A Validation Study on the Use of Traffic Actuated Signal Control and HCM-based Performance Evaluation Procedures in Brazil

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Abstract. The paper contrasts current views about the effectiveness of actuated signal control in urban traffic, based on a field study carried-out in Brazil, and evaluates the viability of analysis methods based on HCM performance models in predicting signal times and vehicle delays for deciding about its implementation. We argue that there is an old view about the use of actuated control on practical warrants (as cure for light-volume traffic signals only) in contrast to a theoretical belief on the performance superiority of actuated control (stated generically). The field study investigates the comparative performance of pre-timed and actuated control in a heavy traffic urban intersection and evaluates field measures of delay against HCM-based model estimates (a basic tool for establishing a clear criteria grounded on cost-benefit analysis). Our field study shows that potential gains from actuated control are not easy to realize, depends on good parameter setting for micro-regulation efficiency and, most of all, on the importance of incidents and trends that ask for an automatic macro-regulation capability. Even for evaluating micro-regulation gains, HCM-based performance models give biased results about traffic actuated operation, commanding care on their use for assessing the real (relative) effectiveness of traffic actuated control on the field, not to say about parameter settings. We conclude with the identification of some research themes that can contribute in establishing better methods.

TWO VIEWS ABOUT THE USE OF TRAFFIC ACTUATED SIGNALS

The use of traffic actuated signals is widespread, mainly in the USA and Australia, but current views about its place in traffic control systems seems to lag behind actual practice. A clear understanding about our point can be grasped if one compares generic/theoretical views and specific/practical guidelines.

The vision in MUTCD/1988 or MUTCD/2000, instead, is clearly more restricted in its recommendations for use of actuation. For example, MUTCD/1988 criteria about traffic signal installation and the use of actuated traffic signals only mentions features that justify the attention to actuation in traffic signals decided based on:

- minimum pedestrian volumes (warrant 3),
- school crossing (warrant 4) and
- accident records (warrant 6),

then restricting the use of actuation to situations were vehicular volumes do not justify the traffic signal installation per se. Even this restricted vision is avoided in the MUTCD/2000.

So the current view on academic and also most professionals areas are far from these guidelines as it is usually believed that traffic actuated signals can always deliver better performance, if properly set, and its use should be decided based on the cost of installation, that must be weighted against operational improvements.

For example, HOMBURGER *et alli*, 1992, views are set on behalf of a believe on the superiority of the traffic performance of actuated signals on most situations, against their greater cost (detectors and perhaps controller).

As another example, one can mention that HCM-based recent delay models clearly argue a better performance of traffic-actuated signals (at least where the coordination of signals has minor influence or it is possible to reach similar quality of progression with semi-actuated signals).

The lack of a clear statement of this judgment and the lack of a documented demonstration about this superiority is, nevertheless, a reason for the subsistence of the old view between some professionals.

This state-of-affairs is even worse in countries with less tradition in the use of traffic actuated signals, as Brazil, and where there is a lack of demonstration effects of other installations to inspire practitioners.

Based in the view set above, we selected a heavy traffic signalized intersection as object of study, aiming at:
- analyze the final performance of traffic actuated and pre-timed operations;
- evaluate the validity of HCM-based performance models as analysis tools.

In the following, we describe the site characteristics and the results of field measurement of vehicle delays for pre-timed and traffic actuated operation in the field and discusses its relation to timing criteria and intrinsic features of each control. Then, we compare the field signal times and delay measures with predictions based on the performance techniques recommended in HCM/2000 (and its previous version HCM/1997).

These are the results we used to evaluate current views and methods and to identify themes for future research in the last section.

THE FIELD EVALUATION OF ACTUATED SIGNALS OPERATION IN A HEAVY TRAFFIC SIGNALIZED INTERSECTION

The selected site can be described as an isolated signalized intersection with high vehicular traffic volume that generate overflow conditions during peak periods (morning and specially afternoon peaks) and is located in Campinas (a city with one million inhabitants and half a million fleet, in the State of São Paulo, Brazil).

As illustrated in Figure 1, it is a T intersection between a main arterial road (Av. Waldemar Paschoal=WP) and a secondary arterial road (Av. Mal.Carmona=MC). After the intersection, driving from the center of the City to its Fringe, the main arterial road change its name (to Av. Monsenhor.João Ladeira=JL) and there are some minor roads that diverges from the main arterial road and carry small volumes. The pedestrian movements are small but present. Left turns are absent (they are forbidden locally and rerouted through the area). As can be seen from Figure 1, pedestrian movements are unopposed and a simple two-phase signal plan can be used.

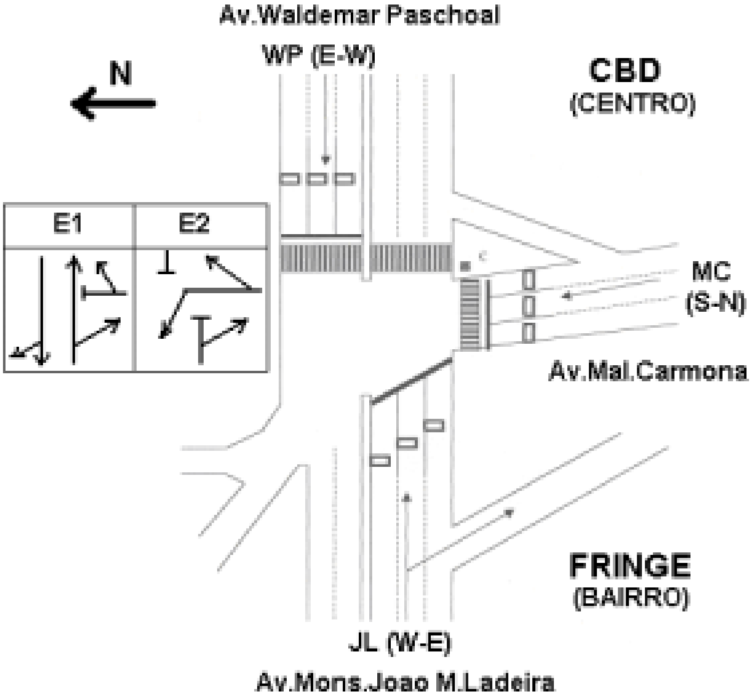

Figure 1
Intersection between a main arterial road (Av. Waldemar Paschoal)
and a secondary arterial road (Av. Mal.Carmona)

Unusual commuting patterns influence traffic flows. So, the critical movements for phase E1 are on the outbound approach (on the WP approach) for the morning peak and also for the afternoon peak (flow on the inbound approach, the JL approach, is never binding). Phase 2 carries only the flow on the side road (on the MC approach). Data on vehicular demand and saturation flow are summarized in Table 1.

Approach	WP (E – W)		JL (W – E)		MC (S – N)	
Period	Q (v/h)	S (v/h)	Q (v/h)	S (v/h)	Q (v/h)	S (v/h)
Morning Peak: 06:30 to 09:00 hs 09:00 to 11:00 hs	2769 2237	4404	2100 1619	4572	976 812	3900
Lunch Peak: 11:00 to 14:30 hs 14:30 to 16:00 hs	2529 1986	5199	1486 1252	4914	947 800	3840
Afternoon Peak: 16:00 to 20:00 hs 20:00 to 21:00 hs 21:00 to 00:00 hs	3435 1370 1170	5112	2215 1262 953	4794	1051 463 713	3906

Table 1 – Vehicular Demands (Q) and Saturation Flows (S) in the Peak and Following Periods.

The field evaluation study on the performance of actuated control carried-out three tasks:
A. Data Gathering and Pre-timed Control Checking:
- the actual pre-timed plan was checked up;

B. Parameters Setting and Checking for Actuated Control:

- the parameters for actuated control were calculated;
- the parameters for actuated control were checked and adjusted;

C. Performance Monitoring for Actuated Control and Pre-timed Control:
- the delay under actuated control was measured;
- the delay under pre-timed control was measured.

Results from each of these tasks are briefly presented in the following. Some tips for interpretation of results are advanced as they are recovered in the concluding section.

A. Data Gathering and Pre-timed Control Checking

Current pre-timed settings for a weekday are displayed in Table 2. As can be seen, there are ten plans operating on a weekday with maximum cycle time being used for the afternoon peak only. Current policies for traffic control include transition plan (pre-peak and post-peak) for each peak period, intended for providing smooth transition and dealing with unpredicted fluctuations on congestion levels. So, the demand patterns display smaller differences than signal settings (that includes transition plans). Field adjustment of timings is also an usual practice and enforces the stability of traffic conditions on the main road.

Plan intervals	Cycle time (s)	Green E1 (s)	Green E2 (s)
05:00 to 06:30 hs	60	33	17
06:30 to 09:00 hs	85	53	22
09:00 to 11:00 hs	60	33	17
11:00 to 14:30 hs	70	40	20
14:30 to 16:00 hs	65	33	22
16:00 to 16:45 hs	90	56	24
16:45 to 19:30 hs	105	70	25
19:30 to 20:00 hs	90	51	29
20:00 to 21:00 hs	60	33	17
21:00 to 00:00 hs	55	30	15

Table 2 - Current Pre-timed Control Plans for a Weekday (from EMDEC).

Revised timings were calculated from current policies that can be summarized as:
- apply Webster optimal cycle time for hourly demand if optimal cycle time is less than maximum cycle time;
- if not, apply a desired peak flow to capacity ratio of 95% for main road (and secondary road, if possible).

Intergreens (with a yellow interval of 3 seconds and an all-red interval of 2 seconds in both phase changes) were checked and kept as used in the current timings.

The revision determined similar timings. Taking into account the current policies (the use of transition plans and the adjustment for main road stability), the current plan was considered adequate and kept as done.

B. Parameters Setting and Checking for Actuated Control

Current recommendations for parameters setting of actuated signals used in Brazil are similar to the international practice as applied to simpler controller types (Volume-Density or Waste

Change methods of control are not implemented on usually available controllers). Brazilian controllers have single ring architecture with some resources for optional phases and coordinated/actuated operation (under semi-actuated control only, as usual). Unit interval (as gap) and green extension are equal on all controllers (also, no delayed green parameter is available for implementing the same functionality).

Under these constraints, usual parameters are restricted to minimum and maximum green and the unit extension (UE as green extension and unit interval) for basic actuation. Each actuated phase is associated with only one detection port that can be set in one of some limited ways:

- linked to the usual critical lane detector;
- linked to a overall lane group detector, or
- linked to a series of lane detectors in a section (for one lane group, one approach or on both opposed lane groups or approaches).

The group wide and series on group configurations are equivalent and are the most usual layout in Brazil. In the example intersection, the approach wide and group wide layouts are also equivalent (as there are one group per approach reaching the stop line in all cases).

When there are more than one lane group running in the same phase, the identification of the critical lane group can be varied for each timing plan but must be set by the signal plan (based on usual commuting patterns).

So, only data on the critical lane group is used and the saturation flow relevant for gap setting is the overall lane group (or approach) saturation flow instead of critical lane saturation flow.

As usual in Brazil also, the detectors were located near to the stop line (at 10 meters) and no advance detectors were used.

Four set of parameters for actuated control were selected for field checking and adjustment (if needed). Timing were similar to usual practices in Brazil (as reported in Vilanova, 1990, for example).

In all of them, maximum green time was set high at the green values calculated for pre-timed control (under conventional policies, as previously described) for a 25% demand overload.

Then, two options for gap setting and two options for minimum green time were calculated. Options for minimum green are based on pedestrian crossing times (A, B) or 75% of the green needed for discharging queues at the minimum cycle time (C, D), when greater than the previous value.

For A and B timing options we used

$$g_{min} = g_p \text{ with } g_p = \delta_p + \frac{L_w}{V_p} - I_a$$

where L_w is the parallel approach width, V_p is pedestrian speed, δ_p is the pedestrian start-up time (4 seconds) and I_a is the yellow time interval (3 seconds also).

For C and D timing options we used

$$g_{min} = \max\{g_p;0,75.g_v\} \text{ with } g_v = \frac{y.T_p}{1-Y_c}$$

where $y = \frac{Q}{S}$ is the critical lane group flow ratio of the phase (Q is demand flow and S is saturation flow), T_p is the overall signal lost time (sum of starting and ending lost time and all-red for each phase chance on the cycle), Y_c is the overall signal critical flow rate (sum of the critical flow ratios for each phase running on the cycle).

Gap setting for unit (green) extension is based on a 5% (for A,D) or 10% (for B,C) probability of failure (premature green cut) for the queue discharging headway (mean queue saturation headway as the inverse of saturation flow), calculated using a poissonian assumption with a correction for under-capacity conditions.

The unit extension (as gap for presence detection) was evaluated with (as usual in Brazil):

$$UE = \kappa.H_c - \frac{\ell_v + \ell_d}{V_a/3,6} \text{ with } \kappa = \begin{cases} \frac{0,90}{Y_c}, Y_c < 0,90 \\ 1,00, Y_c \geq 0,90 \end{cases}$$

where ℓ_v is the representative vehicle length (6,0 m), ℓ_d is the detector length (1,8 m), V_a is the approach speed with queue discharge (40 km/h) and H_c is the maximum allowable headway, calculated as

$$H_c = -\frac{\ell n[0,05]}{S/3600} \text{ for A and D (5\% probability of failure)}$$

and

$$H_c = -\frac{\ell n[0,10]}{S/3600} \text{ for B and C (10\% probability of failure)}$$

where S is the critical lane group (approach) saturation flow (in v/h). This formula is based on the exponential distribution of headways (implicit in the poissonian distribution of arrivals).

In all cases, maximum green times for each phase were calculated as

$$g_{max} = \frac{y_{+25}}{Y_{c+25}}.(T_{c+25f} - T_p)$$

where T_{cf} is the pre-timed cycle time for the period ($_{+25}$ reminds the 25% demand overload).

Table 3 displays the minimum green values, the calculated unit interval values and the maximum green values. As can be seen, no transitional plans were included as the need for adjusting settings to variations in the duration of peaks and their demand can be attributed to the actuated control.

Plans	Phase	Min Green A/B (s)	Min Green C/D (s)	Unit Extension A/D (s)	Unit Extension B/C (s)	Max Green (s)
06:30 to 09:00 hs	E1 E2	12 20	52 21	1,7 2,1	1,2 1,5	91 34
09:00 to 11:00 hs	E1 E2	12 20	28 20	2,4 2,8	1,3 1,9	55 25
11:00 to 14:30 hs	E1 E2	12 20	30 20	1,9 2,7	1,7 2,0	63 27
14:30 to 16:00 hs	E1 E2	12 20	14 20	2,5 3,5	1,7 2,5	33 25
16:00 to 20:00 hs	E1 E2	12 20	55 22	1,4 2,4	0,9 1,4	93 33
20:00 to 21:00 hs	E1 E2	12 20	12 20	4,2 5,7	3,1 4,2	35 25
21:00 to 00:00 hs	E1 E2	12 20	12 20	4,0 5,4	2,9 4,0	28 22

Table 3 – Calculated Parameters for Actuated Control Plans (Min Green, UE and Max Green values).

A field checking procedure is applied for detecting excessive premature green cuts, excessive unsaturated green, excessive minimum or maximum green restraints, adequate pedestrian crossing times, adequate approach capacities and so on.

In this case, the unit extension for Phase 1 and afternoon peak (the most heavily used case) had to be adjusted in view of excessive premature green cut. The critical lane group for this phase has a significant proportion of heavy vehicle (including urban buses that carry high passenger load in the peaks) and runs on a upgrade.

Even noting the special conditions of this approach, one should notes the general under-estimation of the critical headway value with the Poisson assumption. So, the proportion of premature green cut manifests itself due to the lack of correction for under-capacity conditions (that compensates for the noted under-estimation for non-saturated periods) in the afternoon peak. A unit interval of 2,4 seconds for E1 was used after adjustment.

A feeling on the under-estimation can be grasped from data collected during the field checking. One can see that, using the estimated (and adjusted) unit intervals, premature green cuts with smaller minimum green values ranged from 30% to 40% for option B (design value is 10% before correction and adjustment) and 10% for option A (design value is 5% before correction and adjustment). Operation with greater minimum green, even with liberal gap setting, reduces premature green cuts but increases unsaturated green to a significant extent.

The use of alternative distribution for headways would ask for some additional assumptions. For example, the usual option of a cowanian distribution, instead of the exponential distribution, would ask for platooning ratio and minimum headway model parameters. Note, however, that there is a theoretical inconsistency between the supposition of a constant (deterministic) minimum headway within the platoon (that would be associated with the queue discharging operation) and its use for developing a (probabilistic) criteria for setting the unit extension based on the detection of the end of the queue discharging (the platoon itself).

A mixed distribution with two streams (light and heavy vehicles) in known proportion could be easily used, as the traffic composition is a usually recorded data. Although delivering a nonlinear equation for calculation of H_c, it is easily solved and gives very similar results, at least for normal traffic composition observed in urban roads (this was checked in the case study). Off course, more general models (for example, with four streams of inter-vehicle patterns LL, LH, HH, HL of light (L) and heavy (H) vehicle and random proportion and/or sequence of arrival types) could also be used but were not developed or tested against field data in this study.

The proportion of minimum greens and maximum greens were judged to be satisfactory for field operation, noting that A and B options deliver less constrained operation (as minimum greens determined by pedestrians only are smaller).

During the monitoring procedure used with the field checking routine, it was possible to certifies that the actuated signal was able to handle demand and capacity variations, including some small incidents (illegal parking on the approach and lane block due to on the road police work). Also, traffic personnel reported a subjective feeling of better operation (that they described as smaller queues and smaller peak duration).

Based on the qualitative results of monitoring and field checking, case A was selected for further performance evaluation, whose results are reported in the following sections.

C. Performance Monitoring for Actuated Control and Pre-timed Control

After deciding on a practicable parameter setting for actuated control, performance monitoring was made through the measurement of delay. As there was the intention to check HCM performance models for signal times and delay estimates also, measurement followed HCM recommendations (appendix 3 of chapter 9 in HCM/1997 or appendix A of chapter 16 in HCM/2000).

For eliminating the effect of field adjustment on parameter setting, monitoring avoided the afternoon peak and was done in two peak periods (morning and midday) and the off peak period between them. In both cases (actuated and pre-timed control), a set of 4 to 5 measurements were taken by switching from one control type to the other during the same period (separated by 30 minutes or more to have traffic normalization), with at least two distinct periods for each control type. Measures were spread in two different days for every phase.

Results gathered from pre-timed control and actuated control are summarized in Table 4. All measurements were undertaken in regular operation (ie without incidents) for the critical lane group of each phase (all lanes).

Approach	Measure	Morning Peak	Off Peak	Lunch Peak
WP (E – W)	Pre-timed delay (s/v)	5,97 ± 0,95	7,59 ± 1,18	9,47 ± 3,76
	Actuated delay (s/v)	10,63 ± 3,20	8,78 ± 2,05	9,04 ± 0,57
	Change	+78,06 %	+15,74 %	-6,26 %
MC (S – N)	Pre-timed delay (s/v)	32,25 ± 5,97	16,68 ± 2,28	22,79 ± 6,05
	Actuated delay (s/v)	31,46 ± 12,22	18,92 ± 4,36	16,88 ± 3,45
	Change	-2,76 %	+13,37 %	-25,94 %

Table 4 – Field Measurements of Pre-timed and Actuated Control Average Delay and Standard Error.

As can be seen, the performance of actuated control is similar to the performance of pre-timed control, under regular operation. There is a clear bias in the relative values as one can note that the periods in which the actuated control delivers better performance also display greater variation in the pre-timed control.

One can also note the predominance of delay increases for the main road and delay decreases for the secondary road.

This pattern of results can be related to the traffic management policies applied by the municipal agency, that preserves traffic operation on the main road, especially under saturated conditions. So actuated control, in some sense, removed the prioritization for the main road that exists in the current plan of pre-timed control.

Off course, this same kind of policy can be adopted under actuated control by varying criteria for minimum and maximum green and for the probability of failure by premature green cut in a way that favors the main road. In the same way, transitional plans can be used if felt needed.

This is a clear justification for careful timing and checking of actuated signals, for warranting comparable performance under regular operation, for heavy traffic intersection.

Given field results, even considering regular operation only, actuated control was clearly practicable and competitive for a heavy traffic intersection but its performance is not always better than pre-timed control. For light traffic intersections, the superiority of actuated control can was suspect to be more easily achieved.

Nevertheless, as can be seen in field measurement also, benefits from micro-regulation (the adjustment of signal times between cycles for regular operation) are not enough for affirming the superiority of actuated control. This could be attributed to the weak structure of actuated control algorithms (at least to its quasi-myopic feature of deciding on a phase without considering all the movements) and asks for fully adaptative algorithms

The need for evaluating the benefits from macro-regulation, avoiding aging of plans and disruption of operation due to incidents, is a point that will be recovered in the following discussion.

Another point to note is the lack of results on delay to pedestrians. As the actuated control delivered shorter cycle times, it should favor pedestrians in most of the occasions. These effects should also be taken into consideration is full cost-benefit calculation.

From an analytical point of view, this result stresses the importance of an improved timing criteria and careful evaluation of performance for getting a fairer view on the effect of applying actuated control. For this task, current techniques are new and incomplete. The next section discusses the use of HCM based performance models for evaluating actuated control under regular operation and their extension.

THE EVALUATION OF HCM-BASED PERFORMANCE MODELS FOR ACTUATED SIGNAL TIMING AND VEHICLE DELAYS

Current methods for analyzing the operation of traffic signals incorporated several improvements made in theory during last decades and are now capable of proposing techniques that can forecast the differential impact of the use of pre-timed and actuated control for a wide range of controllers and settings.

For analyzing actuated control, performance models should be usually applied in two steps:
- average timing prediction models;
- average delay prediction models.

The best known examples of these model components are the methods included in HCM/1997 and HCM/2000, that are subject to a validation study in this work.

Using normal demand and saturation flow, traditional procedures seem to be applicable for evaluating regular operation only. This is the setting in which the HCM-performance models are evaluated.

The estimation of benefits derived from the capability of handling incidents and avoiding aging of plans, called macro-regulation capability, should be related to frequency and duration of incidents and to the trend of change in demand versus periodicity of plan revision for pre-timed control. These are clearly outside the content of the HCM proposed procedure as originally set out and were not tried in the validation study.

For sure, one can evaluates incidents and aging for some “design” cases (that will have to be detailed in a specific proposed method). Off course, such a procedure will also increase practical and methodological problems. The alternative would be embodied it into model coefficients as long as possible and sensible (a discussion that is recovered at the concluding section).

The field study for validation of HCM-based performance models for analyzing actuated control carried-out two tasks:

A. Validation of the Predictions of Average Timing under Actuated Control:
- mean signal times under actuated control were measured;

B. Validation of the Predictions of Average Delay for Actuated (and Pre-timed) Control:
- performance models were checked for actuated control;
- performance models were checked for pre-timed control.

Results from each of these tasks are briefly presented in the following. Some preliminary views on the qualitative and quantitative behavior of the HCM-based model are also advanced as they are recovered in the concluding section.

A. Validation of the Predictions of Average Timing under Actuated Control

The iterative procedure for estimating the average timing of signals under actuated control was carried-out for the three periods in which performance monitoring measurements were done.

The average timing prediction model features can be summarized as follows:

- it is an iterative procedure that evaluates all phases of a cycle simultaneously (ie, considering all phases, sequentially, in each iteration);
- average green times demanded by each actuated group are calculated as queue discharging plus green extension (for non-actuated groups, the minimum green parameter is assumed as the average green as in the following); then $\overline{g} = \overline{g}_s + \overline{g}_e$ but also $\overline{g} \geq g_{min}$ and $\overline{g} \leq g_{max}$ (that is understood as $\overline{g}_s + \overline{g}_e < g_{min} \Rightarrow \overline{g} = g_{min}$ and $\overline{g}_s + \overline{g}_e > g_{max} \Rightarrow \overline{g} = g_{max}$, without any smoothing pattern);
- queue discharging is a function of average demand and average red time as $\overline{g}_s = f_q.\dfrac{Q.r}{S-Q}$ with $f_q = 1{,}08 - 0{,}1.\left(\dfrac{g}{g_{max}}\right)^2$, given red r, demand Q, saturation flow S (assuming $q_r = q_g = Q$);
- green extension is a function of parameter setting (the unit extension as gap for presence detectors) as $\overline{g}_e = \dfrac{1}{\theta.Q}.e^{\lambda.(UE+t_o-\tau)} - \dfrac{1}{\lambda}$, with $t_o = \dfrac{\ell_d + \ell_v}{V_a}$ and $\lambda = \dfrac{\theta.Q}{1-\tau.Q}$ (for a cowanian distribution with parameters $\{\theta, \tau\}$ as proportion of free vehicles and minimum headway between vehicles; ℓ_d, ℓ_v are detector length and average vehicle length and V_a is the mean approach speed);
- average red time for a phase is recognized as the green of competing phases (plus intergreens, as usual);
- for the non-actuated phase of semi-actuated control, only g_{min} is set and the procedure recommends to use $\overline{g} = g_{max} = g_{min}$
- for optional phases g_{min} should be replaced by $(1-p_0).g_{min}$ where $p_0 = \theta.e^{-\lambda.(t_c-\tau)}$ is the probability of no arrivals in the cycle time t_c.

Despite being a large improvement over previous procedures (as HCM/1985 and HCM/1994), some theoretical critics can be advanced to this kind of method:

- the procedure seems to be applicable for regular operation periods only or some hints on the effects and duration of incidents (that change demand or saturation flow) should be assumed to estimate benefits from its capability of adjustment of timings for handling incidents;
- the procedure does not estimates the benefits of avoiding aging of plans that otherwise can not be evaluated (this should be easier but even so needed);
- some calculation are really crude as the effect of minimum and maximum green values as hard (deterministic) cuts;
- the consideration of semi-actuated operation, with or without coordination of signals on the main road, is also crude.

This last point is really intriguing as there are well-known formulas, developed or surely known by the researchers that developed the method, that can be applied for estimating the mean non-actuated phase duration, given demand and parameters of the actuated phase. There are some variations of detail in the formula suggested by several references (as LIN, 1982, or AKÇELIK, 1995) around $\overline{g}_1 = g_{min_1} + \frac{\theta_2}{\lambda_2}.e^{-\lambda_2.\left(g_{min_1}+UE_2+t_{o_2}-\tau_2\right)}$, were the subscript identifies the main as 1 secondary (actuated) as 2 for phases or lane groups.

The evaluation of coordinated operation on a corridor using semi-actuated control with the assumption that any excess green time due to the need of keeping the common cycle time set for coordinated operation goes to the main road is usual (the iterative process is responsible for splitting this excess green, due to its effect on queue dissipation on the secondary road). A practical method for dealing with insufficient green (negative excess green) is also proposed by AKÇELIK, 1995.

Nevertheless, other researchers point out to features that suggest that there are some minor points on which one can suspect the model structure even for its main subject (prediction of average timing for full-actuated control). For example, the effect of minimum and maximum green values as deterministic cuts are, in some sense, contrary to previous results gathered from simulation studies, as in BENNESON and McCOY, 1995 (in which there is a method that has the needed features, warranting a greater than minimum and smaller than maximum average green, with smooth behavior, in all range of operation).

The iterative results for average cycle time and average green times of each phase are summarized in Table 5, against field data.

Iteration	Morning Peak		Off Peak		Lunch Peak	
Phase	E 1 (green)	E 2 (green)	E 1 (green)	E 2 (green)	E 1 (green)	E 2 (green)
1	58,008	16,000	37,150	16,000	39,941	16,000
2	56,295	28,508	36,054	18,311	39,045	21,755
3	78,467	26,838	38,599	17,878	44,741	20,952
4	73,269	34 *	37,955	18,597	43,601	22,786
5	86,177	34 *	38,768	18,376	45,460	22,226
6	80,523	34 *	38,479	18,611	44,784	22,858
7	85,173	34 *	38,745	18,516	45,448	22,577
8	84,479	34 *	38,623	18,596	45,129	22,812
9	84,585	34 *			45,380	22,686
10	84,569	34 *				
11	84,572	34 *				
Phase green (s)	84,6	34 *	38,6	18,6	45,4	22,7
Cycle time (s)	128,6		67,2		78,1	
Field values (s)	47,6	20,5	37,9	22,2	30,8	22,9
	79,0		70,1		66,0	

* max green value

Table 5 – Average Green and Cycle Times using the HCM Iterative Procedure and in Field Data.

As one can see clearly, errors increase sharply with saturation and delivers unusable results for the morning peak (the same was noted for the afternoon peak, after parameter adjustment on the field). One can also easily check that the HCM procedure quickly approach the maximum settings as forecasted average green times, a behavior that is not observed on the field with the same extreme pattern.

One observes in field data that, despite the increase in the number of max-outs with traffic saturation, there are also a number of phase durations that gap-out even for demand approaching capacity.

Comparing field values with pre-timed plans (see Table 2), one can note that cycle times are shorter than pre-timed ones for peak periods but not for the off peak period between them (despite being practicable).

Data from the afternoon peak, with its heavier demand pattern, confirms the relevance of these shortcomings. The iterative procedure converged to maximum settings for both phases in the fourth iteration and delivered a maximum cycle time of 132 seconds. Nevertheless, field data measurements gave a 98,3 s cycle time (under the 105 s value of the pre-timed plan for the weekday afternoon peak).

This behavior can be attributed to random fluctuations in demand and to failures in detecting the end of queue discharging (two usually admitted phenomena in modeling the operation of traffic signals under actuated and even pre-timed control). Both phenomena seem to require improvement of the current HCM method.

These details should be weighted for importance in real settings against the observation of the traffic signals, so as to determine their practical relevance. These points are made in the following and relate to features that can improve the evaluation of the micro-regulation effect of actuated control.

B. Validation of the Predictions of Average Delay under Actuated Control

Given the availability of measurements of demand, timing and delays, a further step was carried-out to check the validity of average delay forecasted using HCM formulas.
The average delay prediction is fully based on the performance model of pre-timed control with some minor adjustments of model coefficients, using the average timing of the actuated control.

The calculations for actuated control were done with the use of average timing from field measurements (so eliminating the effect of the HCM procedure for estimating timing).

For an isolated intersection near saturation (as the one considered in this case study), HCM delay equation can be summarized (neglecting d_3 and adopting $q_r = q_g = Q$ in HCM/1997 or HCM/2000 formula) as

$d = d_1 + d_2$ where

$d_1 = \frac{(1-u)^2 . t_c}{2.(1-y)}$ is the conventional uniform delay term (t_c is the cycle time, $u = \frac{g_{ef}}{t_c}$ is the effective green ratio, given the effective green time g_{ef} approximated by $g+1$ second as usual in Brazil, and $y = \frac{Q}{S}$, given demand and saturation flow data on the lane group or approach), and

$d_2 = \frac{T_P}{4} . \left((X-1) + \sqrt{(X-1)^2 + \frac{8.k.X}{C.T_P}} \right)$ is the conventional overflow delay term (T_P is the duration of the flow period, taken to be 15 minutes in all cases, $X = \frac{Q}{C} = \frac{y}{u}$ is the demand to capacity ratio, given the capacity $C = u.S$, and k is a parameter based on the degree of saturation and type of control, given in Table 9.14 of HCM1997 or 16.3 of HCM/2000).

Note that the parameter k is the only factor that depends on type of control. The value of the parameter k, combined with the theoretical expectation of shorter cycle times and proper split, justify the academic belief on the uniformly better performance of actuated control, at least if properly settings, that was previously discussed (this can be seen by checking on the k value, that is uniformly favorable compared to pre-timed values or even to the values recommended for non-actuated phases of semi-actuated signals).

The results of this exercise are summarized in Tables 6 (comparing actuated and pre-timed control in the same format used in Table 4, with field values).

Approach	Measure	Morning Peak	Off Peak	Lunch Peak
WP (E – W)	Pre-timed delay (s/v)	29,99	17,02	14,43
	Mean Error	+402,3%	+124,2%	+57,5%
	Actuated delay (s/v)	34,68	20,05	27,38
	Mean Error	+226,2%	+128,4%	+202,9%
	Estimated Change	+15,64%	+17,80%	+89,74%
MC (S – N)	Pre-timed delay (s/v)	44,81	21,97	27,50
	Mean Error	+38,9%	+31,7%	+20,7%
	Actuated delay (s/v)	38,69	20,72	16,98
	Mean Error	+23,0%	+9,5%	+0,6%
	Estimated Change	-13,66%	-5,69%	-38,25%

Table 6 – Average Pre-timed and Actuated Control Delay using HCM Formulas and Field Values.

Despite the large errors in delay measurements, variations in performance are predicted with some agreement (at least the magnitude of the variations and the pattern of favoring the secondary arterial is clearly identified). Off course, the biased error in the procedure for forecasting the average timings of actuated control (delivering greater values) will also generate biased evaluations of performance, against actuated control.

The comparisons of delays obtained from actuated control using model based estimates of delays using field measurements timing simulates a “planning” framework with an improved procedure for estimating average timing under actuated control and displays residual errors from the delay model that can not be judged to be of significance if compared to the identified need of considering macro-regulation gains previously noted.

Nevertheless, a comparison using estimated timing with current methods in the “planning” exercise seems to deliver unusable results for evaluating the potential performance of actuated control.

So, the proposal of improved methods for forecasting average timing of operation of heavy traffic approaches under actuated control seems to be a major effort to be undertaken.

CONCLUSIONS

In conclusion, this paper stresses four points:

- there are two views on the use of traffic actuated signals: the old view of MUTCD guidelines is overly restrictive (as it limits the use of actuated control for low volume signals) ; the theoretical view (that actuated control always delivers better performance, having to be decided weighting the greater cost) is overly optimistic and should not eliminate careful timing and field checking;
- the evaluation of traffic actuated signals should add micro-regulation and macro-regulation; at least with current algorithms and equipments , the performance of actuated control is acceptable but it is not warranted the out-performance of pre-timed control for regular operation (if both are properly timed) ; nevertheless, the gain of flexibility for handling incidents and avoiding the aging of plans is a decisive advantage;
- the new HCM method for evaluating average timings of traffic actuated signals is biased; even for the evaluation of average timing of full-actuated operation, it displays an extreme behavior when simulating increasing degrees of saturation and quickly approach the maximum settings; field behavior shows an smoother increase in average timings (due to demand variation and fail in identifying the end of queues);
- the use of HCM conventional formulas for delay estimates must be improved for precision against field data; even when using average timings from field measurements, forecasted values are consistently greater than field measurements of delay; this same pattern was observed in the application of HCM conventional formulas for delay estimates for pre-timed control (relative values do not display the same bias).

Despite being a large improvement over previous procedures (as HCM/1985 and HCM/1994), the theoretical concern with current methods seems to be justifiable. Practical and theoretical avenues seem to be available.

The main drawback was related to the extreme behavior of the iterative procedure suggested for estimating average timings under actuated control, when used to evaluate heavy traffic intersection (the current procedures for analyzing semi-actuated control seem to be candidate for improvements also).

Despite the significant errors in the estimation of average delay under actuated and even pre-timed control, the prediction of relative performance of alternative control options, using average timing obtained from field measurements, was adequate (as compared to the relative results obtained from field measurements of delay).

This result suggests that the HCM recommendation of using conventional delay formulas for forecasting the delay achieved under actuated control, with minor adjustment in parameters and average phase timing, is a practicable one. Nevertheless, the improvement of the timing prediction algorithm is then a must. Using average timing calculated with the current HCM iterative procedure masks the results in a clearly biased way (against actuated control because of the bias of predicting greater cycle times).

There is a clear suggestion that the benefits from actuated control should be related to the quality of pre-timed control and to the variation of demand or non-regular operation (incidents due to capacity shortage or demand overload). This will add micro-regulation and macro-regulation effects of actuated control. The gains to pedestrians, due to shorter cycle times should also be considered in a wider evaluation.

A minimum agenda should consider the evaluation with several periods during the day (taking into account the variation in pre-timed and actuated control) for a typical pattern of regular operation period and one or two typical patterns of non-regular operation periods (with typical incidents). Costs of plan revisions to avoid the aging of plans or the effects of the aging of plans in performance should also be considered.

Attention to incidents is growing in recent years, mainly for freeway systems, and alternative evaluation methods can be judged to be under development. Attention to aging of plan is an old but neglected issue. Both benefits are also relevant to signal control systems (ROBERTSON and HUNT, 1982, is a basic reference on this subject). Some of the studies adopt the approach of correcting traditional estimates or model coefficients instead of evaluating “design” incidents (and composing model results for regular and incidents performance estimates).

Last, but not least, the results gathered in this research ask for improvement on local methods for traffic control. The quasi-myopic logic of conventional traffic-actuated control seems to ask for improvement so as to reach uniformly better performance than pre-timed control. To this end, the development of adaptative control algorithms is a promising research theme.

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