EMPIRICAL DELAYS FROM ACTUATED AND OPTIMISED STATIC SIGNAL SETTINGS COMPARED

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ABSTRACT

Many intersections have varying mechanism for vehicles right of way as they approach the intersection. With actuated signal, induction loops buried in the roadway stop-line, video, infrared or microwave detection system automatically adjusts timings relative to prevailing degree of saturation. An intersection without such detection system operates on fixed times (static). Signal settings are based on fixed proportional distribution of effective green per cycle time. In the paper, daylight and dry weather traffic performances at standalone signalised 4-way intersection were investigated under actuated and optimised signal timing conditions. Based on the hypotheses that peak traffic performance at standalone between optimised static and actuated signal settings are insignificant; discharge rates, delays and effective green timings for both were estimated compared and contrasted. Given that an optimised static signal assigns predetermined time irrespective of traffic demand; saturation flows were fixed at 1900 for straight, 1800 left turning and 1700 right turning vehicles per hour per lane respectively. Results show marginal differences in peak period effective green, discharge rates and delays. The paper concluded that optimised static signal can produce good results and should also be considered especially at standalone intersections where traffic operations are at peak regularly.

Keywords: traffic signals, intersection, saturation flow, actuated signal control system, static signal system.

1. INTRODUCTION

Traffic signals are control mechanisms aimed at minimizing conflict movements and maximizing traffic flows at roadway intersections. The design of signalised intersections is anchored on ability to manage saturation since that is the basis for intersection performance evaluation and signal timing determination. Saturation flow is the most important single parameter in the capacity analysis of signalised intersections. Besides, saturation flow is the maximum constant departure rate of a queue from the stop line of an approach lane during the green period. A small change in the saturation flow value may result in a relatively large change in the calculated cycle time and the duration of the necessary green intervals. In fixed operation, a controller has a set programmed time to service all movements per cycle irrespective of prevailing traffic demand. The controller will service all movements whether or not there is vehicle demand. When a detector at an actuated signal breaks, that movement will then have to operate as fixed until the detector is repaired.

Advocates of actuated signal control system have argued that the system is more effective and efficient because it's responsive. That may be true in clustered urban areas with short intersection spacing often less that 200m probably less true in a standalone intersection. It's often the case that fixed-time signal system has been relegated and sometimes classified as outdated. Research into semi and fully actuated signal timing is enormous, with little attention paid to fixed timing system systems. Semi-actuated means the intersection has detection on the minor roadway approaches and major road left turns only. To save money on maintenance, some agencies opt to design an intersection as semi-actuated. The major roadway is then programmed to operate a fixed time every cycle, but the controller will service the other movements only when there is demand. In any case, most urban traffic signals operate in a semi-actuated mode. Previous empirical research provides contradictory and inconclusive evidence on the relevance of static signal at stand alone intersection. Most acknowledge the usefulness of static signal, often conceding that their performances are limited and uneconomical without direct comparisons under same empirical conditions.

In this study, static traffic signal settings were optimised and their performances compared with those of actuated signal settings. The study was carried out at a standalone 4-way signalised intersection in Kota Tinggi town, Malaysia. Kota Tinggi is a small historic town 42 kilometres north-east of Johor Bahru in Malaysia. The site was chosen because of it's near endless at peak periods during daylights. In passing, it is undoubted that from year to year, the number of registered vehicles in the area and far beyond has increased significantly. Nevertheless, vehicle registration increases do not translate to direct traffic flow increase even though there is a correlation between the two. The increases are mere pointers to the probability of increase in traffic demand. Even in cases where the degree of correlation is strong, the roadways cannot exceed their traffic carrying capacities.

Although static signal systems are considered as old-fashioned while most intersections are controlled by closed loop controllers, traffic flows during peak hours become nearly especially at standalone intersections. That makes modeling fixed time intersections delays at stand alone nodes still relevant and important even today. The paper provides direct comparisons of delays, discharge rate and degree of saturation between fixed and actuated traffic signal systems. The dynamic and stochastic behaviour of queue lengths and delays at pre-timed traffic control signals based on Webster [4] delay theory. This method allows the analyst to estimate and predict the dynamic evolution of queues and the propagation of their distribution in time.

Based on the hypotheses that peak traffic performance at standalone intersection between optimised static and actuated signal settings are insignificant; discharge rates, delays and effective green timings for both were estimated compared and contrasted. In the light of the discussion so far, the remainder of the paper has been divided into four sections with section 2 reviewing relevant literature on traffic signals. Section 3 is on study settings and data collection; in Section 4 collected data are collated, analyzed and their findings discussed. Section 5 concludes the paper.

2. REVIEW OF LITERATURE

Roadways are maze of links and nodes. Often the nodes are problematic, inducing delays by way of slowing down approaching vehicles, and in cases where pedestrians presence in significant, encroaching on their safe passage. Where conflict of movements at nodes is significant enough traffic signals are often deployed as control mechanism. A key to the design of signalised intersections is by the ability to predict saturation flow, since it is the basis for the evaluation of intersection performance and in determining the traffic signal timings. Cognisance must be taken of factors with impact on saturation flow, such as; Position of lane, Width of lane and gradient, Turing radius, Grades, Bus stops, Pedestrians and bicycles activities, Heavy vehicles, Kerbside parking, Precinct development activities, and Traffic distribution among others.

The fundamental role of traffic delays and queues is confirmed by its use in the computation of the level of service e.g. in the Highway Capacity Manual 2000, in the design of infrastructures, and in the estimation of environmental costs (e.g. fuel consumption and gas emissions). Performance assessment is based on assumptions regarding the characterization of the traffic arrival and service processes [6]. The most important traffic performance, which determines the functionality of signalized intersections, is the signal delay. This characteristic is usually defined as the extra (waiting) time a traveler experiences due to the signal. Delay itself is a function of saturation flow. Saturation flow rate is the number of vehicles served per lane for one hour of green time.

Traffic control at intersections is automated by way of cyclic sequence of green, amber and red lights. Traffic signal operation can either be static or actuated. The timing of the green and red duration can be fixed and predetermined. When the static signal is set at optimum performance level, it is expected to operate like actuated signal at peak. Actuated signal can be fixed or semi depending on the economics of need. The choice of the control type and the determination of the optimal control phases to adopt at one intersection are mainly done with the objective of reducing the delay to the vehicles.

Since signal delay is partly caused by queues forming upstream the signal, the modeling of these queues represents also an important area of considerable interest. However, this paper deals primarily with empirical comparisons of optimised static and actuated signal traffic performance, delay modeling will only be mentioned briefly in passing. Delay at signalised intersections constitutes a major part of travel time in street networks, and is dependent on many factors including the drivers' behaviours, physical specifications of the intersection and the streets connected to it, traffic signal timing at the intersection and the traffic volume or composition in the intersection approaches.

Delays due to signal operations and queues alone are called control-delay, if also lost time due to acceleration and deceleration is added into the computation then its stopped delay. In any case, delay at fixed-time controlled intersections has been a study subject for many years. Several mathematical expressions have been derived to represent the so-called random delay component, the delay caused by the stochastic character of arriving traffic. Beckmann *et al*., [4] presented a linear expression of delay as:

$$E(W) = \frac{t_r}{t_c\left(1-\dfrac{q}{s}\right)}\left[\frac{E(Q_0)}{q} x \frac{t_r+1}{2}\right] \quad (1)$$

Where

E (W) - expectation value of delay (s)
t_c - signal cycle
t_r - length of the effective red time
q - Arrival flow rate
s - Saturation flow
E (Q_0)-Expectation value of overflow queue length (vehicles)

The assumption used by some authors that the queue should be represented by a step function appears to be superfluous. The stepwise character of the delay is transformed to a smooth character of the expected delay, linearly increasing in the red-phase and the first part of the green phase. The expectation value of the queue in the green phase shows a non linear character as soon as the tail of the probability distribution comes close to zero. This phenomenon causes the overflow delay. However, Webster [4] presented delay model where the first term is analytical derivation of uniform delay, while the second term is a characterization of stochastic delay derived analytically assuming Poisson distribution. The last term has been introduced to reduce the discrepancy with results observed from simulation data. The formula has been simplified as:

$$d = \frac{9}{10}\frac{[C(1-\sim)^2}{[2(1-\sim x)} + \frac{x^2]}{2q(1-x)]} \quad (2)$$

Where,

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d = average delay per vehicle
c = cycle length, sec;
q = flow, vehicles/sec;
~ = proportion of the cycle that is effectively green for the phase under consideration length (i.e. g/c)
x = degree of saturation, which is the ratio of the actual flow to the maximum flow that can pass through the approach (i.e q/s)

The optimum cycle time C_o could be obtained by using the following equation,

$$C_o = \frac{1.5L + 5}{1 - \sum y \max} \tag{3}$$

Where,

L is the total lost time per cycle
Y is the sum of the maximum y values for all cycle phases.

In general, the minimum cycle time should not be less than 45 sec and the maximum cycle time should not exceed 120 sec. longer cycle times result in increased delay and queues for all users while shorter cycle times cannot provide adequate green time for all phases. Cycle time is dependent on saturation flow for proportional distribution of signal timings. Saturation flow is the most important single parameter in the capacity analysis of signalised intersections. It is measure of the maximum rate of flow and it is used extensively in junction design and control applications. Besides, saturation flow is the maximum constant departure rate of a queue from the stop line of an approach lane during the green period. A small change in the saturation flow value may result in a relatively large change in the calculated cycle time and the duration of the necessary green intervals [3]. To achieve optimal efficiency and maximize vehicular throughput at the signalized intersection, traffic flow must be sustained at or near saturation flow rate on each approach.

Saturation flow can be defined as number of vehicles that would pass through the intersection during that hour is the saturation flow rate. Arahan Teknik Jalan [1] 13/87, saturation flow is defined as the maximum flow, expressed as equivalent passenger cars that can cross the stop line of the approach where there is a continuous green signal indication and a continuous queue of vehicles on the approach. Based on US Federal Highway Administration (FHWA), saturation flow is the equivalent hourly rate at which vehicles can traverse an intersection approach under prevailing conditions, assuming a constant green indication at all time and no loss time, in vehicles per hour or vehicles per hour per lane. The saturation flow rate at signalised intersections under ideal conditions is taken as 1900 passenger cars per hour of green per lane. Although saturation flow can be determined by many methods, the headway technique and the Webster equation show below:

$$s = 3600 / h \tag{4}$$

Where:

s = saturation flow rate in veh/h,
h = saturation headway in s/veh, and
3600 = number of seconds per hour.

Highway Capacity Manual 2000 measures traffic performance of a signalised intersection by computing the expected delay per vehicle and decomposing it into 3 terms:

$$W = W_1 PF + W_2 + W_3 \tag{5}$$

Where

W_1 is the uniform stopped delay/vehicle (s/veh)
W_2 is the incremental stopped delay (s/veh)
W_3 is the initial queue
PF is the progression factor to account for signal coordination
The first two delay components are given by the following formulas:

$$W_1 = 0.5 t_c \frac{\left(1 - \frac{t_g}{t_c}\right)^2}{1 - \min(1, x) \frac{t_g}{t_c}} \tag{6}$$

$$W_2 = 900T \left\{ (x-1)^2 + \sqrt{(x-1)^2} + \sqrt{\frac{8KI f x}{cT}} \right\} \tag{7}$$

Where 1 is for fixed time, 0.5 for semi-actuated and somewhere between 0.04 and 0.5 for actuated. *If is* the filtering adjustment factor; accordingly, this formula assumes the queue length to be constant and finite if $x < 1$ while it behaves according to the linear deterministic function for $x > 1$. The third component is computed by specifying the parameters of the formula:

$$W_3 = \frac{1800\, Q(0)(1+u)t}{c.T} \tag{8}$$

3. SETTINGS AND DATA COLLECTION

The study deals primarily with derived delays from Webster equation with little importance given to acceleration and deceleration effects. In order to compute delay, degree of saturation and by extension saturation flows must be computed. By applying the headway technique illustrated below, saturation flow can be estimated using equation (4). Delay computed with equation (2) and effective green relied on equation (3) for estimation. Actual green and red were hand timed. Geometric information was culled from Google earth and directly measured for acceptance. Automatic counters were installed at the entry arms for six weeks during daylight and dry weather conditions. Although headway, speed, volume, vehicle types and gap acceptance information were readily supplied, the automatic counters were useful pointer to weekly variations in traffic volumes, surveys were carried at peak hours as indicated by the automatic counters.

The 4-way intersection operates on 4 stages actuated signal timings, that is one stage per arm. Three

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signal phases (green, amber and red light) were simplified into two phases, effective green time and effective red. The effective green time is the green time from which the green start lag is subtracted and a green end lag is added. The green start lag is partly due to the reaction time of the first driver passing the intersection during green, but most of the time is the consequence of the vehicles' acceleration operations, which make the speed of the first few vehicles passing the signal lower than in the middle of the green phase. At the end of the green phase, during the amber, vehicles still enter the intersection. The average time that the amber phase is still used by vehicles entering the intersection is called the green end lag.

Using the concept of effective green time, the flow is assumed to reach instantaneously the maximal saturation flow rate at the moment the signal turns green, and to stop when it turns red. Vehicles arriving during the green phase pass the stop line without delay if there is no queue to be served. Vehicles arriving during the red phase, experience a delay, which depends prevailing conditions. Saturation headway is the headway of the vehicles in a "stable moving platoon" passing through an effective green light. A stable moving platoon is a group of vehicles that are traveling, but not really moving in relation to each other (i.e. all going the same speed). The headway of the first four vehicles leaving an intersection after a red light will have a higher value so the saturation headway will not be realized until the 4^{th} or 5^{th} queued vehicle leaves the intersection. So, if every vehicle requires a time equal to the saturation headway (h), in seconds, to be serviced at a signalized intersection, then the maximum number of vehicles that can be serviced in an hour of green is given by the equation (4).

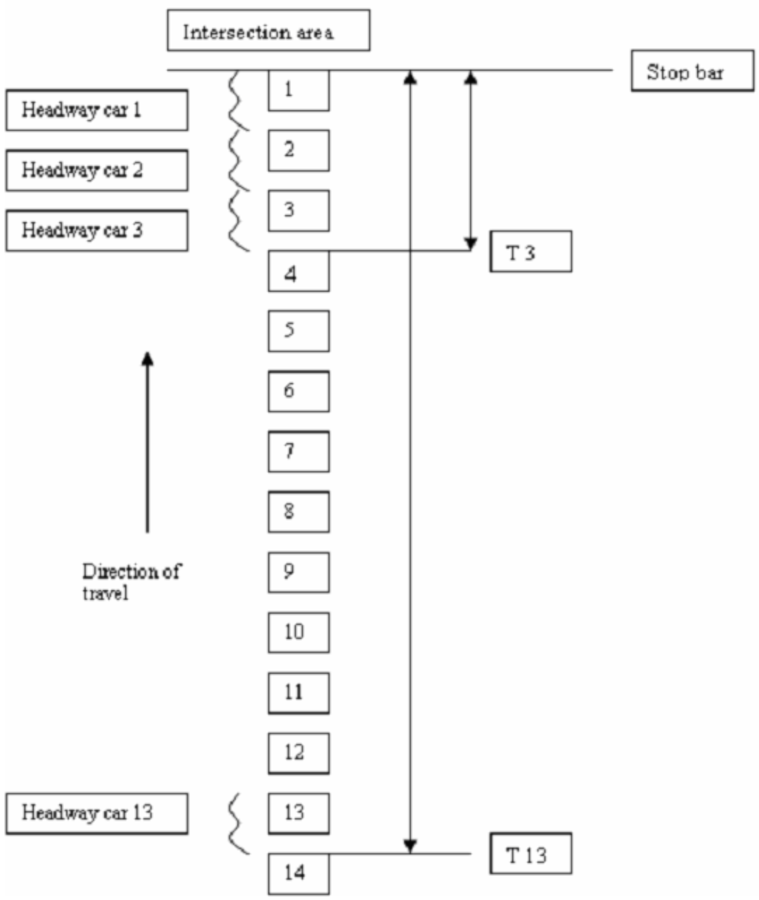


Figure-1. Typical set-up of headway survey

4. ANALYSIS AND DISCUSSIONS

This paper provides direct comparisons of delays, discharge rate and degree of saturation between fixed and actuated traffic signal systems. The dynamic and stochastic behaviour of queue lengths and delays at pre-timed traffic control signals based on Webster [4] delay theory. Equation (2) was used to compute average delay per vehicle with results shown in Table-1. Given that an optimised static signal assigns predetermined time irrespective of traffic demand; saturation flows were fixed at 1900 for straight, 1800 left turning and 1700 right turning vehicles per hour per lane respectively. A saturation flow rate of 1900 vehicles/hour/lane, which corresponds to saturation headway of about 1.89 seconds, is a fairly common nominal value. Design manuals usually provide adjustment factors that take parameters such as lane-width, pedestrian traffic, and traffic composition into account.

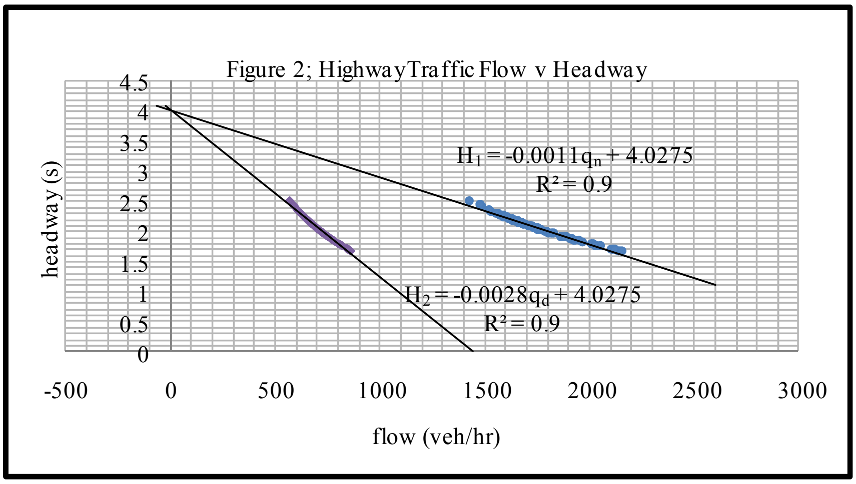
Based on the hypotheses that peak traffic performance at standalone intersection between optimised static and actuated signal settings are insignificant; discharge rates, delays and effective green timings for both were estimated compared and contrasted. Two hundred and fifty samples of headway and phase timings were collected at Kota Tinggi 4-way intersection during dry weather and daylight conditions. Estimated headways were then related to traffic flows and computed saturation flows to produce Figure-2 shown below. Consider headway model equations (9) and (10) shown in Figure-2.

$$H_1 = 4.0275 - 0.0011q_n \quad (9)$$

$$H_2 = 4.0275 - 0.0028q_d \quad (10)$$

Where

H_1 - headway for saturation flow
H_2 - headway for demand flow
q_n - saturation flow
q_d - demand flow



The ultimate headway for the intersection is approximately 4 seconds. The maximum demand flow derived from model equation 10 is about 1440 vehicles per hour or 720 vehicles per lane given that the road carriageway has 2 lanes. The maximum saturation flow derived from model equation (9) is approximately 3660 vehicles per hour, or 1830 vehicles per lane. Obviously when headway is zero, flow is zero because no vehicle is traversing the roadway. In any case one parameter that would of interest is capacity. Capacity is the maximum hourly flow of vehicles that can be discharged through the intersection from the lane group in question under the prevailing traffic, roadway, and signalization conditions. It is an adjustment of the saturation flow rate that takes the real signal timing into account. Capacity (c) is given as;

$$c = (g/C) \cdot s \quad (11)$$

Where

c = capacity (v/hr)
g = Effective green time for the phase in question (sec)
C = Cycle length (sec)
s = Saturation flow rate (v/hr)

By optimising static signal timings, the following assumptions were made irrespective of prevailing traffic demand road per lane; cycle time was fixed at 120s; saturation flow for vehicles going straight was fixed at 1900v/h/l; right turning vehicles at 1700v/h/l and left turning vehicles at 1800 v/h/l. Summaries of the analysis are shown in Tables 1 and 2. Capacity (C) was used as a reference to gauge prevailing operation of the intersection. It was calculated for individual lane per approach arm. Prevailing flow rate at node 11 is about 81% for optimised static; 87% for actuated and at node 12 optimised static is 82%; while fully actuated signal is 89%.

Table-1. Empirical flow data of actuated and optimised static traffic signal.

Stage	Turning movements	Demand flow vph	Saturation		Degree of saturation		Y_{max} per stage	
			Actuated	Opt. static	Actuated	Opt. static	Actuated	Opt. static
Node 11	straight	430	1698	1900	0.253	0.226	0.2782	0.2717
	s/r/l	489	1758	1800	0.278	0.272		
Node 12	right	327	1547	1700	0.211	0.192	0.2255	0.2183
	s/r	393	1743	1800	0.225	0.218		
Node 13	straight	123	1753	1900	0.070	0.065	0.1285	0.1244
	s/r/l	224	1743	1800	0.129	0.124		
Node 14	s/r	255	1690	1800	0.151	0.142	0.2009	0.1788
	right	304	1513	1700	0.201	0.179		
						∑Y	**0.8331**	**0.7933**

Table-2. Traffic performance of actuated and optimised static traffic signal

Descriptions	Stages			
	Node 11	Node 12	Node 13	Node 14
Flow v/hr/lane	459	360	173	279
Capacity (C) v/hr/lane actuated	526	405	240	348
Capacity (C) v/hr/lane optimised	570	438	262	350
Effective green G_f (s) actuated	42	34	19	30
Effective green G_f (s) optimised static	37	30	17	24
Discharge Rate per G_f vehs/G_f - actuated	20 (2.08)	16 (2.19)	9 (2.06)	13 (2.25)
Discharge Rate per G_f vehs/G_f - optimised	20 (1.89)	16 (1.89)	10 (1.89)	16 (1.89)
Actuated delays (s)	34.8	37.5	47.2	44.3
Optimised static delay (s)	31.5	32.3	40.6	36.1
Delay differences	3.3	5.2	6.6	8.2
∑Cal.X^2 (df.3, 5%) = 4.12 < 7.815 Tab X2 Accept Null hypothesis that differences in delays are insignificant	0.35	0.84	1.07	1.86

Note: Headway (s) given in brackets

While each stage of a cycle can service several movements, some will inevitably require more time than others to discharge their queue. If each phase is long enough to discharge the vehicles in the most demanding movement, then all of the vehicles in the movements or lanes with lower time requirements will be discharged as well. The critical movement for each phase was determined using flow ratios shown as Y maximum in Table-1. Three nodes 11, 12 and 14 exhibit flow rates at and above 80% of which node 12 and 14 have nearly the same flow rate. Thus, the remainder of comparative analysis is based on Nodes 11 and 12.

As contained in literatures, efficiency dictates that the cycle length should be long enough to serve all of the critical movements, but no longer. If the cycle is too short, there will be so many phase changes during an hour that the time lost due to these changes will be high compared to the usable green time. But if the cycle is too long, delays will be lengthened, as vehicles wait for their turn to discharge through the intersection. Consequently, optimised static cycle time was fixed at 120s, while equation (3) was used to compute fully actuated cycle time being 138s. Critical flow ratios were used to allocate the available green time. Each phase is given a portion of the available green time that is consistent with the ratio of its critical flow ratio to the sum of all the critical flow ratios. Effective green times for the intersection are shown in Table-2.

Delay is perhaps the most significant performance measurement, it can be argued. Average delay shown in Table-2 can be defined in terms of the level of service (LOS) offered to road users. As contained in most literatures operations with delays between 25.1 and 40.0 seconds per vehicle is classified as LOS D. Where congestion is noticeable and longer delays may result from a combination of unfavourable progression, long cycle lengths, and high V/c ratios. Nodes 11 and 12 under optimised and actuated signaling are in this class. Obvious there is advantage resulting from the type of signal system in place, probably explaining why agencies have shifted attention away from fully actuated to semi-actuated signal timing. In any case the difference in delay between the two types of signal system is insignificant as shown in the statistical test results (Table-2). It can even be asserted that at standalone intersection, optimised static is just as effective as fully actuated signal system. Probably not as efficient in terms of running costs at off peak periods; the deficiency can be accommodated by running the system on part time basis bearing in mind the cost associated to safety. Thus if savings from safety is factored into the system, it can be argued that optimised static is just as efficient.

5. CONCLUSIONS

In evaluating signalized intersection performance, three measures are commonly used flow ratios, delay and queue. Based on the hypothesis that, optimised static signal is just as effective as a fully actuated signal system and also the findings from this study, it can be concluded that:

- Difference in effective green time between fully actuated and optimized static signal timing is minimal;
- Difference in delay per vehicle between fully actuated and optimized signal timings is significant;
- The level of service at standalone intersection from optimized static and fully actuated signal system is the same;
- The choice of signal setting is dependent on the intersection of interest and its traffic characteristics; and
- The assertion that optimised static is not as effective as fully actuated signal system is null and void.

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