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Research Article

Road Impedance Model Study under the Control of Intersection Signal

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Road traffic impedance model is a difficult and critical point in urban traffic assignment and route guidance. The paper takes a signalized intersection as the research object. On the basis of traditional traffic wave theory including the implementation of traffic wave model and the analysis of vehicles' gathering and dissipating, the road traffic impedance model is researched by determining the basic travel time and waiting delay time. Numerical example results have proved that the proposed model in this paper has received better calculation performance compared to existing model, especially in flat hours. The values of mean absolute percentage error (MAPE) and mean absolute deviation (MAD) are separately reduced by 3.78% and 2.62 s. It shows that the proposed model has feasibility and availability in road traffic impedance under intersection signal.

1. Introduction

Road traffic impedance function refers to the relationship between road traveling time and traffic load pressure, which is an important basis of traffic assignment and route guidance. Precise road traffic impedance calculation is a critical technology to make reasonable traffic control decision. It may also effectively avoid wasting travel time and reduce the contamination of environment. With the increasing development of intelligent traffic system (ITS), the signalized intersections become increasingly prevalent in urban roads. Therefore, the study of road traffic impedance grows into a research hotspot. Currently, the road traffic impedance can be divided into two parts: basic travel time and waiting delay time [1].

For the research of basic travel time, the BPR function model proposed by the Federal Highway Administration is the most classical and popular. It is researched by considering the impact of traffic load on the basis of free flow. The research of BPR function has made much progress considering the complex situation of urban traffic flow in China [2]. However, there are still some certain limitations and poor generalization, which still need further study.

For the research of waiting delay time under signalized intersection, the Webster and HCM (1985 and 2000) formulas are typical and widespread. The former is suitable for low traffic load, while the latter is opposite. However, those formulas are not suitable for the intersections in China. The study focuses on improving the classical formulas which have obtained some effects in a number of actual applications [3].

Traffic wave theory is an important branch of traffic flow theory. It describes that the encounter of two traffic flows in the different status with same direction of movement is the transfer process of status which also adheres to the energy conservation law of traffic flow. Thus, the study of road traffic function based on traffic wave theory has attracted wide attention at home and abroad because of its universal adaptability. Traditional traffic wave theory is based on Greenshields linear model [1, 4–9]. Carey and McCartney [1] have analyzed the characteristics of road traffic impedance under the control of intersection signal. Yang [4] has constructed the parking and staring wave model based on Greenberg model, based on analyzing the defects of Greenshields in heavy traffic situation. Yuan [5], Li, and Song [6] and Song et al. [7] have proposed that the road section can be divided into two parts based on queue length: congested and uncongested road

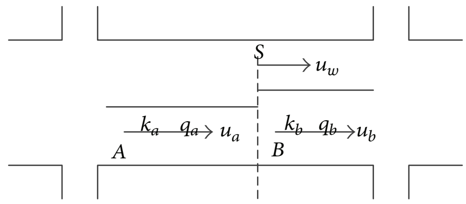

Figure 1: The operation of two different traffic densities.

sections. Then, the road traffic impedance can be calculated separately by each part. It maybe inevitably causes a double error. Wang et al. [8] give the explanations of gathering and dissipating of signalized intersection and analyze different modes under different vehicle tracks. However, it is limited to theoretical research. Sun and Liu [9] have proposed the phase control of intersection signal. The references [10–13] have further analyzed the application of road traffic impedance in intelligent speed estimation, traffic road assignment, and security evaluation of intersection and road network.

In summary, considering the popularity of traditional traffic wave theory, road traffic impedance model under the control of intersection signal is researched based on the Greenshields traffic wave theory with the purpose of obtaining the basic travel time and waiting delay time. The measured data of actual traffic flow and road traffic impedance, in Shenzhen city of Guangdong province of China, is used to conduct calculation and some necessary comparative analysis with the existing model.

The rest of this paper is organized as follows. Section 2 describes the traffic wave theory based on Greenshields model. Section 3 provides the implementation of road impedance model. Section 4 introduces numerical experiment including data collection and results analysis. The conclusion is given in Section 5.

2. Traffic Wave Theory Based on Greenshields Model

2.1. Implementation of Traffic Wave Model. As shown in Figure 1, a road section includes two different areas of traffic density (k_a & k_b). The formation of traffic wave is based on the vehicles movement from a density zone (A zone) to another density zone (zone B). The cross section to segment the two densities is known as the wave-front (S). The speed of the wave-front is named wave velocity (u_w). From the energy conservation of traffic flow, the number of vehicles which have traveled through the wave-front within time t can be calculated as N:

$$N = (u_a - u_w) k_a t = (u_b - u_w) k_b t. \tag{1}$$

Simplify the equation as follows:

$$u_w = \frac{u_b k_b - u_a k_a}{k_b - k_a}. \tag{2}$$

On the basis of basic relationships between three parameters of traffic flow, the equation could be simplified as follows:

$$u_w = \frac{q_b - q_a}{k_b - k_a}. \tag{3}$$

Traditional traffic wave model is derived according to the Greenshields model. In order to simplify the equation, the intensity is normalized. In that way, the Greenshields speed-density model is further calculated as follows:

$$u = v_f \left(1 - \frac{k}{k_j}\right) = v_f (1 - \eta). \tag{4}$$

By formulas (3) and (4), travel wave model will be obtained as follows:

$$u_w = v_f \left[1 - (\eta_a + \eta_b)\right]. \tag{5}$$

The parking and starting wave models are researched on the basis of the following analysis.

When the red signal starts, vehicles will stop and gather at the front of parking line with generating a backward propagating wave which is called parking wave. The road section is blocked at the left of wave-front with the standardized density $\eta_b = 1$. The parking wave model will be obtained as follows:

$$u_0 = v_f \left[1 - (\eta_a + 1)\right] = -v_f \eta_a. \tag{6}$$

When the green signal starts, vehicles will start and dissipate with a forward starting wave. The road section is blocked at the right of wave-front with the standardized density $\eta_a = 1$. Meanwhile, u_b is too small to be negligible.

By formula (4), η_b will be obtained as follows:

$$\eta_b = 1 - \left(\frac{u_b}{v_f}\right). \tag{7}$$

Thus, the starting wave model will be obtained as follows:

$$u_1 = -v_f \eta_b = -\left(v_f - u_b\right) \approx -v_f. \tag{8}$$

In the above formulas, the parameters will be shown as follows:

- u: average speed of road section.
- v_f: travel speed of free flow.
- k_j: blocking density.
- η: standardized density.
- η_a, η_b: standardized density of different adjacent regions.

2.2. Vehicles' Gathering and Dissipating Analysis. As shown in Figure 2, the situation of vehicles' gathering and dissipating under intersection signal will be analyzed. Suppose that there are no vehicles before the red light starting. When the red light starts, vehicles will stop at the parking line and produce parking wave. With the arrival of vehicles at back, the vehicles start queuing. When the green light starts, vehicles will move

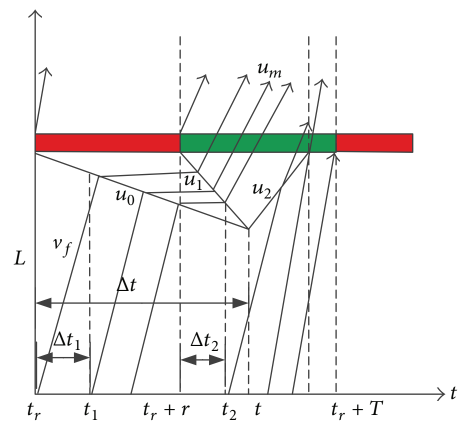

FIGURE 2: Time-space map of signalized intersection.

and produce starting wave. Meanwhile, the length of the queue reaches the maximum when the two traffic waves encounter. Subsequently, the influx of vehicles behind will catch up with the dissipating vehicles and produce forward wave. When forward wave spreads to the stop line, vehicles will arrive at the speed of free flow.

Of course, the absolute value of starting wave velocity must be bigger than parking wave; otherwise, vehicles cannot dissipate forever. At the same time, the parking and starting wave must encounter before the end of the green light; otherwise, vehicles cannot be dissipated within a signal period which will cause queuing more than twice. In this paper, vehicles queue is only considered at most once.

In Figure 2, the parameters will be shown as follows:

u_m: speed of dissipating vehicles.

u_2: speed of forward wave.

T: time of signal period.

t_r: the moment of the red light starting.

r: time of the red light.

t: moment of the two waves encountering.

t_1: moment of the vehicle i encountering parking wave.

t_2: moment of the vehicle i encountering starting wave.

L: length of road section.

3. Implementation of Road Traffic Impedance Model

Road traffic impedance under signal control will be divided into two parts: basic travel time and waiting delay time.

Set ΔT as road impedance, set ΔT_m as basic travel time, and set ΔT_d as waiting delay time; then, ΔT will be expressed as follows:

$$\Delta T = \Delta T_m + \Delta T_d. \tag{9}$$

3.1. Implementation of Basic Travel Time. Suppose that the speed of the road section is adopted average speed (u) which will solve the difficult problem of measuring the speed (u_m) of dissipating vehicles. Thus, ΔT_m can be easily expressed as follows:

$$\Delta T_m = \frac{L}{u} = \frac{L}{v_f(1-\eta)}. \tag{10}$$

For further calculation, the standardized density η can be obtained on the basis of basic relationship of traffic flow parameters and Greenshields linear model which is shown as the following formula:

$$q = uk = v_f\left(1 - \frac{k}{k_j}\right)k. \tag{11}$$

So, density k can be obtained through the above solution of a quadratic equation which is shown as follows:

$$k_1 = \frac{1}{2}k_j\left(1 - \sqrt{1 - \frac{4q}{k_j v_f}}\right),$$
$$k_2 = \frac{1}{2}k_j\left(1 + \sqrt{1 - \frac{4q}{k_j v_f}}\right). \tag{12}$$

In the current study, k usually appears as absolute value without distinguishing the differences between k_1 and k_2. Absolutely, k_1 is the density of normal driving range and k_2 is the density of blocking interval in which the speed of vehicles is almost close to 0. Thus, k_1 is chosen in this paper considering the actual road section. Then, η is shown as follows:

$$\eta = \frac{k}{k_j} = \frac{1}{2}\left(1 - \sqrt{1 - \frac{4ql}{v_f}}\right). \tag{13}$$

In all, basic travel time will get as follows:

$$\Delta T_m = \frac{L}{v_f\left[1 - (1/2)\left(1 - \sqrt{1 - 4ql/v_f}\right)\right]}. \tag{14}$$

3.2. Implementation of Waiting Delay Time. Suppose that, during observation period, the number of vehicles is N and the probability of vehicles with waiting delay time is p. Thus, ΔT_d can be expressed as follows:

$$\Delta T_d = p\sum_{i=1}^{N}\Delta t_{di}, \quad i = 1, 2, \ldots, n. \tag{15}$$

When the vehicle arrives at the road section and encounters the parking wave, it will stop behind the current end of the queue. After waiting a period of time, the vehicle will encounter starting wave and drive away from the intersection. When the vehicle arrive at the range $[t_r, t]$, the vehicle will produce waiting delay time and the probability (p) will be expressed as follows:

$$p = \frac{\Delta t}{r + g}. \tag{16}$$

It can be further calculated by the following formulas:

$$u_0 \Delta t = u_1 (\Delta t - r)$$
$$\Delta t = \frac{u_1 r}{u_1 - u_0} \tag{17}$$
$$p = \frac{r}{(r+g)(1-\eta)}.$$

At the same time, the waiting delay time of the vehicle i will be expressed as follows:

$$\Delta t_{di} = t_2 - t_1 = \Delta t_2 + r - \Delta t_1. \tag{18}$$

It can be further calculated by the following formulas:

$$\Delta t_1 = t_1 - t_r,$$
$$\Delta t_2 = t_2 - (t_r + r)$$
$$u_0 \Delta t_1 = u_1 \Delta t_2 \tag{19}$$
$$\Delta t_{di} = r - \left(1 - \frac{u_0}{u_1}\right) \Delta t_1 = r - (1-\eta) \Delta t_1.$$

Assuming that the vehicles will queue at most once, no matter when they arrive at the intersection; that is, the maximum waiting delay time of each vehicle is r. Thus, waiting delay time of each observation period will be obtained as follows:

$$\begin{aligned} \Delta T_d &= p \sum_{i=1}^{N} \Delta t_{di} \\ &= p \frac{1}{r} \int_0^r [r - (1-\eta) \Delta t_1] \, d\Delta t_1 \\ &= p \frac{r(1+\eta)}{2} = \frac{1}{2} \frac{r^2 (1+\eta)}{(r+g)(1-\eta)}. \end{aligned} \tag{20}$$

In summary, road traffic impedance is shown as follows:

$$\begin{aligned} \Delta T &= \Delta T_m + \Delta T_d \\ &= \frac{L}{v_f \left[1 - (1/2)\left(1 - \sqrt{1 - 4ql/v_f}\right)\right]} \\ &\quad + \frac{r^2 \left[1 + (1/2)\left(1 - \sqrt{1 - 4ql/v_f}\right)\right]}{2(r+g)\left[1 - (1/2)\left(1 - \sqrt{1 - 4ql/v_f}\right)\right]}. \end{aligned} \tag{21}$$

4. Numerical Example

4.1. Data Collection and Calculation Results. Firstly, the road section containing a signalized intersection in Shenzhen city of Guangdong province is selected as data observation object. The purpose of the experiment is to obtain road traffic impedance of each period (one hour) under the signal control. Take the north import of the intersection as an example; the collected data from vehicle inspection device is only for straight and left turning vehicles, because there exists right turning channelization in the intersection. The green phase of north import is 48 s and a signal cycle is 108 s. According to the road speed of actual limit situation, the free flow speed is 40 km/h. The car spacing is 2 m when the section reaches blocked state. The average length of the car is 5 m. The capacity of north import is 1600 veh/h.

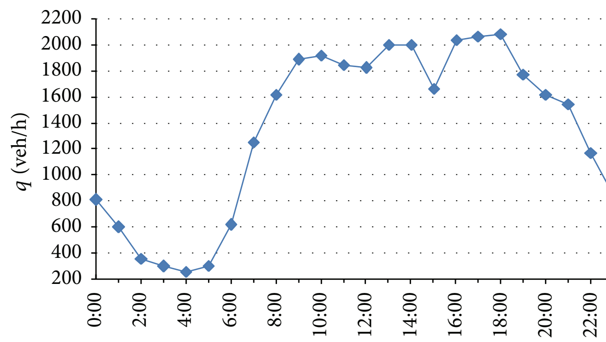

Figure 3: Average traffic flow in 24 hours a day.

Subsequently, set 500 m upstream of the intersection as the observation point of road import; in the meanwhile, intersection export is set as the observation point export. The road traffic impedance of every vehicle is obtained through the video capturing license plate of the import and export vehicles.

Thirdly, the data of traffic flow can be obtained from vehicle inspection device on the selected road section.

At last, it is necessary to count the data of road traffic impedance and traffic flow. In order to get more accurate results, five working days are selected to get the average data of one day and the data of road traffic impedance and traffic flow for each period (one hour) are collected. The traffic flow is shown in Figure 3 and actual road traffic impedance is shown in Figure 4 (no separate figure in order to avoid duplication).

In Figure 3, 0:00 represents the time period [23:00-0:00], 1:00 represents the time period [0:00-1:00], 2:00 represents the time period [1:00-2:00], 3:00 represents the time period [2:00-3:00],. . ., 21:00 represents the time period [20:00-21:00], 22:00 represents the time period [21:00-22:00], and 23:00 represents the time period [22:00-23:00]. From the curve trend of traffic flow and road traffic impedance, it can be seen that the two curves have the same trend. At the same time, the morning peak hour remains at [9:00-10:00], the noon peak hour remains at [13:00-14:00], and the evening peak remains at [17:00-18:00]. The other periods are flat hours.

4.2. Results and Analyses. To evaluate the calculation performance of each model, relative error (RE), mean absolute deviation (MAD), and mean absolute percentage error (MAPE) are calculated as follows and histogram for 24 periods is shown in Figure 5:

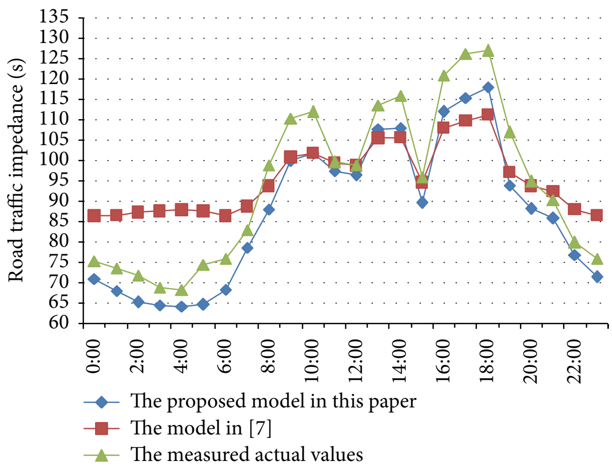

Figure 4: The road traffic impedance curve graph.

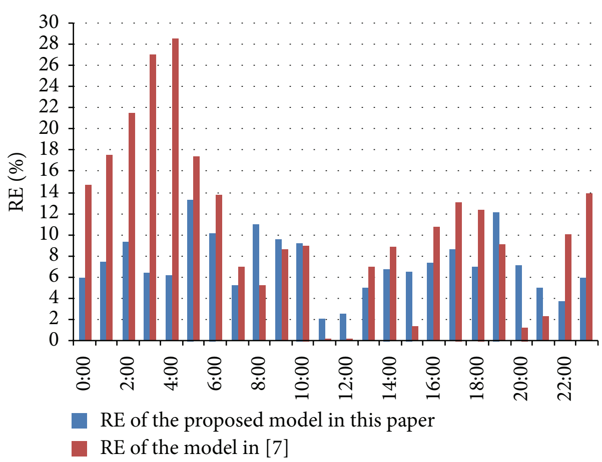

Figure 5: Different road traffic impedance models relative error comparison.

$$\text{RE} = \left| \frac{\widehat{Y}_i - Y_i}{Y_i} \right| \times 100\%$$
$$\text{MAD} = \frac{\sum_{i=1}^{n} \left| \widehat{Y}_i - Y_i \right|}{n} \qquad (22)$$
$$\text{MAPE} = \frac{\sum_{i=1}^{n} \left| \left(\widehat{Y}_i - Y_i \right) / Y_i \right|}{n} \times 100\%,$$

in which $\widehat{Y}_i$ is the computed value of each period for observation i, Y_i is the actual value for observation i, and n is the number of computed periods.

Figure 4 shows the curves of the measured actual values and computed values in the two different models (the model in this paper and the model in [7]). The fluctuation of the model in this paper is similar with traffic flow as well. Meanwhile, it is much closer to the measured actual values. So, it is in line with actual road section which demonstrates the feasibility of this model.

Table 1: Statistical table of error indexes.

Hours	Error indexes	The model in this paper	The model in [7]
Flat hours	MAD/s	6.54	9.16
	MAPE/%	7.20	10.98
Peak hours	MAD/s	9.03	11.98
	MAPE/%	7.67	10.04
The whole hours	MAD/s	6.85	9.51
	MAPE/%	7.25	10.86

Figure 5 shows the histogram of relative error (RE) values for 24 periods. As shown in Figure 5, the maximum relative error of the model in [7] is more than 28%, while the model in this paper is less than 14%. In order to further analyze the effectiveness of the two different models in the whole hours, flat hours, and peak hours, it is necessary to establish statistical table of error indexes as shown in Table 1.

In Table 1, on one hand, through horizontal analysis, the MAPE values of the model in this paper are reduced separately by 3.78% and 2.37% in flat hours and in peak hours. Meanwhile, the MAD values are separately reduced by 2.62 s and 2.94 s in flat hours and in peak hours. In the whole hours, the MAPE and MAD values are reduced separately by 3.61% and 2.67 s. By horizontal comparison, the model in this paper presents better performance than the model in [7], especially in flat hours.

On the other hand, through longitudinal analysis of the model in this paper, the MAPE value in flat hours is reduced separately by 0.47% and 0.06% less than in peak hours and the whole hours, while the MAD value is decreased by 2.49 s and 0.31 s. By longitudinal comparison, the model in the paper shows excellent feasibility and applicability whether in flat hours, peak hours, or the whole hours.

5. Conclusion

Real-time and accurate calculating of road impedance under the control of intersection signal is effective in guiding travel and saving time and energy. This paper proposes road impedance model based on traditional traffic wave theory. Meanwhile, numerical example is employed to conduct the comparison between different models and measured actual situation. The result shows that the model presents better performance. It makes up the deficiency of a double error in the current study. The proposed model is feasible and has a certain engineering value as well.

It should be noticed that waiting delay time under intersection signal is obtained through traditional parking and starting wave theory. Meanwhile, the calculation of basic travel time is done by the average speed of road section which is also based on Greenshields linear relationship. It is well known that Greenshields model is more suitable for the traffic situation of medium density. In this paper, the model has obtained some certain effect at the circumstances of queuing at most once, even though there exist some errors. The model of Greenshields relationship in more severe traffic condition still needs to be further researched in urban road section of China.

However, the road traffic impedance under the control of signalized intersection is also affected by other complex factors such as weather conditions, driver or pedestrian behaviors, and emergencies. We still have a long way to go on the research.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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