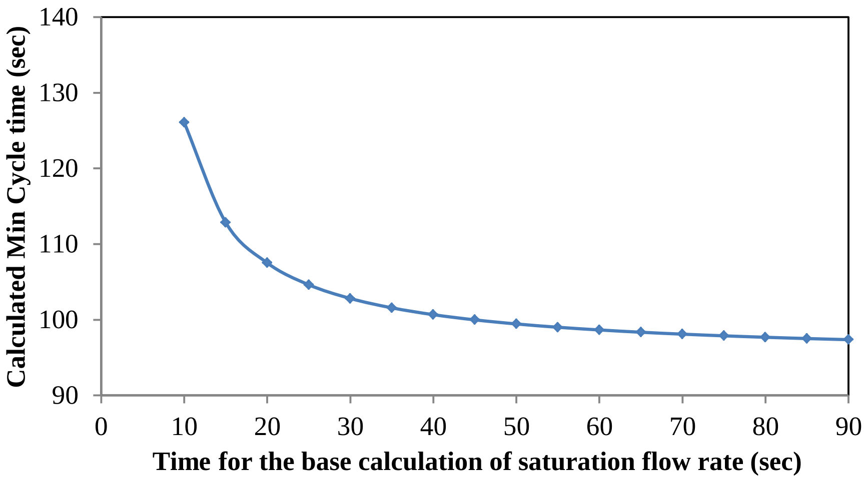


Figure 8 Signalized intersection example



a) Practical cycle time based on variable queue discharge rate



b) Optimal cycle time based on variable cycle time

Figure 9 Cycle time based on variable queue discharge rate

7. CONCLUDING REMARKS

We have presented results of field observations conducted at six signalized intersections in Auckland to investigate queue discharge behavior of drivers. These field observations have confirmed the findings of previous researchers that the queue discharge rate at signalized intersection cannot be approximated using a constant flow rate termed as saturation flow rate. It rather has an increasing trend towards the back of queue. We have proposed two empirical models to reflect the observed queue discharge behavior and implemented one of them to calculate the capacity and signal timings. The proposed nonlinear model is relatively simple am compatible with the existing methodologies yet it closely replicates the queue discharge behavior observed at the signalized intersections in Auckland. The approach capacity is shown to have increased by 5 to 7% using the proposed model. The model is yet to be investigated comprehensively for its practical applications. It can be applicable to any intersection experiencing a gradual compression of discharge headway provided that the model is properly calibrated for the local traffic and control conditions.

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