Design and Evaluation of Isolated Traffic Signal using New Minimum Delay Cycle Length Equation

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Design and Evaluation of Isolated Traffic Signal using New Minimum Delay Cycle Length Equation

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To overcome the limitations of Webster cycle length equation, new minimum delay cycle length equations are proposed for a four phase intersection by researchers in Texas A&M University. These equations are applied in the design of an isolated four legged intersection which is located in Warangal city (Andhra Pradesh). This is followed by the measurement of classified traffic volume counts and stopped delays at the same intersection for both morning and evening peak periods. Using Highway Capacity Manual (HCM) - 2000 methodology the intersection level of service is evaluated. Later signals are designed using proposed minimum delay cycle length equation and also using Webster cycle length equation. From the results it is observed that proposed new cycle length model overcomes the shortcomings of the Webster model. This model produced optimum cycle lengths even for higher traffic volumes.

Keywords : Optimal cycle length; Intersection control; Delay; HCM

INTRODUCTION

IRC: 93-1985 recommends Webster's cycle length equation to calculate the optimum cycle length. This method is widely adopted since it needs limited data and presents dependable results. However when sum of critical phase flow ratios exceeds one, this method fails to produce optimum cycle length. Unfortunately, in India, many times the sum of critical phase flow ratios exceeds one. Moreover the delays calculated using Webster's delay equation yields higher values than the original field results, especially for high traffic volume scenarios. Webster optimum cycle length equation was derived based on this delay equation. Therefore it is logical to conclude that the Webster's cycle length model overestimates the optimal cycle length[1–3]. To over come these difficulties the researchers (Cheng, Tian, Messer, and Liu) proposed new minimum optimum cycle length equation; which is used for the estimation of optimum cycle length of isolated four legged traffic signals in Warangal city. This is followed by estimation of intersection control delay and evaluation of level of service (LOS) using HCM-2000 methodology.

THEORETICAL BACKGROUND

In the 1950s, Webster conducted a series of experiments on pretimed isolated intersection operations. Two traffic signal timing strategies came from his study. One is signal-phase splits. Webster demonstrated, both theoretically and experimentally, that pre-timed signals should have their critical phases timed for the equal degrees of saturation for a given cycle length to minimize the delay. The other is the minimum delay cycle length model, which is shown as equation (1). In developing the model for the optimal minimum delay cycle length, it was assumed that the effective green times of the phases were in the ratio of their respective y (flow ratio v/s) values.

Optimum cycle length is given by $$Co = \frac{(1.5 \times L + 5)}{1 - \sum Y} \quad (1)$$

where C_0 is the optimal minimum delay cycle length(s); L, total lost time within the cycle(s); Y, sum of critical phase flow ratios[4,5].

Webster Delay Equation

The delay calculation for the Webster method is expressed as equation (2).

$$d = \frac{C(1-\lambda_i)^2}{2(1-\lambda_i X_i)} + \frac{X_J^{\,2}}{2q(1-X_i)} - 0.65\left(\frac{C}{q}\right)^{1/3} X^{2+5\lambda_I} \quad (2)$$

where d is the average delay per vehicle on a particular lane group of an intersection (s /veh); C, cycle length (s); v, flow (vehicles/ s); λ, proportion of the effective green with respect to cycle length [*ie*, g/C and g is the effective green (s)]; and x the degree of saturation. This is the ratio of the actual flow to the maximum flow, which can be passed through the intersection from this lane group, and is given by $x = v/(\lambda_s)$, where s is the saturation flow in vehicles per second. The first term of equation (2) represents the delay when traffic is assumed to be arriving uniformly. The second term of the equation

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makes some allowance for the random nature of the arrivals. It is an expression for the delay experienced by vehicles arriving randomly in time at a 'bottleneck', queuing up, and leaving at constant headways. The third term of the equation is an empirical correction term to give a closer fit for all values of flow. Normally, the last term is relatively small compared to the total delay and frequently is omitted by reducing 10% of the first two terms.

The above two strategies are very useful for traffic design and planning. When the two rules are applied together, one can practically minimizes the resulting delay at an isolated pretimed signalized intersection. However, when the traffic demand of an intersection is high, which causes a high value of degrees of saturation, the optimal cycle length based on Webster's equation will become extremely high, may be 30 s to 40 s higher than the value based on the HCM 2000 delay calculation[1-2].

HCM 2000 Delay Equation

The average control delay per vehicle for a given lane group in the HCM 2000 is calculated by using the equation :

$$d = d_1 \times PF + d_2 + d_3 \tag{3}$$

where d is the control delay per vehicle, s/veh; d_1, uniform control delay assuming uniform arrivals, s/veh; PF, uniform delay progression adjustment factor, which accounts for effects of signal progression (in this paper, PF = 1 since it was an isolated intersection); d_2, incremental delay to account for effect of random arrivals and over saturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes no initial queue for a lane group at the start of analysis period, s/veh; and d_3, initial queue delay, which accounts for delay to all vehicles in analysis period due to an initial queue at the start of analysis period, s/veh. A zero initial queue is assumed in this paper.

$$d_1 = \frac{0.50 \times C \times \left(1 - \frac{g_e}{C}\right)^2}{\left(1 - \min(X, 1.0)\frac{g_e}{C}\right)} \tag{4}$$

The equation used to calculate the uniform control delay, described in equation (4), is essentially the same as the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals. Note that degrees of saturation beyond 1.0 are not used in the computation of d_1. The terms in the equation are the same as defined earlier in Webster delay equation.

$$d_2 = 900 \times T\left[(X-1) + \sqrt{\left((X-1)^2 + \frac{240KIX}{cT}\right)}\right] \tag{5}$$

where T, duration of analysis period, hour; k, incremental delay factor that is dependent on actuated controller settings; I, upstream filtering/ metering adjustment factor; C, lane group capacity, vph; and X, lane group v/c ratio or degree of saturation.

Equation (5) is used to estimate the incremental delay due to non uniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of over saturation (over saturation delay). The equation assumes that there is no unmet demand that causes initial queues at the start of the analysis period. The incremental delay term, d_2, is valid for all values of X, including highly oversaturated lane groups.

There are significant differences between the second terms of Webster's delay equation and HCM 2000's second term of delay calculation. When the degree of saturation is close to one, the delay based on the Webster's equation will approach infinity, which is unrealistic. However, the HCM 2000 delay will be somewhat along the solid line of Figure 1 for saturated and oversaturated conditions.

Level of Service Criteria

The level of service is closely related to the average control delay of the intersection.

For easy reference, the HCM 2000 level of service criteria are listed in Table 1.

Modifications to Webster Method

For an isolated intersection, based on Webster's delay equation, the delay will become infinity when the degree of saturation of a lane group approaches one, which is unrealistic, while the delay based on HCM 2000 method can accommodate some random failures and short-term over saturation situations. The HCS software was used to conduct experiments for a typical four-phase intersection over a wide range of volume and lost time scenarios, and the results were used to modify the original Webster minimum delay cycle length equation. The new minimum delay cycle length equations based on this study significantly improve the accuracy of predicting the optimal cycle length for isolated intersections at high traffic volume conditions. The proposed exponential type of nonlinear regression model is presented in equation (6).

$$Co = \alpha \times L \times e^{\beta x} \tag{6}$$

where, α and β are two regression parameters. The value of α and β were calibrated as 1.5 and 1.8, respectively from the experimental data. Thus, the exponential cycle length model shown below was obtained.

Table 1 Level of service criteria for signalised intersection

Level of service (LOS)	A	B	C	D	E	F
Maximum average control delay (sec/veh)	10	20	35	55	80	>80

$$Co = 1.5 \times L \times e^{1.8y} \quad (7)$$

The exponential model is recommended for use in the place of Webster optimum cycle length method because of its simple form and relatively good estimation of the optimal cycle length of an intersection[1-2].

Aggregating Delay Estimates

The procedure for delay estimation yields the control delay per vehicle for each lane group. It is also desirable to aggregate these values to provide average delay for an intersection approach and for the intersection as a whole. In general this is done by computing weighted averages, where the lane group delays are weighted by the existed flows in the group. The delay for an approach is computed as

$$d_A = \frac{\sum d_i v_i}{\sum v_i} \quad (8)$$

where d_A, delay for approach A (s/veh); d_i, delay for lane group i (on approach A) (s/veh); V_i is adjusted flow for lane group i (veh/hr).

Approach delays then can be further averaged to provide the control delay.

$$d_I = \frac{\sum d_A v_A}{\sum v_A} \quad (9)$$

where d_I, delay per vehicle for intersection (s/veh); d_A, delay for approach A (s/veh); V_A, adjusted flow for approach A (veh/hr)[6]

DATA COLLECTION AND ANALYSIS

Study Area Description

It is a four armed junction with two major approaches on Western and Eastern part located in Warangal city in Andhra Pradesh. The schematic diagram of the intersection is as shown in Figure 1. The type of land use is commercial, having central business district (CBD) near to the junction. A significant volume of pedestrian traffic also exists at this junction.

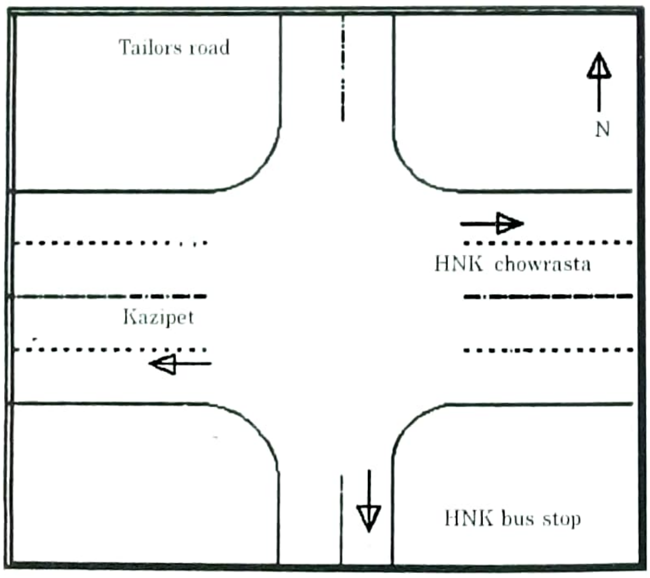

Figure 1 Schematic diagram of Ashoka junction, Warangal city

Physical Inventory of Intersection

At the selected signalized intersection physical inventory was carried out with regards to condition of footpaths, condition of pavement, location of signals and signal setting times, and carriage way widths. Signal setting times were taken from local authorities and later they were verified with stop watches. Approach widths of all the legs of the intersection were measured on field using measuring tape.

Data Collection

Manual counting method is adopted for classified traffic volume count surveys. The surveys are conducted on a typical weekday. The duration of the study was fixed based on the past traffic data. The study durations covering morning and evening peaks programmed for this study were: 8 hours count; 8.00-12.00 noon, and 4:00-8.00 PM. The measured approach widths and the corresponding saturation flow values are presented in Table 2. The saturation flows are calculated as per IRC: 93-1985.

Observed Delays

To measure the stopped delays at the intersection, it requires observers to record number of vehicles stopped at a certain time interval. This time interval is obtained by splitting the red interval (preferably four time intervals). The assumption involved in this method is that each vehicle counted was stopped for an entire interval. The total count of vehicles during each interval multiplied by the duration of stopping, gives the total stopped delay for the corresponding signal cycle. Dividing this delay by the number of vehicles departing the approach provides an estimate of average stopped delay, *ie*, stopped delay per vehicle. The approach wise calculations were done to find the total delay per cycle and average delay per stopped vehicle. The average delays observed in the field for the existing signal settings are presented in Table 3.

Table 2 Approach widths and saturation flow values

Approach name	Approach width, m	Saturation flow, PCUs/h
Kazipet	8.50	4463
Tailors Road	4.00	1950
HNK Chowrasta	6.50	3413
HNK Bus stand	5.00	2550

Table 3 Observed delays at the intersection for existing cycle length

Phases	Approach name	Green time, s	Observed delay, s/veh	Intersection delay, s/veh
A	Kazipet	33	45	
B	Tailors Road	13	52	51
C	HNK Chowrasta	23	56	
D	HNK Bus stand	35	46	

Data Analysis

To demonstrate the usefulness of the proposed models, the signals were designed using both the cycle length models, *ie*, using Webster's method and the Exponential Model. Signal design was carried out for both morning and evening traffic volume for different hours. This is followed by the determination of level of service (LOS) of the intersection as per Highway Capacity Manual (HCM)-2000 methodology[9]. The existing phase diagram is shown in Figure 2. The results for morning and evening traffic were shown in Table 4 and Table 5, respectively.

CONCLUSIONS

1. The New Exponential Model overcomes the shortcomings of the Webster's cycle length model, *ie*, Webster's cycle length model failed in all the cases in the study (sum of critical phase flow ratios exceeded one) but the Exponential Model produced optimum cycle length for all the cases of this study. The comparison between both

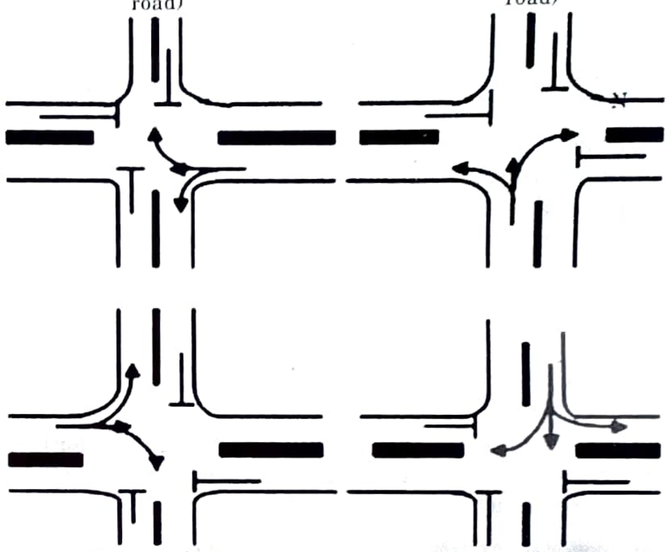

Figure 2 Phase diagram for signal design

Table 4 Comparisons of exponential model and webster method for morning traffic

Analysis for morning flow			Webster method		Exponential method		
Time	Volume, PCU/h	flow ratio	Cycle length, s	Control delay, s	Cycle length	Control delay	LOS
09:00-10:00	2868	1.02	Since flow ratio exceeds one in all the cases, Webster Model fails to yield optimum cycle length		75	34	C
09:15-10:15	2927	1.04			78	36	D
09:30-10:30	2965	1.04			78	37	D
09:45-10:45	3029	1.06			81	39	D
10:00-11:00	3047	1.07			82	40	D
10:15-11:15	3040	1.05			80	37	D
10:30-11:30	3026	1.03			77	36	D

Table 5 Comparison of exponential model and webster method for evening traffic

Analysis for evening flow			Webster method		Exponential method		
Time	Volume, PCU/h	flow ratio	Cycle length, s	Control delay, s	Cycle length	Control delay	LOS
4:30-5:30	3097	1.14	Since flow ratio exceeds one in all the cases, Webster Model fails to yield optimum cycle length		93	31	C
4:45-5:45	3156	1.14			94	32	C
5:00-6:00	3197	1.16			96	32	C
5:15-6:15	3328	1.21			107	37	D
5:30-6:30	3403	1.26			116	40	D
5:45-6:45	3470	1.30			126	44	D
6:00-7:00	3484	1.31			127	44	D
6:15-7:15	3352	1.26			116	40	D

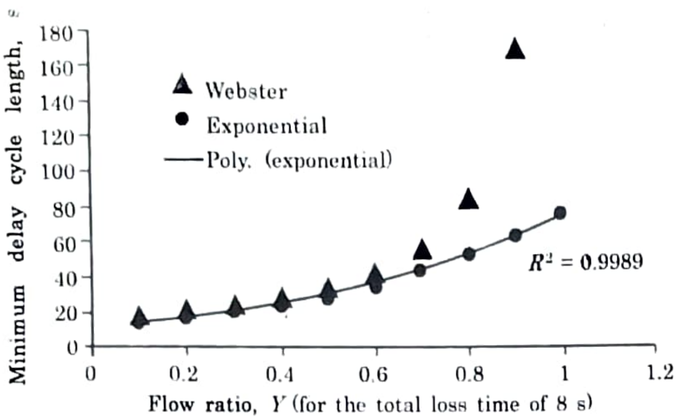

Figure 3 Comparison of webster model with exponential model

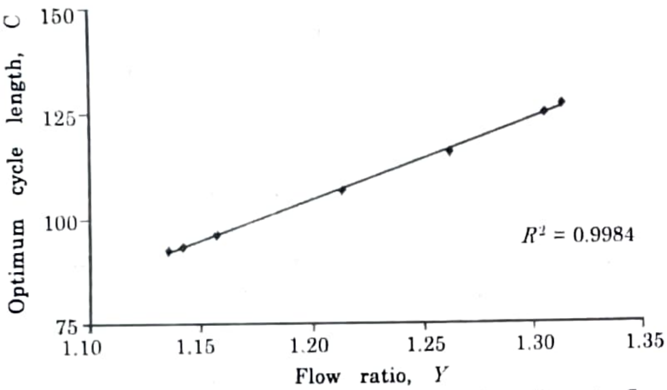

Figure 4 Variation of cycle length with different flow ratios

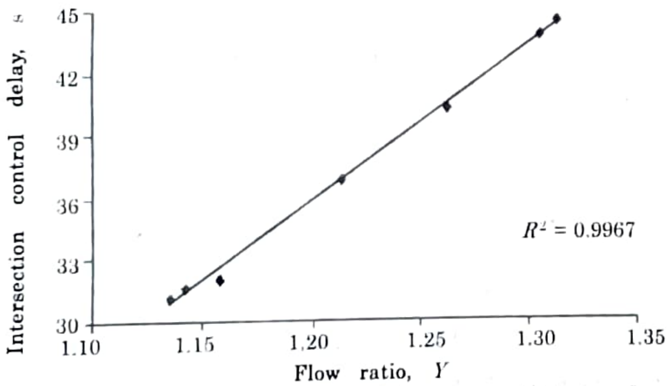

Figure 5 Variation of control delay with different flow ratios

the cycle length models for different flow ratios is shown in Figure 3.

2. The Exponential Model is very useful especially for higher sum of critical phase flow ratios. The minimum delay cycle length values obtained for different flow ratios are shown in Figure 4.

3. The intersection control delays of evening traffic for different flow ratios are shown in Figure 5. The variation of intersection control delay is linear with flow ratio.

4. The level of service (LOS) of the intersection was determined for different traffic volumes both for evening and morning traffic. The level of service (LOS) of the intersection varies between C and D as per Highway Capacity Manual (HCM)-2000 methodology.

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