Development of an Improved Cycle Length Model over the *Highway Capacity Manual 2000* Quick Estimation Method

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Abstract: Chapter 16 of the *Highway Capacity Manual 2000* (HCM 2000) includes models and procedures for calculating capacity and delay at signalized intersections. However, the procedures do not provide estimation of the optimal cycle length which would result in the minimal intersection delay. A quick estimation method for determining the cycle length is described in Appendix A, Chap. 10 of the HCM 2000 for planning level applications. In this method, a simple equation is used to estimate the cycle length if it is not available. However, the estimated cycle length may not be the optimal cycle length from the point of view of achieving minimum intersection delay. To develop a new cycle length model, the Webster's minimum delay cycle length model is first considered. However, based on our study, Webster's minimum delay cycle length model overestimates the optimal cycle length compared to the results from the HCM 2000 delay calculation method, especially under high traffic volume conditions. After investigating three new models developed during this study, an exponential-type cycle length model is recommended. Based on a series of *CORSIM* simulation runs, the cycle length predicted by this model provides better results than the current quick estimation method of the HCM 2000.

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Introduction

Chapter 16 of the *Highway Capacity Manual 2000* (HCM 2000) includes models and procedures for calculating capacity and delay at signalized intersections. However, the procedures do not provide estimation of the optimal cycle length, which would result in the minimal intersection delay. A quick estimation method for determining the cycle length is described in Appendix A, Chap. 10 of the HCM 2000 on Urban Street Concepts. In this method, a simple equation is used to estimate the cycle length if it is not available:

$$C = \frac{L}{1 - \left[\frac{\min(\mathrm{CS},\mathrm{RS})}{\mathrm{RS}}\right]} \tag{1}$$

where C=cycle length (s); L=total lost time (s/cycle); CS=sum of the critical phase traffic volumes (veh/h); RS=reference sum flow rate $(1{,}710 \times \mathrm{PHF} \times fa)$, (veh/h); PHF=peak-hour factor; and fa=area type adjustment factor [0.90 if central business district (CBD), 1.00 otherwise].

The problem with Eq. (1) is that it does not guarantee that the cycle length would produce the minimum delay at an intersection. A good estimate of the cycle length is an important step in performing operational analysis because it is a major parameter in determining signal timing plan, calculating v/c ratio, and estimating signalized intersection delay. If cycle length is wrongly estimated, the other results may not be accurate, consequently.

The purpose of this study is to develop an improved cycle length estimation model which can provide better intersection delays. First, the Webster's optimal cycle length model is evaluated. The study shows that the Webster's optimal cycle length model generally overestimates the cycle length, i.e., the cycle length given by the Webster's model is larger than the one that produces the minimum intersection delay. The overestimation in cycle length is more significant especially when the traffic demand is high. Therefore, a new model, which can better estimate cycle length of an intersection, is proposed based on a study of a wide range of traffic volume and lost time scenarios. Further validation of the proposed model is then conducted using the *CORSIM* simulation model based on traffic volume and geometric data from four intersections in Houston. Finally, major conclusions from this study are presented.

Theoretical Background

In the 1950s, Webster conducted a series of experiments on pre-timed isolated intersection operations (Webster 1969). 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

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minimize the delay. The other is the minimum delay cycle length model, which is shown as Eq. (2). 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.

$$C_0 = \frac{1.5L + 5}{1 - Y} \tag{2}$$

where C_0=optimal minimum delay cycle length (s); L=total lost time within the cycle (s); and Y=sum of critical phase flow ratios (Webster and Cobbe 1966).

The above two strategies are very useful for traffic design and planning. When the two rules are applied together, one can practically minimize the resulting delay at an isolated pre-timed 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 model will become extremely high; it may be 30–40 s higher than the value based on the HCM 2000 delay calculation. This is due to the fact that the Webster's model could only apply to under-capacity conditions while the HCM model can be used for near-capacity or overcapacity conditions.

The optimal cycle length, which gives the minimum average control delay experienced by all vehicles that arrive in the analysis period, is closely related to the delay calculation methodologies. In order to find out why the Webster's optimal cycle length does not provide the minimum HCM control delay, both the Webster's and HCM delay equations need to be studied.

Webster Delay Equation

The delay calculation for the Webster method is expressed as Eq. (3):

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2v(1-x)} - 0.65\left(\frac{C}{v^2}\right)^{1/3} x^{(2+5\lambda)} \tag{3}$$

where d=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 [i.e., g/C and g is the effective green (s)]; and x=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 Eq. (3) represents the delay when traffic is assumed to be arriving uniformly. The second term of the equation 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," queueing 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 (Roess et al. 1998).

Highway Capacity Manual 2000 Delay Equation

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

$$d = d_1 \times PF + d_2 + d_3 \tag{4}$$

where d=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 because an isolated intersection is assumed); d_2=incremental delay to account for effect of random arrivals and oversaturation 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.

The equation used to calculate the uniform control delay, described in Eq. (5), 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 degree of saturation beyond 1.0 is not used in the computation of d_1.

$$d_1 = \frac{0.50c\left(1 - \frac{g}{C}\right)^2}{1 - \left[\min(1,x)\frac{g}{C}\right]} \tag{5}$$

where the terms in the equation are the same as defined previously.

Eq. (6) is used to estimate the incremental delay due to nonuniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of oversaturation (oversaturation 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.

$$d_2 = 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{8kIx}{cT}}\right] \tag{6}$$

where T=duration of analysis period (h); k=incremental delay factor that is dependent on actuated controller settings; I=upstream filtering/metering adjustment factor; c=lane group capacity (veh/h); and x=lane group v/c ratio or degree of saturation.

There are significant differences between the second term of Webster's delay equation and HCM 2000's second term of delay calculation. When the degree of saturation approaches 1, the delay based on the Webster's equation will approach infinity, which is unrealistic.

Optimal Minimum Delay Cycle Length Model

For an isolated intersection, the optimum cycle length corresponds to the minimum total delay of the intersection. This minimum total delay situation can be obtained by selecting an appropriate cycle length and green splits. For a given cycle length, the effective green phases can be selected in proportion to the critical flow ratio of the phases. Now, for the optimal cycle length, because the delay calculations are different between the Webster and HCM 2000 method, as shown previously, one would expect that the optimal cycle length based on HCM 2000 delay will be different from the Webster's optimal cycle length. The

Table 1. Results for Total Lost Time of 12 s

Total volumes (veh/h)	Intersection average control delay (s)	X_{int}	Y	Optimal cycle lengths from *Highway Capacity Software 2000* (s)	Webster's C_0 (s)	$1/(1-Y)$
1,080	17.1	0.48	0.32	37	34	1.47
1,296	19.9	0.54	0.4	42	38	1.67
1,620	24.7	0.62	0.5	53	46	2.00
2,052	32.5	0.77	0.62	58	61	2.63
2,160	35.9	0.8	0.64	63	64	2.78
2,268	39	0.82	0.66	70	68	2.94
2,376	42.6	0.86	0.72	70	82	3.57
2,484	47.3	0.88	0.74	78	88	3.85
2,592	52.6	0.9	0.76	85	96	4.17
2,700	58.7	0.93	0.82	92	128	5.56

following experimental procedure will be used to develop the new cycle length model, which is based on the HCM 2000 delay calculation.

In the experiments, the following assumptions are made in terms of the parameters used in Eq. (6). The duration of analysis period "T" is selected as 0.25 h (15 min); the incremental delay factor "k" is selected as 0.5 because the modeled cycle length is for pre-timed traffic control; and the upstream filtering/metering adjustment factor "I" is selected as 1.0 because the effects of metering arrivals from upstream signals are ignored.

Experimental Procedure

The major parameters that are affecting the optimal cycle length of an intersection are total lost time of the intersection, L, and the approaching traffic volume of the intersection. In fact, the sum of the critical phase flow ratio, Y, instead of the total volume is the real factor to affect the minimum delay cycle length. By keeping the same L and Y, the optimal cycle length should be similar even if the volumes are different. This is why Webster's optimal cycle length model only includes L and Y.

In order to modify the Webster's optimal cycle length model, a series of experiments with a wide range of volume and lost time scenarios were conducted. *Synchro 5* (Trafficware 2001) was used to derive the initial value of optimal cycle length and green splits. The primary reason of using *Synchro 5* versus the *Highway Capacity Software 2000* is that *Synchro 5* is very easy to use in terms of data input and timing optimization. Although the HCS 2000 is compatible with the HCM calculations, its generic algorithm-based optimization approach does not guarantee the best result. Because *Synchro 5* and HCM 2000 are not completely compatible (Benekohal et al. 2002) in their capacity calculation procedures, the final optimal cycle length and green splits were either verified or changed by using the gradient search methodology from the *Highway Capacity Software 2000* delay calculations. The same traffic volume, lost time, and roadway conditions were input into an Excel spreadsheet and the optimal cycle lengths for each case were calculated using Webster's optimal minimum delay cycle length model. In order to compare the Webster and HCM 2000 results, the optimal cycle lengths from the two different methods were plotted together. The following summarizes the major steps of the procedure to conduct the experiments:

Table 2. Results for Total Lost Time of 14 s

Total volumes (veh/h)	Intersection average control delay (s)	X_{int}	Y	Optimal cycle lengths from *Highway Capacity Software 2000* (s)	Webster's C_0 (s)	$1/(1-Y)$
1,080	19.1	0.5	0.32	40	38	1.47
1,296	21.9	0.56	0.4	45	43	1.67
1,620	26.4	0.65	0.5	54	52	2.00
1,944	33.1	0.77	0.58	58	62	2.38
2,052	35.8	0.78	0.62	66	68	2.63
2,160	39.1	0.8	0.64	72	72	2.78
2,268	42.7	0.83	0.66	76	76	2.94
2,376	46.7	0.86	0.72	79	93	3.57
2,484	51.5	0.88	0.74	88	100	3.85
2,592	57	0.92	0.76	90	108	4.17
2,700	64.1	0.95	0.82	93	144	5.56

Table 3. Results for Total Lost Time of 16 s

Total volumes (veh/h)	Intersection average control delay (s)	X_{int}	Y	Optimal cycle lengths from *Highway Capacity Software 2000* (s)	Webster's C_0 (s)	$1/(1-Y)$
1,080	21.3	0.51	0.32	44	43	1.47
1,404	25.3	0.6	0.42	53	50	1.72
1,620	28.9	0.66	0.5	60	58	2.00
1,944	35.9	0.77	0.58	64	69	2.38
2,052	39	0.79	0.62	70	76	2.63
2,376	50.5	0.87	0.72	86	104	3.57
2,484	55.9	0.9	0.74	92	112	3.85
2,592	61.7	0.93	0.76	95	121	4.17
2,700	68.8	0.95	0.82	105	161	5.56

Table 4. Results for Total Lost Time of 18 s

Total volumes (veh/h)	Intersection average control delay (s)	X_{int}	Y	Optimal cycle lengths from *Highway Capacity Software 2000* (s)	Webster's C_0 (s)	$1/(1-Y)$
1,080	23.2	0.5	0.32	50	47	1.47
1,296	25.8	0.57	0.4	56	53	1.67
1,512	29.2	0.64	0.44	60	57	1.79
1,836	35.9	0.75	0.54	68	70	2.17
2,052	42.1	0.82	0.62	72	84	2.63
2,160	45.3	0.83	0.64	80	89	2.78
2,268	49.7	0.85	0.66	88	94	2.94
2,376	54.3	0.88	0.72	93	114	3.57
2,592	65.5	0.93	0.76	106	133	4.17
2,700	73.5	0.96	0.82	114	178	5.56

Table 5. Results for Total Lost Time of 20 s

Total volumes (veh/h)	Intersection average control delay (s)	X_{int}	Y	Optimal cycle lengths from *Highway Capacity Software 2000* (s)	Webster's C_0 (s)	$1/(1-Y)$
1,080	24.5	0.51	0.32	54	51	1.47
1,188	26.4	0.54	0.34	58	53	1.52
1,296	28.1	0.57	0.42	63	60	1.72
1,728	36	0.74	0.52	66	73	2.08
1,836	38.5	0.75	0.54	74	76	2.17
2,052	45.2	0.82	0.62	80	92	2.63
2,268	53.4	0.87	0.66	90	103	2.94
2,592	72	0.97	0.76	100	146	4.17
2,700	77.9	0.97	0.82	122	194	5.56

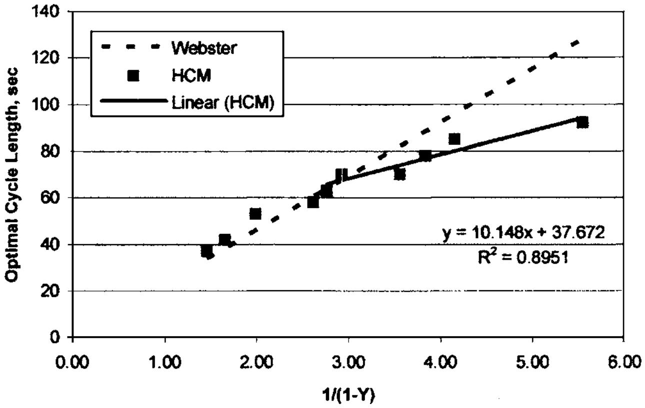

Fig. 1. Optimal cycle lengths versus $1/(1-Y)$ for the total lost time of 12 s

1. Start with a lower total lost time, i.e., 12 s, and lower total volume, i.e., 1,080 veh/h for a four-phase isolated intersection. For this base condition, each intersection approach has one exclusive left-turn lane and one shared right and through lane. For the east–west (EW) street, the left, through, and right turning movement volumes are 50, 200, and 20 vph, respectively. For the north–south (NS) street, the left, through, and right turning movement volumes are 50, 200, and 20 vph, respectively. The signal operates as protected leading left four phases. Input the traffic data into *Synchro 5*. Optimize the green splits and cycle length using *Synchro*'s optimization tool.
2. Output *Synchro*'s data into HCS 2000 software.
3. Calculate the total control delay of the intersection using HCS 2000. For the same cycle length, conduct gradient search, i.e., increasing and decreasing 5% green splits, to find the minimum delay for different green splits.
4. Perform the gradient search for the optimal cycle length, i.e., increasing or decreasing the cycle length from Synchro by 1 or 2 s; and reoptimize the green splits for each new cycle length. Find the optimal cycle length corresponding to the minimum total delay from the HCS 2000.
5. Increase the traffic volume for the intersection. Repeat Steps 1–4 to find the optimal cycle length for the new traffic volume.
6. Repeat Step 5 until the v/c ratio or the degree of saturation approaches 1.

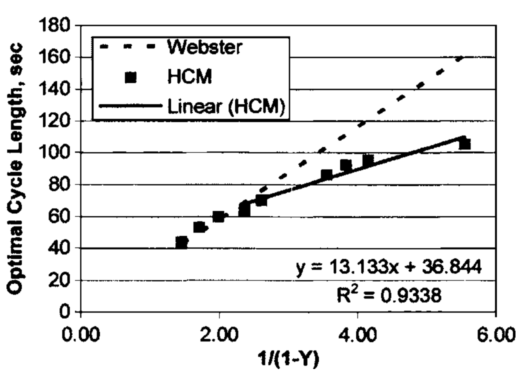

Fig. 2. Optimal cycle lengths versus $1/(1-Y)$ for the total lost time of 16 s

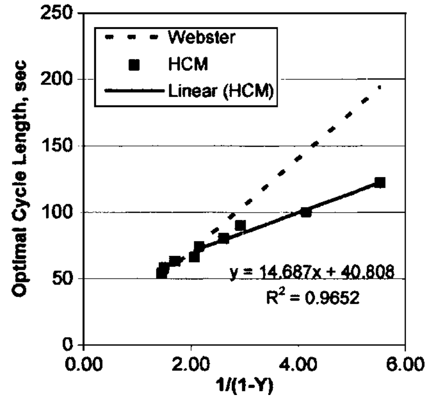

Fig. 3. Optimal cycle lengths versus $1/(1-Y)$ for the total lost time of 20 s

7. Increase the total lost time to 14 s. Repeat Steps 1–6 to find the new optimal cycle lengths for each new lost time and traffic volume.
8. Repeat Step 7 for the total lost time to 16, 18, and 20 s to get all the optimal cycle length for different traffic volumes and lost times using the HCM method.
9. Calculate optimal cycle length by using Webster's optimal cycle length model corresponding to the same traffic volumes and lost times in Steps 1–8.

Results and New Minimum Delay Cycle Length Models

The experiments were conducted over a wide range of volume and covered most lost time situations. Various parameters and results under different total lost time scenarios are shown in Tables 1–5, including arrival volumes, average control delays, degrees of saturation, X_{int}, the sums of flow ratios, Y, the HCM optimal cycle lengths, and the Webster's optimal cycle lengths, C_0.

From the results shown in Tables 1–5, for the same lost time, the optimal cycle lengths from both the HCM 2000 and the Webster's methods increase with the increase of volumes and degrees of saturation. The optimal cycle lengths are similar for both the HCM 2000 and the Webster's optimal cycle length model under low volumes and degrees of saturation. However, at higher volumes and degrees of saturation scenarios, Webster optimal minimum delay cycle lengths are always higher than those from the HCM 2000 method. From the results of Tables 1–5, up to level of Service C (LOS C) (delays less than or equal to 35 s), the residuals between the HCM 2000 and the Webster's optimal cycle length model are small. Thus, the Webster's optimal cycle length model is satisfactory for level of Service C or better.

Table 6. Calculated R-Squared Values for the Minimum Delay Cycle Length Models

	Webster model	Recalibrated Webster model	Modified Webster model	Exponential cycle length model
SS_T	19,196	19,196	19,196	19,196
SS_E	18,938	7,620	824	2,011
R^2	0.013	0.603	0.957	0.895

Note: SS_T=total corrected sum of squares and SS_E=error sum of squares.

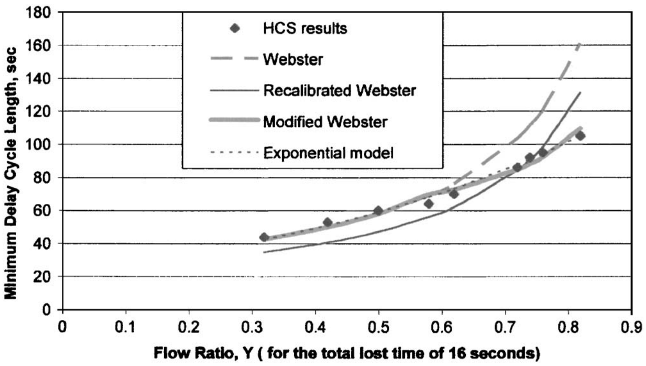

Fig. 4. Comparison of Webster minimum delay cycle length model with three new models

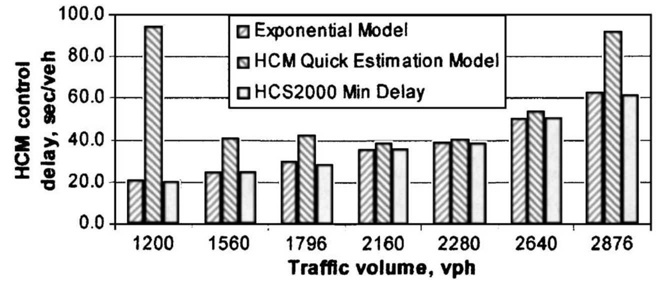

Fig. 5. Delay comparison of exponential model, *Highway Capacity Manual* quick estimation method and *Highway Capacity Software 2000* minimum delay cycle

For level of Service D (LOS D) and worse, the Webster optimal cycle length model clearly overestimates the cycle lengths. The higher the volume and total lost time, the higher the overestimation.

In order to improve the Webster's optimal cycle length model, three regression models were proposed in this paper. The first one recalibrated the Webster's minimum delay cycle length model. The form of the recalibrated Webster model is shown in Eq. (7) (Pacelli 1999).

$$C_0 = \frac{aL + b}{1 - Y} \tag{7}$$

where a and b=optimal minimum delay cycle length calculation coefficients. By using SPSS software (SPSS 2000), the a and b were obtained as 1.0 and 7.6, respectively. Therefore, the recalibrated Webster's model is shown as Eq. (8):

$$C_0 = \frac{1.0L + 7.6}{1 - Y} \tag{8}$$

In order to develop the second model, the optimal cycle lengths based on HCM 2000 and Webster's model were plotted with the inverse of $1-Y$. Figs. 1, 2, and 3 show the plots for the total lost times of 12, 16, and 20 s, respectively.

From the graphs, one can see that for the lower values of $1/(1-Y)$ or Y, Webster's results fit well with the HCM 2000 results. However, for the higher values of $1/(1-Y)$ corresponding to the data points with the LOS D or worse, a new improved linear regression model was applied for each total lost time case. From these linear regression equations, the slope of these equations changes almost linearly with increasing total lost time, but the intercepts are quite similar for different lost time cases. Therefore, the following modified Webster's model was proposed:

$$C_0 = \frac{a + bL}{1 - Y} + c \tag{9}$$

Because a range of C_0, L, and Y data were available from Tables 1–5, a, b, and c were estimated as 0.6, 2.9, and 40, respectively, by using nonlinear regression. The modified Webster's model for the LOS D or worse situation is shown in Eq. (10) for a four-phase intersection:

$$C_0 = \frac{0.6L + 2.9}{1 - Y} + 40 \tag{10}$$

The modified Webster's model is a two-piece model. Selection of formula for cycle length C_0 is related to the LOS of intersections. If LOS$<$D, then use Eq. (2) to calculate C_c, and if LOS $>=$D then use Eq. (10) to calculate C_0. However, because the modified Webster's model is a two-piece model, it does not have a smooth transition around the connection point of the two models shown in Eqs. (2) and (10). To overcome this shortcoming, the third model, the exponential type of nonlinear regression model, was proposed:

$$c_0 = \alpha L e^{\beta Y} \tag{11}$$

where α and β=two regression parameters. The α and β were calibrated as 1.5 and 1.8 from the experimental data, respectively. Thus, the exponential cycle length model shown in Eq. (12) was obtained.

Table 7. Comparisons of Exponential Model, Quick Estimation Method, and *Highway Capacity Software* Optimal Results

			Cycle Length, (s)			HCM control delay (s/veh)		
L (s)	Y	Volume (veh/h)	HCM 2000 optimal cycle	Exponential model	HCM quick estimation method	HCM 2000 minimum delay	Exponential model	HCM quick estimation method
16	0.32	1,200	45	43	26	20.8	21.6	94.8
16	0.42	1,560	50	51	32	25.1	25.1	41.4
16	0.5	1,796	57	59	38	28.7	30.0	42.3
16	0.58	2,160	68	68	54	36.2	36.2	39.2
16	0.62	2,280	70	73	62	39.0	39.0	40.7
16	0.72	2,640	92	88	112	50.8	51.0	53.9
16	0.76	2,876	97	94	244	61.5	63.5	92

Note: HCM=*Highway Capacity Manuals.*

Table 8. *CORSIM* Simulation Results for HCM Quick Estimation Method and the Exponential Model

Intersection				Turning movement count			HCM quick estimation method			Modified cycle length model		
NS street	EW street	Peak hour	Direction	Left (VPH)	Through (VPH)	Right (VPH)	Cycle (s)	Intersection control delay (s/veh)	S.D. (s/veh)	Cycle (s)	Intersection control delay (s/veh)	S.D. (s/veh)
Gessner	Westheimer	a.m.	NB	114	530	103	166	37.17	0.66	113	27.79	0.68
			SB	326	503	138						
			EB	174	2,670	144						
			WB	128	861	88						
		p.m.	NB	187	690	84	230	79.69	1.2	165	56.18	2.61
			SB	311	1,023	265						
			EB	409	1,542	179						
			WB	269	2,415	210						
Gessner	Westview	a.m.	NB	55	714	104	27	56.51	0.67	48	11.4	0.18
			SB	65	1,197	49						
			EB	49	62	129						
			WB	37	42	21						
		p.m.	NB	101	1,372	104	34	35.7	0.37	58	11.79	0.73
			SB	57	1,248	111						
			EB	78	81	131						
			WB	68	145	103						
SH 6	Westheimer	a.m.	NB	96	1,795	375	361	108.13	4.6	138	43.16	2.22
			SB	484	1,000	87						
			EB	782	1,607	80						
			WB	159	211	225						
		p.m.	NB	460	1,188	223	400	157.42	11.72	119	38.18	0.32
			SB	449	1,612	570						
			EB	395	560	95						
			WB	476	1,497	264						
Wilcrest	Westheimer	a.m.	NB	133	327	126	231	113.14	4.85	165	37.13	1.57
			SB	178	376	70						
			EB	151	3,387	105						
			WB	103	962	92						
		p.m.	NB	307	505	107	249	118.47	2.4	190	68.74	4.27
			SB	201	396	141						
			EB	118	1,746	117						
			WB	153	3,206	134						

Note: HCM=*Highway Capacity Manuals*; NS=north–south; EW=east–west; and S.D.=standard deviation.

$$c_0 = 1.5Le^{1.8Y} \tag{12}$$

To further compare the Webster optimal minimum delay cycle length model with the three new models, the *R*-squared values (coefficients of determination), R^2, for the above models were calculated using Eq. (13) (Montgomery and Runger 2000):

$$R^2 = \frac{SS_R}{SS_T} = 1 - \frac{SS_E}{SS_T} \tag{13}$$

where SS_R=regression sum of squares; SS_E=error sum of squares; and SS_T=total corrected sum of squares.

Based on the *R*-squared results for the models shown in Table 6, the recalibrated Webster model is better than the Webster model and the modified Webster model is the best. Fig. 4 illustrates the better fittings of the modified Webster model and the exponential cycle length model than the original Webster minimum delay cycle length model for the case of the total lost time of 16 s.

Validation of the New Minimum Delay Cycle Length Models

Both the modified Webster's model and the exponential model could be adopted in the HCM simple estimation method to obtain a good initial estimate on the optimal cycle length once the volume, geometry, and lost times are given for an intersection. However, the exponential model is recommended in the quick estimation worksheet because it is very simple. In addition, the modified Webster's model is a two-piece model that has two equations and may not be accurate on the transition part between two equations. Although some commercial software, such as *Synchro*, *TEAPAC*, *TRANSYT7F*, *HCS 2000*, etc., have their own optimizing cycle length techniques, none of them are simple enough to put into the simple estimation method or worksheet of the manual.

In order to illustrate the difference between the new exponential model and the existing HCM cycle length model, the following example was conducted. The cycle lengths for both the exponential model and the existing HCM model were calculated with the same lost time, geometric layout, and over a range of traffic volumes. In addition, by using HCS 2000 software, the HCM control delays corresponding to these cycle lengths were calculated. By conducting a gradient search using HCS 2000, the optimal cycle lengths and their delays were also calculated. The optimization tool of HCS 2000 was not used because the genetic algorithm by this software does not always guarantee the optimal cycle length. The above results are listed in Table 7. From Table 7, cycle lengths from the exponential model agree well with the HCS 2000 optimal cycle lengths. However, cycle lengths from HCM quick estimation method are different from HCS 2000 optimal cycle lengths for the same traffic scenarios. Fig. 5 illustrates the HCM control delay calculated from three different methods. From Fig. 5, for the same amount of traffic volume, the control delay from the exponential model is closer to the HCM minimum delay and the delay from the HCM quick estimation method is always higher than the HCM minimum delay. For both high traffic volume and low traffic volume scenarios, the HCM quick estimation method produced significantly higher delays than the HCM minimum delay, sometimes over 200%.

In order to further validate that the proposed exponential model provides better delay results than the current HCM 2000 quick estimation method, a series of simulation runs were conducted using the *CORSIM* model. The a.m. and p.m. peak hour traffic counts from four intersections in Houston, were collected. For each intersection and traffic scenario, the cycle length was estimated using both the HCM 2000 quick estimation method and the exponential model. With the obtained cycle length from each model, three *CORSIM* simulation runs were conducted and the average intersection control delays were calculated. The results from the *CORSIM* runs are presented in Table 8. Based on the results shown in Table 8, the exponential model resulted in significantly lower delays than the HCM 2000 quick estimation method.

From the above analysis, the exponential model demonstrates improvement over the existing HCM quick estimation method because the cycle length obtained from the exponential model matches closer to the optimal cycle length of an intersection.

Summary and Conclusions

In this paper, we proposed a new cycle length estimation model to overcome the shortcomings of the existing quick estimation method of the HCM 2000. The model was developed based on the minimum delay criteria from the HCM delay equation and was evaluated under a wide range of traffic and lost time situations. The model was further validated using the *CORSIM* simulation model based on traffic and geometric data from four intersections in Houston. Based on the results of this study, the following conclusions were reached:

1. The cycle length calculated from the current equation of the HCM 2000 quick estimation method can sometimes be significantly different from the optimal cycle length of an intersection. The delay corresponding to this cycle length is found to be always higher than the minimum delay of the intersection. Therefore, the timing plan, volume capacity ratio, and delay estimated using the current cycle length equation of HCM 2000 quick estimation method may not be valid under some circumstances.
2. The Webster's cycle length model overestimates the optimal cycle length based on HCM delay calculation, especially for high traffic volume scenarios.
3. After comparing three new proposed cycle length models, the exponential model is recommended to use in the simple estimation method of HCM because of its simple form and relatively good estimation of the optimal cycle length of an intersection.
4. Based on the *CORSIM* simulation runs, the proposed exponential model provides significantly better delay results comparing to the HCM quick estimation method.
5. This study is limited to the four-phase intersection's optimal cycle length analysis. Further studies should be conducted on two, three, and other multiphase situations to develop a more generalized model. In addition, the analysis duration T is limited as 15 min (i.e., 0.25 h). The effect of T should also be included in the generalized model. Nevertheless, a similar research methodology as proposed in this study could be applied.

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