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Research Article

Development of an Optimization Traffic Signal Cycle Length Model for Signalized Intersections in China

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The primary objective of this study is to develop an optimization traffic signal cycle length model for signalized intersections. Traffic data were collected from 50 signalized intersections in Xi'an city. Using comprehensive delay data, the optimization cycle length model is re-recalibrated to the Chinese traffic conditions based on the Webster delay model. The result showed that the optimization cycle length model takes vehicle delay time, pedestrian crossing time, and drivers' anxiety into consideration. To evaluate the effects of the optimization cycle length model, three intersections were selected for a simulation. The delay time and queue length based on the optimization cycle length model and the TRRL model were compared. It was found that the delay times and queue lengths with the optimization cycle length model were significantly smaller than those with the TRRL model. The results suggested that the optimization traffic signal cycle length model was more optimal than the TRRL model.

1. Background

The intersection is an important part of urban road networks, the smooth flow of which plays a key role in the vehicle speed and operating efficiency of the entire road network. However, during the past two decades, with motorization rapidly increasing, more and more bottleneck effects have been exposed at intersections in urban areas. In recent years, transportation professionals in China have developed signalized intersections. There are lots of benefits for developing the signalized intersections. On the one hand, the control afforded by the traffic lights separates the conflicting traffic flows in time and improves vehicle safety and operation efficiency; on the other hand, the vehicles on the approach are suspended periodically, causing delays. Therefore, the traffic signal cycle plays a key role in intersection traffic control. A reasonable cycle length can effectively alleviate or prevent traffic congestion and reduce emissions, noise pollution, energy consumption, and travel delay time.

Probabilistic cycle length calculation was first taught at the Yale School of Highway Traffic in the 1950s and earlier, which used the Poisson tables to determine a cycle that would serve some percentage (based on probabilities) of waiting queues for successive cycles [1]. This is linked to the objective of reducing cycle failures as the authors indicate. However, it has also been recommended that the method is useful only in light traffic conditions.

During the past decades, several studies have been conducted that address traffic signal cycle length. One class of methods seeks to minimize the delay time of vehicles at intersections [2–8]. The most well-known and typical traffic signal cycle length models are the TRRL model and the ARRB model.

The TRRL model [3] has been widely used. In developing the TRRL model for the optimal minimum delay cycle length, it was assumed that the effective green times of the phases were in the range of their respective flow ratio values. The TRRL model is given as

$$c_0 = \frac{1.5L + 5}{1 - Y}, \tag{1}$$

where c_0 represents the optimal cycle length (sec); L represents the total lost time (sec); and Y represents the sum of the critical flow ratio of all phases.

The ARRB model [4] introduced the "parking compensation coefficient" to the TRRL model and it was combined with the vehicle delay time to evaluate the degree of optimization of the signal timing program. The ARRB model is given as

$$c_0 = \frac{(1.4 + k) + 6}{1 - Y}, \tag{2}$$

where c_0 represents the optimal cycle length (sec); k represents the parking compensation coefficient; and Y represents sum of the critical flow ratio of all phases.

Although the above models are widely used, they have some limitations: (1) under near-saturated or saturated conditions, the optimal cycle length formulation proposed by Webster becomes infeasible because it generates an unreasonably large cycle length as the intersection critical flow ratio approaches one; (2) it becomes inapplicable if the intersection critical flow ratio is equal to or greater than one.

The *Highway Capacity Manual* [9] also proposed the cycle length model and the delay time model of signalized intersections. The delay time model, which can be applied to variety of saturation states, has been widely used. The cycle length model is based on the expected saturation; therefore, it does not guarantee that the cycle length would produce the minimum delay at an intersection. It is given as

$$C = \frac{L}{1 - [\min(\mathrm{CS}, \mathrm{RS}) / \mathrm{RS}]}, \tag{3}$$

where C represents the cycle length (sec); L represents the total lost time (sec/cycle); CS represents the sum of the critical phase traffic volumes (veh/h); RS represents the reference sum flow rate (1710∗PHF∗fa), (veh/h); PHF represents the peak-hour factor; and fa represents the area type adjustment factor [0.9 if central business district and 1.0 otherwise].

Day et al. [8] evaluated the effectiveness and efficiency of Webster's cycle length and the HCM intersection saturation metric. The authors showed that calculations from Webster's model and the HCM provided a framework for identifying periods of time when the cycle length could be substantially shortened, periods of the day when an increase in the cycle length would provide some modest improvements, and periods of the day when the cycle length was adequate and capacity problems could be addressed by adjusting the splits. A similar study conducted by Cheng et al. [7] compared Webster's minimum delay cycle length model and the HCM 2000 optimal cycle model and recommended an exponential-type cycle length model.

Based on the idea of minimizing the delay time, many optimization cycle length models were developed using linear or nonlinear regression methods and probabilistic approaches among others [10–12]. Lan [10] proposed a nonlinear optimal cycle length. Using the optimal timing variables obtained based on the delay minimization criterion, the functional relationship between the optimal cycle lengths and the traffic flow parameters, including the intersection critical flow ratio, the total lost time, and the duration of the analysis period, was established through a nonlinear regression analysis. The formulation was found to generate fairly accurate estimates of optimal cycle length with a 5.7% average deviation from the analytical solutions. Han and Li [11] studied a probabilistic approach to cycle length optimization. Based on the idea of selecting a cycle length that is small enough to ensure low delay while providing adequate capacity to handle most of the fluctuating demand conditions, a five-step framework was proposed for carrying out the analyses. Subsequent sensitivity analyses, level-of-service assessments, and cycle failure rate estimations were conducted on the basis of random demand. It was found that longer cycle lengths did not yield the optimal delay results and, with extremely short cycle lengths, the delay was usually high because of a lack of capacity and hence frequent cycle failures were guaranteed.

A second class of methods is based on studies aimed at optimizing the cycle length for a saturated or oversaturated intersection [13–15]. Chang and Lin [13] studied the optimal signal timing for an oversaturated intersection. The authors presented a timing decision methodology which considers the whole oversaturation period and discrete dynamic optimization models were developed. The optimal cycle length and the optimal assigned green time for each approach were determined for the case of two-phase control. It was found that the proposed discrete type performance index model is a more appropriate design for congested traffic signal timing control.

A third class of methods is based on taking the factor of emissions, fuel, and other environmental factors into consideration when developing the cycle length model [5, 6, 16]. Li et al. [5, 6] established a signal timing model which optimizes signal cycle length and green time by using integrated optimization of the traffic quality, fuel consumption, and emission pollution. It was found that when the signal cycle length increased from 20 to 200 s, there was an optimal value corresponding to the performance index function. When the traffic flow rate was larger, the optimal signal cycle length corresponding to the performance index function increased. However, the parameters of the model are complicated and difficult to obtain and is therefore useful for the purpose of study only and is not practical in engineering practice.

The fourth class of methods is based on simulation and an intelligent algorithm to optimize the cycle length or to develop an optimization cycle length model [17–19]. Using the theory analysis and computer simulation test, Yang et al. [19] established a delaying calculation model of through running and left-turning vehicles on a roundabout. Based on the model, an optimal cycle length calculation method targeting minimum delay was derived, which was suitable for the multiapproach going in coordination with roundabout controls. Park et al. [17] evaluated a stochastic signal optimization method based on a genetic algorithm using the microscopic simulation program CORSIM. According to the method, the cycle length, the green ratio, and the phase difference were optimized at the same time. Kim et al. [18] compared the performance between an artificial neural network (ANN) and analytical models for real-time cycle length design. It was found that the ANN model provides optimal cycle lengths stably adjusted by the minimum value, the maximum value,

and the cycle increment, while the analytical model promotes congestion under certain operational conditions.

As can be seen by the brief general discussion above, most of the previous studies focused on the optimal cycle length. A variety of methods and models were put forward either based on the minimum delay time or minimum emission or considering the oversaturation consideration of an intersection. Even though previous studies have, to some extent, improved the traffic signal cycle, they suffer from several limitations. The literature review indicated that the following issues have not been addressed in previous studies: (1) most of the previous cycle length models only aimed at one target (minimum delay time or minimum emission) or only applied to the saturation status; however, a cycle length model of a different status was not considered and (2) most of the cycle length models did not consider the factor of pedestrian crossing and that a long cycle length would cause driver anxiety, which could reduce the traffic safety of the intersection.

The primary objective of this study was to develop and evaluate a new optimization traffic signal cycle length for signalized intersections in China. More specifically, the study presented in this paper focused on the following two tasks: (1) to develop an optimization traffic cycle length model for signalized intersections and (2) to evaluate the effects of the optimization cycle length model.

2. Data Collection

Field data collection was conducted at 50 signalized intersections in Xi'an which is one of the typical biggest cities in China. The sites were carefully selected such that their geometric design and traffic control feathers represent the most common situations in major cities in China. To achieve the research objective, the following criteria were applied in the site selection process.

(1) The selected intersection should be a typical signalized intersection (i.e., four-leg intersection or T-leg intersection). Roundabouts and other deformity intersections must not be included.

(2) The selected signalized intersections should include different types (i.e., where different grade roads intersect), for example, the intersection of a main road and a minor arterial road and the intersection of a main road and branch road.

(3) Two hours of traffic data of each selected sites should be recorded in peak period.

The traffic data of the selected signalized intersection, including the traffic volume, delay time, cycle length, green split, and saturation, were investigated, among which the direct traffic data were traffic volume and cycle length and the indirect traffic data were delay time, green split, and saturation. The calculation methods of the indirect traffic data were shown as follows.

Delay Time. The individual sample survey method was used to determine the delay time, which is given as

$$D = N * t,$$
$$\overline{d} = \frac{D}{V}, \tag{4}$$

where D represents the total delay time and N represents the total number of suspended vehicles; t represents the interval time; $\overline{d}$ the average delay time of each vehicle at the approach; and V represents the total volume at the approach.

Green Split. The green split is given as

$$\lambda = \frac{g}{C}, \tag{5}$$

where λ represents the green split; g represents the effective green time; and C represents the cycle length.

Saturation. The saturation is given as

$$x_i = \frac{q_i}{\lambda_i s_i}, \tag{6}$$

$$X = \max\{x_i\}, \quad i = 1, 2, 3, \ldots, n, \tag{7}$$

where x_i represents the saturation for phase i; q_i represents the traffic volume; λ_i represents the green split; s_i represents the saturation traffic volume; and X represents the saturation degree of the intersection.

In order to collect all the traffic data mentioned above, the following detailed information were collected during the investigation: (1) the exact time at which each phase began and ended, as well as the cycle time; (2) the number of vehicles at every approach of each intersection at 15-minute intervals; and (3) the number of delayed vehicles at every approach at 15-second intervals as well as the number of queued vehicles and nonqueued vehicles. The data collection results are shown in Table 1.

3. Optimization Traffic Signal Cycle Length Model

In the Webster delay model, the average delay time consists of three components: the delay time due to the uniform traffic flow (d_1), the delay time due to the random traffic flow (d_2), and compensation term due to the different traffic environment (d_3). Thus, the Webster delay model should be corrected according to the traffic conditions in China. Three parameters α, β, and γ were introduced. α is the correction coefficient of the first component of delay time; β is the correction coefficient of the second component of delay time;

Table 1: Data collection results.

Sites number	Traffic parameters				
	q	$\overline{d}$	λ	C	X
1	2400	26.46	0.50	120	0.64
2	1470	53.92	0.28	160	0.70
3	990	56.07	0.20	120	0.81
4	1835	36.77	0.36	120	0.85
5	1150	37.84	0.32	110	0.47
6	1207	33.89	0.45	160	0.60
7	1600	22.98	0.68	180	0.92
8	1490	24.06	0.65	120	0.92
9	870	11.05	0.55	80	0.21
10	630	25.57	0.36	100	0.23
11	2065	17.76	0.72	155	0.90
12	185	10.49	0.47	60	0.18
13	790	18.05	0.50	75	0.82
14	845	14.94	0.47	60	0.81
15	1845	37.08	0.51	100	0.98
16	1530	33.74	0.50	105	0.98
17	770	20.22	0.56	95	0.85
18	1540	37.25	0.56	95	0.70
19	410	12.49	0.41	60	0.35
20	770	14.15	0.42	60	0.67
21	875	19.43	0.41	75	0.73
22	745	24.03	0.42	100	0.65
23	745	36.92	0.41	130	0.63
24	1180	33.73	0.33	110	0.76
25	1040	41.36	0.31	120	0.64
26	1395	40.36	0.39	120	0.96
27	495	24.05	0.31	90	0.30
28	735	44.18	0.33	150	0.47
29	1025	37.62	0.16	80	0.32
30	1805	36.75	0.31	120	0.51
31	2503	22.01	0.55	120	0.69
32	1439	66.19	0.27	170	0.67
33	1054	60.08	0.19	130	0.88
34	1770	38.17	0.33	120	0.77
35	1184	12.07	0.61	110	0.45
36	1063	34.16	0.44	140	0.56
37	2810	61.32	0.36	170	0.83
38	1865	37.28	0.54	130	0.95
39	891	12.76	0.56	80	0.19
40	683	26.63	0.33	95	0.24
41	2236	25.27	0.68	170	0.88
42	198	11.90	0.46	60	0.17
43	720	18.64	0.51	70	0.86
44	888	19.11	0.42	75	0.79
45	1716	39.55	0.53	100	0.96
46	1971	36.40	0.46	120	0.98
47	1710	58.43	0.56	90	0.91
48	1462	19.64	0.54	100	0.75
49	441	16.25	0.43	60	0.32
50	755	17.54	0.40	60	0.63

q represents the maximum traffic volume of the intersection in 15 minutes, pcu/15 min;
$\overline{d}$ represents the average delay time of each vehicle, s;
λ represents the maximum green split;
C represents the cycle length of the saturation, s;
X represents the saturation degree of the intersection.

γ is the correction coefficient of the third component of delay time. The corrected Webster delay model is given as

$$\begin{aligned} d &= \alpha d_1 + \beta d_2 + \gamma d_3, \\ d_1 &= \frac{C(1-\lambda)^2}{2(1-\lambda x)}, \\ d_2 &= \frac{x^2}{2q(1-x)}, \\ d_3 &= -0.65\left(\frac{C}{q^2}\right)^{1/3} x^{2+5\lambda}, \end{aligned} \tag{8}$$

where d represents the average vehicle delay time; d_1 represents the uniform delay time; d_2 represents the random delay time; d_3 represents the delay compensation value; α, β, and γ represent the unknown coefficient; and the other parameters are as above.

As can be seen, the model is a multivariate linear regression equation. Thus, the unknown coefficient can be calibrated using the least square method. The Matlab software package was used for the data mining process. The result is as follows:

$$\begin{aligned} d_i = {} & 1.16\frac{C(1-\lambda_i)^2}{2(1-\lambda_i x_i)} + 0.66\frac{x_i^2}{2q_i(1-x_i)} \\ & - 0.79\left(\frac{C}{q_i^2}\right)^{1/3} x_i^{2+5\lambda_i}. \end{aligned} \tag{9}$$

The green time split λ_i is given as

$$\lambda_i = \frac{g_i}{C} = \frac{C-L}{C}\frac{y_i}{Y} = \left(1-\frac{L}{C}\right)\frac{y_i}{Y}, \tag{10}$$

where L represents the total lost time; g_i represents the green time for phase i; y_i represents the maximum traffic volume ratio for phase i, which can be calculated by $y_i = q_i/s_i$; and Y represents the sum of traffic volume ratio of all phases.

Thus, the saturation in (6) can be calculated as

$$\begin{aligned} x_i &= \frac{q_i}{\lambda_i s_i} = \frac{q_i}{s_i(1-L/C)(y_i/Y)} = \frac{y_i}{(1-L/C)(y_i/Y)} \\ &= \frac{Y}{(1-L/C)}. \end{aligned} \tag{11}$$

Therefore, (9) can be rewritten as

$$\begin{aligned} d_i = {} & 1.16\frac{C(1-\lambda_i)^2}{2(1-y_i)} + 0.66\frac{(Y/(1-L/C))^2}{2q_i(1-Y/(1-L/C))} \\ & - 0.79\left(\frac{C}{q_i^2}\right)^{1/3}\left(\frac{Y}{1-L/C}\right)^{2+5\lambda_i}. \end{aligned} \tag{12}$$

The assumption for the optimization cycle length model is that the total delay of all the phases should be minimized, which is given as

$$\text{Min}\, d_{\text{total}} = \text{Min} \sum_i q_i d_i, \tag{13}$$

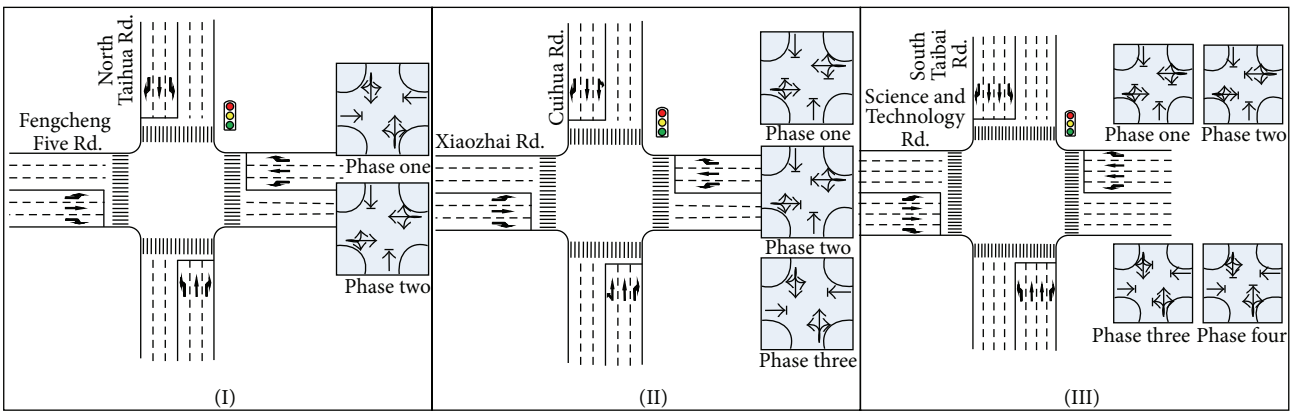

Figure 1: Geometric characteristics and signal phase of the selected sites.

where d_{total} represents the total delay time of all phases; q_i represents the traffic volume; and d_i represents the delay time of phase i.

As can be seen from (12), the total delay d_{total} is a function of only one parameter C. Therefore, the optimum of (13) can be found at $\partial d_{\text{total}}/\partial C = 0$.

Thus, the parameter C is given as

$$C = \frac{1.45L + 3}{1 - Y}, \tag{14}$$

where L represents the total lost time and Y represents the sum of traffic volume ratio of all phases.

According to (9), when the saturation x_i is close to 1, the delay will tend to be infinity. It is implausible and not realistic. Thence, (14) does not apply to all the traffic conditions. The cycle length model should be rewritten as follows:

$$C = \frac{1.45L + 3}{1 - Y}, \quad \text{subject to } C_{\min} \le C \le C_{\max}. \tag{15}$$

The $C_{\min}$ and $C_{\max}$ are disused as follows.

(1) When the traffic volume is low, the important consideration is the pedestrian rather than the vehicle. Therefore, the crossing time for the pedestrian should be focused on. The minimum green time for the pedestrian crossing on the street is given as

$$g_{\min} = 7 + \frac{L_p}{v_p}. \tag{16}$$

Therefore, the cycle length can be calculated as

$$C_{\min} = \sum_i \left(7 + \frac{L_p}{v_p}\right) + L, \tag{17}$$

where L_p represents length of the crossing street for the pedestrians; v_p represents the average speed of the pedestrian; and L represents the total lost time.

(2) When the saturation is large, which represents a crowded traffic flow environment, it is no longer able to solve the problem of traffic congestion by optimal cycle length. If the signal control remains, the main factor for determining the cycle length should focus on avoiding the anxiety of drivers for waiting a long time. Therefore, the suggested optimization cycle length is 180 seconds [4, 20]; that is, $C_{\max} = 180$.

In summary, the optimization traffic signal cycle length model is given as

$$C = \frac{1.45L + 3}{1 - Y}, \quad \text{subject to } C_{\min} \le C \le C_{\max}, \tag{18}$$

with

$$\begin{aligned} C_{\min} &= \sum_i \left(7 + \frac{L_p}{v_p}\right) + L, \\ C_{\max} &= 180. \end{aligned} \tag{19}$$

Note that the development process of optimization traffic signal cycle length model is not dependent on any particular phase-control scheme. Therefore, it is for 2-phase-, 3-phase-, and 4-phase-control signalized intersection.

The green time for the above three cases is determined as follows.

The total green time is $g_{\text{total}} = C - L$; thus the green time for phase i is given as

$$\begin{aligned} g_i &= g_{\min,i} = 7 + \frac{L_{p,i}}{v_{p,i}} \quad \text{for } c = C_{\min} \\ g_i &= (C - L)\frac{y_i}{Y} \\ &\qquad \text{for } c = C,\ c = C_{\min},\ \text{subject to } g_i \ge g_{\min,i}. \end{aligned} \tag{20}$$

4. Evaluation of the Optimization Traffic Signal Cycle Length Model

4.1. Simulation Experimental Design. In order to evaluate the effect of the optimization traffic signal cycle length model (termed "NEW model"), the traffic microsimulation technology was used. The experiment was conducted using the simulation software package VISSIM. Three signalized intersections were selected for the simulation models. Geometric characteristics and phase-control scheme of the three selected sites are shown in Figure 1. Traffic volume and the cycle length

Table 2: Information for the selected signalized intersections.

Intersection	Approaches	Traffic volume (pcu/h)			Cycle length (sec)	
		LT	TM	RT	TRRL model	NEW model
Fengcheng Five Rd. and North Taihua Rd. (I)	Westbound	72	341	124	30	40
	Northbound	73	545	151		
	Eastbound	61	350	117		
	Southbound	55	474	157		
Xiaozhai Rd. and Cuihua Rd. (II)	Westbound	225	1278	203	140	120
	Northbound	80	959	321		
	Eastbound	240	1124	265		
	Southbound	91	934	301		
Science and Technology Rd. and South Taibai Rd. (III)	Westbound	196	1420	237	230	180
	Northbound	320	1555	263		
	Eastbound	313	1312	526		
	Southbound	229	1555	274		

are given in Table 2. Note that the cycle length was calculated using the NEW model. As a comparison, the cycle length calculated using the TRRL model, which is the traditional traffic signal cycle length model, was also given in Table 2.

The intersection of Fengcheng Five Rd. and North Taihua Rd. (termed "I") is a no-busy intersection with a low traffic volume. The intersection of Science and Technology Rd. and South Taibai Rd. (termed "III") is a very busy intersection with a high traffic volume. The intersection of Xiaozhai Rd. and Cuihua Rd. (termed "II") is a medium traffic flow intersection. The three signalized intersections, which were carefully selected, can meet the different case of the optimization traffic signal cycle length model. The phase-control schemes for the three intersections are 2-phase-control, 3-phase-control, and 4-phase-control schemes, respectively. Therefore, the selected intersection can also verify the adaption of the optimization cycle length model for all phase-control schemes.

There were two signal programs (termed "TRRL model program" and "NEW model program") for each intersection. The traffic volume, phase design, and other signal timing algorithm were the same. The only difference was the cycle length, for which one program used the TRRL model, while the other program using the NEW model. For both signal programs, five one-hour simulation runs were completed with unique random seeds. Of the five simulation runs, the ones with the highest and the lowest resulting delay times are discarded and the remaining three were averaged to obtain the final results.

4.2. Statistical Methods. t-tests were conducted to identify whether the difference in delay time and the queue length at signalized intersections with the TRRL model and the NEW cycle length model was statistically significant. The null hypothesis of the tests was that the average delay time (queue length) based on the two programs is the same. The null hypothesis can be rejected if [21]

$$t = \frac{\left|\overline{X_1} - \overline{X_2}\right|}{\sqrt{s_1^2/n_1 + s_2^2/n_2}} \geq t_{\alpha/2}, \tag{21}$$

where α is the level of significance; $\overline{X_1}$ and $\overline{X_2}$ are the average delay time (queue length) with the TRRL model and the NEW model, respectively; s_1 and s_2 are the sample standard deviations; n_1 and n_2 are the number of simulation data with TRRL model and the NEW model, respectively; and $t_{\alpha/2}$ is the $100(1-\alpha/2)\%$ percentile of the t distribution with degrees of freedom given by

$$\mathrm{df} = \frac{\left(s_1^2/n_1 + s_2^2/n_2\right)^2}{\left(s_1^2/n_1\right)^2/\left(n_1-1\right) + \left(s_2^2/n_2\right)^2/\left(n_2-1\right)}. \tag{22}$$

5. Data Analysis Results

Through the microsimulation using VISSIM software package, the research team obtained 900 data records for delay time and 900 data records for queue length, both of which were used for future data study. Cross-sectional analysis was conducted to compare the delay time and queue length of the two programs.

5.1. Result of Delay Time. The summary statistics of the delay time for each selected intersection based on the two programs was shown in Table 3. The average delay timea for the selected intersections based on the TRRL model program were 3.64, 34.51, and 50.13, respectively. Correspondingly, the average delay times for the selected intersections based on the NEW model program were 4.27, 31.08, and 42.58, respectively. The data in Table 3 suggested that the average delay times of II intersection and III intersection based on the NEW model program were smaller than those based on the TRRL model program. However, the average delay time of I intersection based on the NEW model program was larger than that based on the TRRL mode program.

A series of t-tests were conducted to identify whether the differences in the delay times between the different programs were statistically significant. The results of the t-tests were shown in Table 4. With a 95% level of confidence, the t-tests results showed that the difference between delay times of the two programs for II intersection and III intersection was found to be statistically significant. However, the difference

Table 3: Summary statistics of delay time (sec/per vehicle).

Model	Statistical index	Intersection	Sample size	Maximum	Minimum	Mean	Standard
TRRL model	Delay time	I	120	13.30	0.20	3.64	2.47
		II	150	64.8	5.4	34.51	13.04
		III	180	144.9	20.6	50.13	25.84
NEW model	Delay time	I	120	11.6	0.2	4.27	2.61
		II	150	79.4	9.5	31.08	12.57
		III	180	135.2	19.4	42.58	20.63

Table 4: Results of t-tests for delay time.

Intersection	Statistical index	Mean value		P value
		TRRL model	NEW model	
I	Delay time	3.64	4.27	0.054
II	Delay time	34.51	31.08	0.021
III	Delay time	50.13	42.58	0.002

between delay times of the two programs for I intersection was found to be not statistically significant. The results indicate that the NEW model significantly affected the delay time and was more optimal than that of the TRRL model.

5.2. Result of Queue Length. The summary statistics of the queue length for each selected intersection based on the two programs was shown in Table 5. The average queue lengths for the selected intersections based on the TRRL model program were 2.74, 8.12, and 15.98, respectively. Correspondingly, the average queue lengths for the selected intersections based on the NEW model program were 3.24, 6.04, and 10.02, respectively. According to the data in Table 5, the queue lengths of II intersection and III intersection based on the NEW model program were smaller than those based on the TRRL model program. However, the queue length of I intersection based on the NEW model program was larger than that based on the TRRL model program.

To identify whether the differences in queue lengths between different programs were statistically significant, t-tests were then conducted. The results of t-tests were shown in Table 6. With a 95% level of confidence, the t-tests results showed that the difference between queue lengths of the two programs for II intersection and III intersection was found to be statistically significant. However, the difference between queue lengths of the two programs for I intersection was found to be not statistically significant. The results were similar to the results of delay time. The result further demonstrated that the NEW model is more optimal than the TRRL model.

6. Summary and Conclusions

The study presented in this paper proposed an optimization traffic signal cycle length model for signalized intersection. The effect of the new model was studied through the use of traffic microsimulation technology. Cross-sectional analysis was conducted to compare the impacts of both the traditional traffic signal cycle length and the optimization traffic signal cycle length model. Based on the results of this study, the following conclusions were reached.

(1) An optimization traffic signal cycle length model was proposed. Using comprehensive delay data, the model from Webster for optimal cycle time is re-recalibrated to the Chinese traffic conditions. Not only does the optimization model take the vehicle delay time into consideration, but also the lower boundary considered the factors of pedestrian crossing and the upper boundary considered the drivers' anxiety for waiting a long time which is related to traffic safety, especially to risk-taking behavior.

(2) The results of the microsimulation and cross-sectional analysis suggested that the difference in the delay times and queue lengths of II and III signalized intersections with the optimization cycle length model was significantly smaller than that in the TRRL model. It suggested that the optimization traffic signal cycle length model was more optimal than the TRRL model since it took the different traffic conditions of the intersections into consideration.

The work presented in this paper provided insight into the traffic signal cycle length model. In China, the traffic congestion at the intersection resulted in many problems, such as time delay, air pollution, and even traffic safety. A main reason for that is the unreasonable traffic signal cycle length. Findings of this study provide an optimization traffic signal cycle length model considering the different traffic conditions. These findings are helpful for traffic engineers and traffic police to design a more reasonable cycle length for a signalized interaction given its traffic status.

However, there are several limitations in the present study. The data used in the study was collected in one Chinese city. Similar studies should be done in other cities in China. In addition, the proposed traffic signal cycle time length model did not consider the effects of nonmotorized vehicles, which is mixed motorized vehicles at the intersections. Future work will focus on these two issues.

Table 5: Summary statistics of queue length (vehicles/per cycle).

Model	Statistical index	Intersection	Sample size	Maximum	Minimum	Mean	Standard
TRRL model	Queue length	I	120	14	0	2.74	2.79
		II	150	30	2	8.12	4.87
		III	180	38	3	15.98	7.16
NEW model	Queue length	I	120	12	0	3.24	3.02
		II	150	25	2	6.04	3.66
		III	180	24	2	10.02	4.39

Table 6: Results of t-tests for queue length.

Intersection	Statistical index	Mean value		P value
		TRRL model	NEW model	
I	Queue length	2.74	3.24	0.184
II	Queue length	8.12	6.04	<0.001
III	Queue length	15.98	10.02	<0.001

Conflict of Interests

The authors declare that there is no conflict of interests.

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