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SATURATION FLOW VERSUS GREEN TIME AT TWO-STAGE SIGNAL CONTROLLED INTERSECTIONS

Ahmed Boumediene[1], Kouider Brahimi[2], Noreddine Belguesmia[3], Khaled Bouakkaz[4]

[1, 2]Université des Sciences et de la Technologie Mohamed Boudiaf Oran BP 1505 El-Menaour 31000 Oran, Algeria
[3]Université de Mostaganem, BP 677 RP 27000 Mostaganem, Algeria
[4]Université de Tiaret, Algeria
E-mails: [1]aboumediene@univ-usto.dz; [2]kbrahimi@univ-usto.dz; [3]nbelguesmia@univ-mosta.dz; [4]bouakkazk@yahoo.fr

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Abstract. Intersections are the key components of road networks considerably affecting capacity. As flow levels and experience have increased over the years, methods and means have been developed to cope with growing demand for traffic at road junctions. Among various traffic control devices and techniques developed to cope with conflicting movements, traffic signals create artificial gaps to accommodate the impeded traffic streams. The majority of parameters that govern signalised intersection control and operations such as a degree of saturation, delays, queue lengths, the level of service etc. are very sensitive to saturation flow. Therefore, it is essential to reliably evaluate saturation flow for correctly setting traffic signals to avoid unnecessary delays and conflicts. Generally, almost all guidelines support the constancy of saturation flow irrespective of green time duration. This paper presents the results of field studies carried out to enable the performance of signalised intersections to be compared at different green time durations. It was found that saturation flow decreased slightly with growing green time. Reduction corresponded to between 2 and 5 pcus/gh per second of green time. However, the analyses of the discharge rate during the successive time intervals of 6-seconds showed a substantial reduction of 10% to 13% in saturation flow levels after 36 seconds of green time compared to those relating to 6–36 seconds range. No reduction in saturation flow levels was detected at the sites where only green periods of 44 seconds or less were implemented.

Keywords: saturation flow, traffic signals, green time, intersection, capacity.

1. Introduction

Among various traffic control devices and techniques developed to cope with conflicting movements traffic signals create artificial gaps to accommodate the impeded traffic streams. There are no standard criteria on which traffic signalling is based. It depends on many factors including traffic flows, pedestrians' safety, traffic conflicts, economical benefits, etc. Traffic signals must be correctly set to avoid unnecessary delays and optimise the objective function being considered as a measure of effectiveness. However, this cannot be achieved unless the number of vehicles that can discharge through the intersection is known. This is true whether the signals are operating independently or linked to other signals of the network in the area. Saturation flow and cycle time settings are two principal factors that govern junction capacity (Junevičius and Bogdevičius 2007; Berezhnoy *et al*. 2007; Daunoras *et al*. 2008; Kinderytė-Poškienė and Sokolovskij 2008; Gowri and Sivanandan 2008; Akgüngör 2008a, b; Jakimavičius and Burinskienė 2009).

A considerable amount of work has been done in the development of design criteria and in the theoretical investigation of intersection control by traffic signals to evaluate the effectiveness of signal techniques. However, the optimum control of traffic signal-controlled intersections still continues to be one of the major concerns of traffic engineers.

As flow levels and experience have increased over the years, methods and means have been developed to cope with growing demand for traffic. Methods developed for calculating the capacity of signalised intersections are based on assumptions that should be adapted to local traffic environment.

Optimum cycle length and splits formulae have been developed including different factors as a measure of effectiveness. Webster and Cobbe (1966) considered

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the minimum average total delay imparted to all vehicles passing through the intersection (eq. 1). Miller (1963) took into account variations in traffic distribution from cycle to cycle (eq. 2), whilst Akçelik's (1981) approximate optimum cycle time formula (eq. 3) embodied a stop penalty parameter.

$$C = \frac{1.5L + 5}{1 - \sum (Qi/Si)}, \tag{1}$$

$$C = \frac{L + \alpha\sqrt{IL/S}}{1 - \sum (Qi/\pi Si)}, \tag{2}$$

$$C = \frac{(1.4 + K)L + 6}{1 - \sum (Qi/Si)}, \tag{3}$$

where: C – cycle time (sec.); Q_i – the actual flow of the subject lane for stage i (pcu/h); S_i – the saturation flow of the subject lane for stage i (pcu/h); L – total lost time over cycle time (sec.); I – the variance/mean ratio of the counts of arrivals per cycle; K – stop penalty parameter; A – 1.41 if queue length during the peak period is an important factor; π – the proportion of the total available effective green time per cycle.

All these methods make use of the saturation flow concept as the basis for signal timing and hence capacity calculation. Thus, to determine this 'capacity', it is essential to be able to reliably evaluate saturation flow on each approach of the intersection. Indeed, knowledge of saturation flow improves the accuracy of capacity calculation and provides the mechanism of a better understanding of the phenomena of intersection performance as the majority of parameters that govern signalised intersection control and operations such as degree of saturation, delays, queue lengths etc. are very sensitive to variations in saturation flow.

2. Saturation Flow and Green Time

A number of researches have been (and certainly will be) undertaken for a better understanding of the saturation flow concept. Almost all of these studies support the constancy of saturation flow irrespective of green time duration, although most investigators do not specify the range of green times tested. Since green times of about 30 to 40 seconds are usually encountered, it is highly probable that observations have been mainly limited to such a range when this assumption was made. However, the question of checking the constant saturation flow assumption over a wider range of green time has been addressed since 180's and some of the results are remembered hereafter.

It is conventionally assumed that during green time, the discharge pattern of the queue built-up during the red period consists of three stages:

- Drivers take some time to react to the green light of way signal and then move off from the standing position and accelerate to normal desired speed.
- After a few seconds, the queue discharges at a more-or-less constant rate called saturation flow until green time runs out.
- As amber signal starts, some drivers make use of it, whereas others being more cautious do not, and therefore the discharge rate falls to zero a second or two into the all-red period.

Saturation flow is defined as a flow that would be obtained if there were a continuous queue of vehicles and they were given 100 per cent green time. The current approach assumes a constant saturation flow value during the green time period after the first few seconds affected by the start-up delay (Fig. 1). This means that the longer is green time the greater is the number of vehicles discharged. In addition, lost times are usually assumed to remain constant, both within and between cycles, which implies that the longer is cycle time the smaller is the proportion taken-up by the lost time, and consequently capacity is higher.

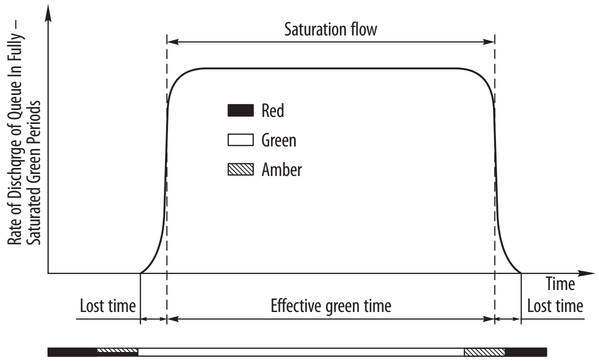

Fig. 1. An idealised model of saturation flow

While the stability of the discharge rate (and/or saturated headways) for longer green stage periods has generally been accepted and adopted world-wide for signalised intersection treatment, several researchers have investigated the constancy of saturation flow versus green time given to a traffic stream.

May and Montgomery (1986) found that there was a fall-off in saturation flow with stage length, which implies that longer cycles used by the Police in Bangkok are less likely to be efficient. This result remains descriptive and no information is provided on the range of green investigated neither on losses observed.

The authors of the Highway Capacity Manual (1985) pointed out that there were some indications that saturation flow might decrease with increasing green time. However, the question is then raised about the duration of green that may cause such an effect. As a guess, a green time period of sixty seconds or more is thought to be more probable.

A reduction in saturation flow with increasing green time was also noted by Teply *et al.* (1985) who reported that 'Canadian Research indicates that saturation flow declines after about 30 to 40 seconds of green' (Fig. 2). From this graph it seems that greens up to 60 seconds were investigated. However, to support such conclusion, no data is provided.

McDonald and Hounsell (1986) carried out observations at an oversaturated junction in Southampton

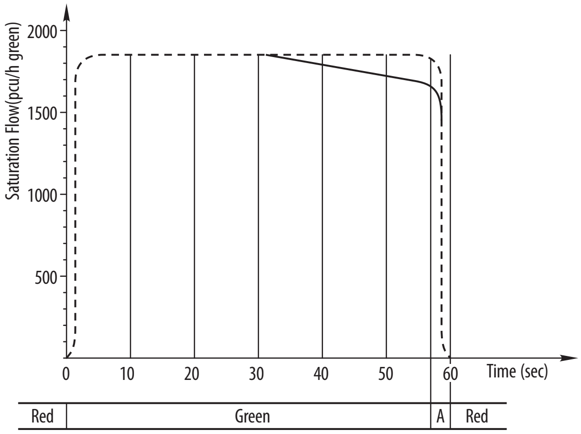

Fig. 2. The concept of saturation flow (Teply *et al.* 1985)

(UK). While cycle time setting normally used was 120 seconds with a maximum green time of 56 seconds, they varied the maximum green period from 56 to 18 seconds. For a dry road surface under day-light conditions, a steady fall of saturation flow appears in the range of 18 to 56 seconds of green time. However, the paper indicates only the correspondence between the length of the green stage as a whole and the saturation flow recorded for that specific green time. Nevertheless, we fitted linear relationship (eq. 4) to the data provided in the paper with a coefficient of determination $R^2 = 0.97$:

$$S = 2099 - 5.5\,G\ (R^2 = 0.97). \tag{4}$$

Reid (1986) reported that a number of schemes implemented to improve control performance in Nottinghamshire (UK) suggested that short cycle times gave higher saturation flows than long cycle times would. He summarised the results he had obtained on the county-wide of Nottinghamshire by the following approximate relationship (eq. 5). In this case, the obtained formula expresses only a descriptive summary of the county-wide observations:

$$S = 2099 - 5C. \tag{5}$$

Brahimi (1989) carried out field investigations into Sheffield urban area (UK) and found that discharge rate fell-off around 40 seconds after the green signal showed. Investigations were conducted using short time intervals; however, the green stages were limited to an upper limit of 50 seconds as normal greens used in Sheffield urban area did not exceed, generally, 40 seconds. This result was considered by Meehan (2003) for modelling a Microprocessor Optimised Vehicle Actuation (MOVA), an auto-adaptive control system that responds dynamically to vehicle demands.

Tseng and Lin (2005) carried out extensive work collecting field data in Taiwan and the USA to analyse discrepancies between the observed queue discharge behaviour and the saturation flow model considering vehicle position in the queue in the subject entry. In the paper titled 'Fallacies and implications of conventional saturation flow model of queue discharge at signalised intersection', they discussed the implications of the continued use of saturation flow for capacity estimation. Then, the above introduced researchers recommended an alternative for using the saturation rate of analysis on capacity for signalised intersections (Lin and Tseng 2003) to overcome the pitfalls of the conventional model. The data provided relate to the discharge rates of different positions in the queue, i.e. a multi-asynchronous analysis of saturation flow. In addition, the performed investigations seem to be limited to a maximum of 60 seconds (for a saturated headway of 2 seconds) since the maximum position in the queue was 30. Moreover, they indicate that saturation flow continues to rise with longer queues.

Nguen and Montgomery (2006) published a well documented paper relating to the observed discharge patterns at traffic signals. They concluded that the pattern of discharge rate at signalised intersections depended on a degree of saturation, traffic composition, a type of operating control and some other features. Then, the scientists considered a somewhat tedious car following model trying to explain drivers' behaviour. This paper reports that Mahalel *et al.* (1991) surveyed two signalised junctions operating on very long cycle times ranging from 263 to 351 seconds and found that the discharge rate reached its maximum after 50 to 60 seconds and then gradually declined throughout the remaining portion of green time. This is different from conventional models stating that the discharge rate attains its greatest value after the period of 6 to 10 seconds from the beginning of green time and then stays constant till the queue has disappeared. Again, in all these studies, variability depicted in saturation flow levels is not located with respect to green time but remains only descriptive.

3. Data Collection

The worldwide used current methodologies of capacity, delays, queue lengths and the level-of-service analysis of signalized intersections are based on the concept of saturation flow such as Akçelik (1981), Teply *et al.* (1995), Kimber *et al.* (1986), Petersen and Imre (1977) and Highway Capacity Manual (1985) etc. As shown in Fig. 1, saturation flow is considered to be a steady maximum rate of queue discharge after green light turns on. It is traditionally assumed that, after green onset, the discharge rate of queuing vehicles will quickly reach its saturation flow after four or five vehicles have entered the intersection and that saturation flow will be sustained until shortly after the signal change interval begins. Based on this assumed behaviour, the U. S. Highway Capacity Manual (1985) suggests that saturation flow should be determined as the average discharge rate of the queuing vehicles after the fourth queuing vehicle enters the intersection. The saturation flow model presented in Fig. 1 makes it simple to estimate the capacity of a lane or lane group at a signalized intersection. However, as mentioned above, the validity of the model is questionable.

To determine the ability of an intersection to handle traffic, it is essential to be able to reliably evaluate saturation flow on each of the approaches. Capacity evaluation is of a great importance for correctly setting traffic signals to avoid unnecessary delays. On the light of the above, experimental work was carried out to investigate the effect of signal settings on saturation flow. The overall purpose was to enable the performance of traffic signal-controlled intersections to be compared at different cycles and green time durations.

The choice of intersections, planning observations and the way investigations were conducted took into account all remarks of the previous works, particularly causes raised by Nguen and Montgomery (2006), to explain variability in discharge patterns. After a detailed survey, four junctions were identified to meet these requirements that were likely to be fully saturated for a suitable period of time when operating on relatively lengthy cycle times and where pedestrian movements were not important. In addition, selection was made upon the basis of:

- the possibility of a suitable camera position, including seeing the signals from that position,
- avoiding interference effects from turning movements (especially the left-turn for the right-hand rule on the road),
- the duration of the saturated period,
- avoiding the tendency for vehicles to pull alongside each other when queuing during the saturated period,
- allowance for alterations to traffic signal settings without affecting too much normal junction operation.

From a test record aimed at investigating traffic features and listing verbal comments that had to be made to facilitate data abstraction and analysis in the laboratory, it was found that the coefficient of variation in saturation flows between cycles was about 10%. Assuming that discharge rates over cycles are normally distributed, which can be reasonably expected in situations involving human beings, the minimum sample size required has been determined for 5% error on the mean at 95% level of confidence to be more than 16 readings at the minimum. However, the higher numbers of readings are required in case of green times around 30 seconds (Branston and Gipps 1981) so, a larger sample size of (say) 25 readings was expected to give correct estimates.

Because traffic flow levels may decrease on holidays, the survey timetable excluded observations during school holidays to avoid any changes in the traffic pattern. While traffic signals were normally vehicle actuated, for the purpose of this experiment, alterations were made to each controller to make them operate on a fixed cycle basis with given settings.

At site n°1, where the settings were randomly implemented during observations, it was felt that drivers seemed to slow down towards the end of the normally expected green time period when actual setting was longer than this before accelerating again and clearing the intersection. For other sites, the survey timetable was designed so that drivers became accustomed to signal timings to allow stable conditions to be reached. An adaptation period was therefore introduced to get drivers accustomed to each different signal setting.

For comparison purposes and to avoid any unusual behaviour and/or interference, data on daylight conditions and during the same peak period for each site was collected. Different signal settings (operating on a two-stage fixed-cycle basis) were implemented. The signals were first set at the highest possible green time (as McDonald and Hounsell (1986) in Southampton did) but then, changed to lower desired values at site n°1. Green times were taken so as to keep the ratio of greens for any cycle approximately constant and equal to that in normal use under heavy traffic.

As the overall objective of work was to look at whether or not any variation in saturation flow with varying signal settings could be noticed and, if any, to determine the interval of time from which the step could be observed, a 6-second interval counting method (Webster 1963) was used recording each type of the vehicle then converting them to passenger car units (Brahimi and Ashworth 1987).

Only fully saturated cycles free from any impedance whatsoever were identified for final analysis. This means that all cycles where:

- there was not a continuous queue;
- there were gaps due to diverting or merging vehicles;
- if traffic was impeded by a stalled car or a driver being slow,

were rejected to remove any explicable form of decline in the discharge rate and focus on eventual fall-offs due to lengthy green periods only. In total, the survey consisted of 30 records of observations at least one hour period each.

At site n°1, five settings were implemented varying green time from 30 to 51 seconds while green time under normal operations was 42 seconds. In addition, only one survey was carried out for each signal setting (Table 1). For site n°2, we concentrated on about 30–40 seconds length of green and four settings were used with green times varying from 28 to 44 seconds surveying twice each setting (Table 2). At site n°3, the weather conditions limited us to only one single setting with 44 seconds which is normal green time under normal operation (Table 3).

However, the analysis of the variance test indicated that differences in means computed for sites n°1 and 2 were not statistically significant at 95% level of confidence. This suggested that the green period should be longer and far away from the values considered at sites n°1, 2 and 3. Therefore, observations were carried out at a saturated intersection located in the outer fringe of the urban area that allowed changes in signal timing to be made over many weeks during the survey period without affecting normal junction operation at other times (site n°4). Thirteen records were completed over the period of more than three months. Three settings were implemented with the greens of 35, 58 and 90 seconds while normal setting was 60 seconds for cycle time with green running up to 35 seconds under v.a operations (Table 4).

4. Data Analyses and Results

Work was aimed not only at estimating cycle capacity but also at checking variations, if any, in the discharge rate within the cycle. To meet this requirement, it was necessary to analyse each cycle in successive short intervals as mentioned above. Discharge rates both per cycle and per 6-second intervals were computed. For comparison purposes and since the number of vehicles discharging per cycle varied with green time lengths, they were converted to saturation flows per cycle excluding the first and last intervals to remove the effect of the start-up and end losses. In the case of different surveys on the same signal settings, statistical tests were performed to check whether or not data could be aggregated

The above completed average saturation flows were calculated for each signal settings and for each site separately to avoid any site-to-site variations. The obtained results are summarised in Tables 1 to 4. Linear regression relationships were fitted to the observed data which showed that there was an apparent decrease in mean saturation flow levels with increasing green time from 2.32 pcu/gh to 4.73 pcu/gh per second of green (eq. 6 to 8). Whilst a good correlation exists between saturation flow and green time for sites n1 and n4, there is a lack

Table 1. Data summary of site n°1

		Discharge rates in pcus 6-second intervals										
Survey	**Green / Cycle(s)**	**0–6**	**6–12**	**12–18**	**18–24**	**24–30**	**30–36**	**36–42**	**42–48**	**>48**	**Sat. Flow pcu/hg**	**Stan. Dev. pcu/hg**
1	51/90	2.46	3.11	3.03	2.69	3.05	3.43	2.87	2.67	2.54	1787	173
2	48/84	2.82	2.86	3.38	3.11	3.12	2.87	2.88	2.53	1.06	1777	129
3	42/76	2.63	2.97	3.04	3.14	3.17	2.95	2.66	0.71		1793	177
4	40/73	2.22	2.91	2.90	3.01	3.11	3.14	2.94	0.92		1801	202
5	30/56	2.12	2.96	2.91	3.24	3.09	1.42				1830	189
All Surveys and Settings	**Mean**	**2.36**	**2.96**	**3.00**	**3.06**	**3.10**	**3.12**	**2.85**	**2.61**	**2.54**	**1800**	**120**
	Var.	**0.63**	**0.52**	**0.69**	**0.43**	**0.63**	**0.60**	**0.98**	**1.06**	**0.88**	**–**	**–**

Table 2. Data summary for site n°2

		Discharge rates in pcus 6-second intervals									
Survey	**Green / Cycle(s)**	**0–6**	**6–12**	**12–18**	**18–24**	**24–30**	**30–36**	**36–42**	**42–44**	**Sat. Flow pcu/hg**	**Stan. Dev. pcu/hg**
1	44/75	2.38	3.37	3.17	3.15	3.16	3.09	3.33	2.18	1927	67
2	44/75	2.56	3.50	3.11	3.10	3.16	3.17	3.33	2.43	1937	94
All Obs	**44/75**	**2.47**	**3.43**	**3.14**	**3.12**	**3.16**	**3.13**	**3.33**	**2.30**	**1932**	**184**
3	36/63	2.56	3.31	3.03	3.29	3.44	3.14	1.32		1945	96
4	36/63	2.28	3.04	3.16	2.96	3.20	3.01	1.38		1844	61
All Obs	**36/63**	**2.42**	**3.18**	**3.10**	**3.12**	**3.32**	**3.07**	**1.35**		**1877**	**162**
5	32/59	2.45	3.09	3.15	3.19	3.58	2.59			1953	159
6	28/51	2.55	3.16	3.32	3.19	3.44	0.24			1966	208
All Surveys and Settings	**Mean**	**2.43**	**3.21**	**3.19**	**3.14**	**3.31**	**3.09**	**3.33**		**1919**	**90**
	Var.	**0.35**	**0.54**	**0.56**	**0.69**	**0.73**	**0.59**	**0.72**		**–**	**–**

Table 3. Data summary of site n°3

			Discharge rates in pcus 6-second intervals									
Road Surface	**Green / Cycle(s)**		**0–6**	**6–12**	**12–18**	**18–24**	**24–30**	**30–36**	**36–42**	**42–44**	**Sat. Flow pcu/hg**	**Stan. Dev. pcu/hg**
Wet + Rain	44/81	Mean	2.33	2.85	3.07	2.84	3.08	2.83	3.06	0.78	1772	150
		Var.	0.24	0.65	0.62	0.81	0.51	0.50	0.70		–	–
Wet	44/81	Mean	2.35	3.19	3.20	3.22	3.10	3.21	3.06	1.27	1896	148
		Var.	0.35	0.42	0.52	0.45	0.57	0.77	0.51		–	–

Table 4. Data summary of site n°4

Discharge rates in pcus 6-second intervals																	
Survey	**Green / Cycle(s)**	**0–6**	**6–12**	**12–18**	**18–24**	**24–30**	**30–36**	**36–42**	**42–48**	**48–54**	**54–60**	**60–66**	**66–72**	**72–78**	**78–84**	**Sat. Flow pcu/hg**	**Stan. Dev. pcu/hg**
1	80/120	2.64	2.95	3.66	3.27	3.32	3.08	2.99	2.66	2.56	2.91	2.81	2.90	2.75	2.05	1793	185
2	80/120	3.19	3.51	3.52	3.45	3.84	3.27	2.92	3.56	3.06	2.87	2.82	2.99	3.37	2.23	1959	196
3	80/120	3.07	3.08	3.19	3.11	3.35	3.22	2.89	2.78	3.20	2.50	2.74	2.69	2.73	2.22	1774	161
4	80/120	2.73	3.40	2.87	3.57	2.64	2.97	2.84	2.88	2.90	2.60	2.73	2.35	2.68	2.09	1722	201
All Obs	**80/120**	**2.91**	**3.24**	**3.31**	**3.35**	**3.29**	**3.14**	**2.91**	**2.97**	**2.93**	**2.72**	**2.78**	**2.73**	**2.88**	**2.15**	**1791**	**202**
5	58/90	3.04	3.48	3.35	3.38	3.27	2.92	3.03	2.72	2.92	2.14	1.38				1880	163
6	58/90	2.80	3.15	3.27	3.17	3.30	3.02	2.73	2.98	2.93	2.00	1.13				1841	115
7	58/90	2.86	3.42	3.12	3.41	3.40	3.20	2.69	3.50	2.62	2.09	1.43				1902	205
8	58/90	2.92	3.38	3.68	3.63	3.40	3.13	3.18	2.90	2.98	1.66	1.20				1971	172
9	58/90	2.78	3.11	3.41	3.62	3.12	3.55	2.83	2.80	3.06	1.59	1.48				1913	186
All Obs	**58/90**	**2.88**	**3.31**	**3.37**	**3.44**	**3.30**	**3.16**	**2.89**	**2.98**	**2.90**	**1.90**	**1.32**				**1909**	**168**
10	35/60	2.78	3.42	3.18	3.57	3.15	2.52	1.60								1999	120
11	35/60	2.94	3.36	3.46	3.40	3.42	2.40	1.59								2046	272
12	35/60	3.19	3.10	2.92	3.43	3.84	2.38	1.70								1994	243
13	35/60	2.79	3.09	3.48	3.23	3.33	2.59	1.15								1970	99
All Obs	**35/60**	**2.93**	**3.24**	**3.26**	**3.41**	**3.44**	**2.47**	**1.51**								**2004**	**244**
All Surveys and Settings	**Mean**	**2.90**	**3.27**	**3.32**	**3.40**	**3.34**	**3.15**	**2.90**								**1876**	**202**
	St. Error	**0.03**	**0.04**	**0.07**	**0.03**	**0.09**	**0.04**	**0.02**								–	–

of correlation for site n 2 where green stage length does not exceed 44 seconds.

$$S = 1896 - 2.32G \text{ (Site n°1 } R^2 = 0.88), \quad (6)$$

$$S = 2021 - 2.54G \text{ (Site n°2 } R^2 = 0.20), \quad (7)$$

$$S = 2174 - 4.73G \text{ (Site n°4 } R^2 = 0.99). \quad (8)$$

As indicated earlier, the analysis of variance showed that differences in mean saturation flows were not statistically significant for sites n°1 and 2. However, for site n°4, the results (Table 4 and Fig. 6) do indicate a decrease in mean saturation flows of 4.73 pcus/gh per second of green. Differences in means are statistically significant at 95% level of confidence when the intervals of 30–36 seconds are removed in computing saturation flows for the experiment of 60/35 (cycle/green) seconds.

The analysis of variations within each interval of 6-seconds from one cycle settings to another showed that differences in mean discharge rates were not statistically significant at 95% level of confidence which suggested that data might be aggregated per 6-second interval for all observations carried out at the same site. Having completed this procedure, further statistical analyses were performed comparing discharge rates for each consecutive pair of 6-second intervals. The results are presented in Figs 3 to 6 indicating the data that has been grouped (solid line) in those cases where consecutive intervals showed no significant differences in mean discharge rate at 95% level of confidence.

The graphs and tables disclose that a reduction in the discharge rate of about 10% (site n°1) to 13% (site n°4) is found in the ranges of 36–48 seconds and 36–78 seconds respectively compared with that related to the range of 6–36 seconds. However, this is obscured when only stages of less than (say) 45 seconds are taken into account such as at sites 2 and 3. These results are consistent with the findings of other researchers reported in literature. It is worth noting that significance is obscured when green intervals including a part of the amber stage are considered.

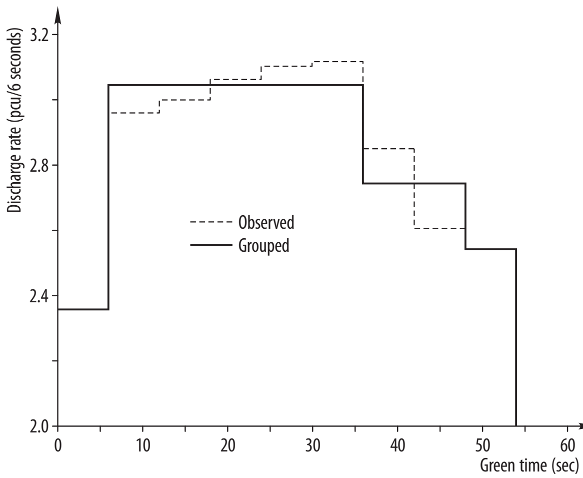

Fig. 3. Discharge rate for site n°1 per 6-second interval

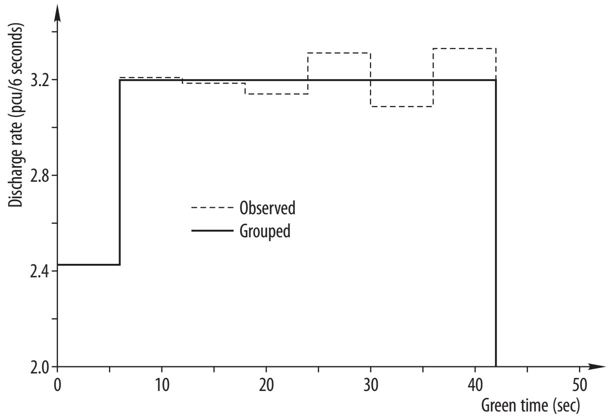

Fig. 4. Discharge rate for site n°2 per 6-second interval

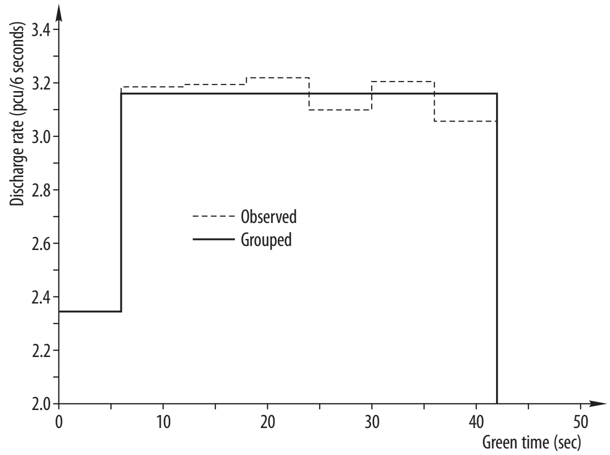

Fig. 5. Discharge rate for site n°3 per 6-second interval

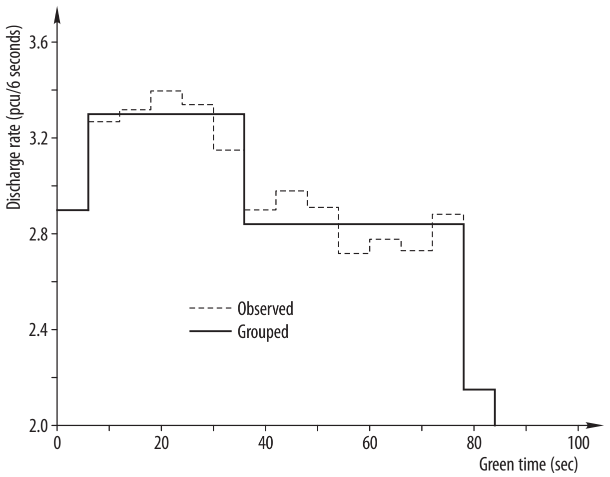

Fig. 6. Discharge rate for site n°4 per 6-second interval

5. Conclusions

Significant discrepancies between the saturation flow of the idealized model and the observed characteristics of discharge rates may have significant implications on the performance of signalized intersections. Saturation flow and hence capacity estimates are used in the majority of manuals for guidelines on signalized intersections to estimate delays, queue lengths, levels of service, fuel consumptions, air pollution etc. relating to the operation of signalized intersections. The level of the service concept, which is a resulting compound of the governing features of intersection vicinity, is the basis for the decision making process relating to planning, design and operating concerning those signalized intersections. In other words, even small errors in saturation flow/capacity estimates may have negative consequences on the resulting output of the decision making process.

The results obtained from experimental investigation do indicate a significant reduction (10–13%) in flow levels from 36 seconds onwards compared with those related to the range of 6–36 seconds for aggregated data and suggest that the performance of a signal controlled intersection is likely to decrease with longer green time stages. As the length of the green stage increases, the loss of capacity becomes greater than gain due to decreasing the proportion of a cycle taken-up by the lost time. In terms of capacity, reduction may reach a proportion as high as 10% depending on the length of the green stage. However, a decline in saturation flow levels has not been detected with green time periods of 44 seconds or less. It is suggested to limit the duration of the green time stage to an upper limit of 45 seconds wherever possible. A lower limit should not be less than (say) 20 seconds as capacity may be substantially reduced for short green time stages because of the proportion of cycle time taken-up by the lost time. It seems worthwhile that more research will be undertaken on the subject for a better understanding of traffic signal-controlled intersection capacity because significant discrepancies with respect to the idealized model of saturation flow may have significant implications on the performance of signalized intersections.

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