Multi-objective Optimization of Traffic Signal Systems on Urban Arterial Roads

Tao Liu, Feng Qiao, Lingzhong Guo, and Yifan Chen

Abstract In this paper, multi-objective optimization is used to solve the signal synchronization problem in arterial traffic roads, where a traffic dispersion module is introduced to further expand the solution space. By incorporating the models of delay time, queue length and stop times into the optimization, a first model called M1 is established. In the second model M2, the free flow speed assumption is replaced by a traffic dispersion module for better estimating the link travel time. A simulation study is then carried out on an arterial road, and the results show that the proposed strategy improves the performance of the traffic system compared to the current timing scheme and M2 has the best performance among all solutions in this paper, and the delay is reduced for about 24%.

Keywords Multi-objective optimization · Arterial urban traffic · Signal timing scheme

1 Introduction

Urban arterial roads are an important part of the urban transportation system and bear the main traffic load of an entire city. An effective signal control strategy is critical for ensuring a higher traffic capacity. Limited by urban space and economic practicability, the infrastructure load capacity of urban highways cannot permanently be kept above the increasing traffic flow. Therefore, the key to solving the problem of traffic congestion in the urban road network is to reduce the traffic congestion in the series of intersections on an arterial road [1].

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Saka et al. [2] classified the intersections according to different traffic conditions and carried out signal control for each situation. A fuzzy hierarchical control method of urban road intersections was proposed by Kim [3], where he adjusted the control strategy in real time according to different traffic flow conditions and applied the genetic algorithm (GA) to the fuzzy control of intersection signals in [4] to improve the performance of the fuzzy controller. Genetic algorithm was also used to study the real-time adaptive control optimization method of traffic signals in [5]. Based on multi-intelligence framework, Khamis et al. [6] studied the adaptive multi-objective enhanced learning traffic signal optimization control method and verified its application in the experimental platform built. Aiming at obtaining wider green wave bandwidth, Qiao et al. [7] adopted particle swarm optimization (PSO) to conduct bidirectional green wave optimization for arterial traffic system.

In this paper, considering the model of delay time, queue length and stop times, a model M1 is constructed to solve the multi-objective optimization problem of arterial traffic system. Then, in order to provide more practical results, the traffic dispersion module is adopted to estimate the link travel time in the model M2. Using the non-dominated sorting genetic algorithm-II (NSGA-II) [8] and commercial optimization software, feasible solutions can be found quickly. Simulation studies on a sample road are made with VISSIM to verify the effectiveness of the proposed coordination traffic signal timing scheme. The results show that model M2 provides the best time scheme.

The rest of the paper is organized as follows. In Sect. 2, the multi-objective optimization issue is formulated as model M1 for the arterial traffic system; in Sect. 3, we formulate the extended model M2 by introducing traffic dispersion module into model M1; in Sect. 4, the simulation results are presented and analyzed. Finally, the conclusions are drawn in Sect. 5.

2 Problem Statements

According to the actual traffic situation, the optimization target of arterial system is generally to minimize the average delay or to maximize the green wave bandwidth. The minimum delay method is devoted to reducing vehicle delays at each intersection by reasonably allocating the period and offset of each intersection on the main road, while the maximum green wave zone method is devoted to increasing the number of vehicles passing through each green wave time [9]. In this paper, based on the minimum delay method, the models of line delay, queue length and stop times are incorporated into the multi-objective optimization problem.

The average delay model of the arterial traffic systems is expressed as two parts: the upward delay and the downward delay. This delay model has a good optimization effect for the traffic situation with unequal traffic flow on each road [10]. Now define D_u and D_d as the upstream delay time and the downstream delay time, respectively; denote l as queue length; and H as the stop times. The multi-objective function (1) is to minimize D_u, D_d, l and H

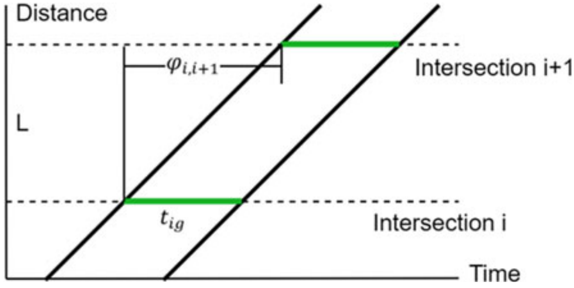


Fig. 1 Geometric presentation of adjacent intersection

$$z = \min[D_u, D_d, l, H] \tag{1}$$

As shown in Fig. 1, let $\varphi_{i,i+1}$ be the offset between intersection i and intersection $i+1$, $t_g(t_{ig})$ be green time (of intersection i), $t_r(t_{ir})$ be red time (of intersection i), C be the length of cycle, L be the distance between intersection i and $i+1$, v be free flow speed. If the light is turned red when the front of traffic flow arrives at intersection $i+1$, let t_u be the waiting time of the traffic flow, $t_{i,i+1}$ be the link travel time from intersection i to intersection $i+1$.

$$t_{i,i+1} = \frac{L}{v} \tag{2}$$

$$t_u = \varphi_{i,i+1} - t_{i,i+1} \tag{3}$$

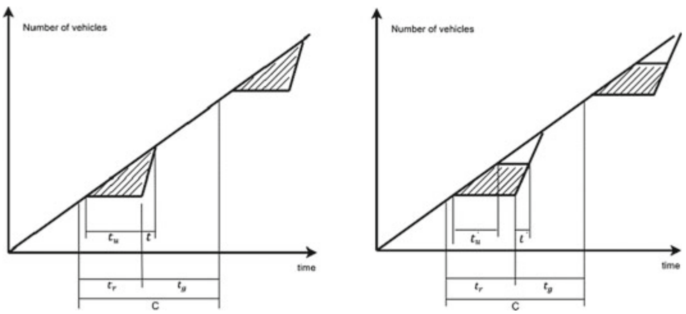
Let t be the evacuation time of vehicles queuing after the green light; q_u be the actual traffic flow in the upstream direction; q_{um} be the upstream saturation flow.

$$t = \frac{t_u q_u}{q_{um} - q_u} \tag{4}$$

It shows, in Fig. 2a, a schematic diagram of the delay time when the front of traffic flow is blocked by the intersection $i+1$. In this case, we need to improve $\varphi_{i,i+1}$ to advance the green phase, so that the traffic flow will meet the green light when arriving at the intersection $i+1$. The total delay time is represented as the area of shaded area in Fig. 2a.

Unless otherwise specified, a and a' represent the same variable in different case of vehicle delay in Fig. 2. Let d_{iu} (d'_{iu}) be the upstream delay time of the vehicle arriving at intersection i at the red (green) light. Let t_{ir} be the time of red light at the intersection i. By combining Eqs. (3) and (4), we have

$$d_{i+1,u} = \frac{q_u q_{um}\left(\varphi_{i,i+1} - t_{i,i+1}\right)^2}{2(q_{um} - q_u)} \tag{5}$$



(a) All vehicles are blocked (b) Some vehicles are blocked

Fig. 2 Delay time. **a** All vehicles are blocked, **b** some vehicles are blocked

It shows, in Fig. 2b, a schematic diagram of the delay time when the end of traffic flow is blocked by the intersection i. In this case, we need to reduce $\varphi_{i,i+1}$ to delay the green phase, so that the traffic flow can pass through the intersection in the green time. The total delay time is represented as the area of shaded area in Fig. 2b. Let t_{u}' be the time taken for the end of the traffic flow to pass through the intersection $i+1$.

$$t_{\mathrm{u}}' = t_{i,i+1} - \varphi_{i,i+1} \tag{6}$$

The number of vehicles that fails to pass intersection $i+1$ in time in the traffic flow is $t_{\mathrm{u}}' q_{\mathrm{u}}$, and they need to wait for the green light in the next cycle to pass. The time required for these vehicles to pass is t'.

$$t' = \frac{t_{\mathrm{u}}' q_{\mathrm{u}}}{q_{\mathrm{um}}} \tag{7}$$

By combining Eqs. (6) and (7), we have

$$d_{i+1,\mathrm{u}}' = q_{\mathrm{u}} t_{\mathrm{ir}} \left(t_{i,i+1} - \varphi_{i,i+1}\right) - \frac{1}{2} q_{\mathrm{u}} \left(t_{i,i+1} - \varphi_{i,i+1}\right)^2 + \frac{q_{\mathrm{u}}^2}{2q_{\mathrm{um}}} \left(t_{i,i+1} - \varphi_{i,i+1}\right)^2 \tag{8}$$

Let α_i be the Boolean function of intersection i in the upstream direction, n be the number of intersections.

$$D_{\mathrm{u}} = \sum_{i=2}^{n} \left[\alpha_i d_{\mathrm{iu}} + (1 - \alpha_i) d_{\mathrm{iu}}'\right] \tag{9}$$

Similarly, for the delay time in the upward direction, let d_{id} $\left(d_{\mathrm{id}}'\right)$ be the downstream delay time of the vehicle arriving at intersection i at the red (green) light. Let q_{d} be the actual traffic flow in the downstream direction; q_{dm} be the downstream saturation flow; $\varphi_{i+1,i}$ be the offset from intersection $i+1$ to intersection i; β_i be the Boolean function of intersection i in the downstream direction.

$$D_{\mathrm{d}} = \sum_{i=1}^{n}\Big[\beta_i d_{\mathrm{id}} + (1-\beta_i)d_{\mathrm{id}}^{'}\Big] \tag{10}$$

$$d_{\mathrm{id}} = \frac{q_{\mathrm{d}} q_{\mathrm{dm}}\left(C - \varphi_{i+1,i} - t_{i+1,i}\right)^2}{2(q_{\mathrm{dm}} - q_{\mathrm{d}})} \tag{11}$$

$$\begin{aligned} d_{\mathrm{id}}^{'} &= q_{\mathrm{d}} t_{\mathrm{r}}\left(t_{i+1,i} - C + \varphi_{i+1,i}\right) - \frac{1}{2} q_{\mathrm{d}}\left(t_{i+1,i} - C + \varphi_{i+1,i}\right)^2 \\ &+ \frac{1}{2} q_{\mathrm{d}}^2 \frac{\left(t_{i+1,i} - C + \varphi_{i+1,i}\right)^2}{q_{\mathrm{dm}}} \end{aligned} \tag{12}$$

For the queue length l, let q_{m} be the saturation flow; x be the saturation; N be the number of vehicles arriving at the intersection in a cycle.

$$l = \frac{\exp\left(-\frac{4}{3}\sqrt{(\mathrm{C} - t_{\mathrm{r}}) q_{\mathrm{m}} \frac{1-x}{x}}\right)}{2(1-x)} + N\left(1 - \frac{\mathrm{C} - t_{\mathrm{r}}}{C}\right) \tag{13}$$

The average stop times H are expressed as the total number of stops divided by the number of vehicles arriving in a cycle, which is denoted by S and N, respectively. Let q be the actual traffic flow in all inlet; q_{r} be the maximum waiting traffic flow in red.

$$H = \frac{S}{N} = \frac{q\left(\frac{q_{\mathrm{r}}}{q_{\mathrm{m}} - q} + r\right)}{N} \tag{14}$$

To ensure that the results are reasonable, we must set boundaries for the decision variables. Let $t_{i,l}$ be the green loss time; $t_{i,b}\left(t_{j,b}\right)$ be the yellow time.

$$s.t.\begin{cases} t_{i,\mathrm{r,min}} \le t_{i,\mathrm{r}} \le t_{i,r,\mathrm{max}} \\ C - t_{i,\mathrm{r}} - t_{i,l} \ge \lambda_{i,\mathrm{r,min}} C \\ C_{\min} \le C \le C_{\max} \\ t_{i,\mathrm{r}} + t_{j,\mathrm{r}} + t_{i,b} + t_{j,b} = C \end{cases} \tag{15}$$

Combining Eqs. (1), (5), (8), (9)–(15), we obtain the first model M1. For the final optimization in Eq. (1), decision variables include the cycle (C), red light time $\left(t_{i,r}, t_{j,r}\right)$, the green split ($\lambda_i$) and offset $\left(\varphi_{i,i+1}\right)$. Model parameters include actual traffic flow (q), saturation flow (q_{m}) and travel time.

3 Incorporate Traffic Dispersion

In the model M1, the calculation method of link travel time from intersection i to intersection $i + 1$ is the ratio of the distance in between to the free flow speed of that link, as in Eq. (2). In most existing models, the free flow speed is set to a fixed value. However, in reality, the free flow speed is rarely achieved, especially when the traffic is far from being sparse. And, assuming the link travel time for all traffic that remains constant is far from being realistic. Some other models improve on this by setting the speed boundary conditions and change functions, but this introduces new variables into the model, resulting in an increase in model complexity [11].

Based on the above observations, we turn to the widely used signal timing method TRANSYT series of traffic dispersion module to estimate link travel time. The dispersion module ensembles computing the expectation of geometric variables to estimate the link travel time. The travel time can be adjusted in accordance with the upper and lower travel speeds on that link. For the upper travel speed, the free flow speed or the speed limit of that link is used. As for the lower travel speed, since there usually is no lower speed limit for urban traffic systems, the bound is set based on the simulation results from VISSIM.

Now, let t_l $\left(t_l'\right)$ be travel time from the intersection i $(i + 1)$ to intersection $i + 1$ (i) required by the vehicle with the lowest speed on line. Let t_{f} $\left(t_{\mathrm{f}}'\right)$ be the travel time from intersection i $(i + 1)$ to intersection $i + 1$ (i) required by the vehicle with the highest speed on line. Therefore, travel time in the upward $\left(t_{i,i+1}\right)$ and downward $\left(t_{i+1,i}\right)$ directions can be calculated using the following equations.

$$t_{i+1,i} = \sum_{t=t_l'}^{t_{\mathrm{f}}'} \frac{F(1-F)^{t-t_l'}}{C} t \tag{16}$$

$$t_{i+1,i} = \sum_{t=t_l'}^{t_{\mathrm{f}}'} \frac{F(1-F)^{t-t_l'}}{C} t \tag{17}$$

$F = 1/(1 + \kappa t)$, and κ is an adjusting factor and is set to 0.35 [12]. As a result, the second proposed model M2 is given by replacing Eq. (2) in M1 with Eqs. (16) and (17). By comparing M1 and M2 models and substituting Eq. (16) into Eq. (2), we can solve that, when all traffic flows are assumed to travel at the speed $V = v/C - \kappa L$, model M1 is equivalent to model M2. In this case, link travel time is $t_l = t_f = LC/(v - \kappa LC)$. Mathematically, for the above to be true, the boundary condition for κ is

$$0 < \kappa < \frac{v}{lC} \tag{18}$$

As a result, the dispersion module can be reduced to the free flow model by the above transformation. Equation (2) can be considered a special case of Eq. (16), which means that model M2 has a larger solution space than model M1.

4 Simulation Study and Result Discussion

For the above proposed model, we selected three important intersections of one urban arterial road, to collect traffic data and conduct simulation experiments. Four paths with significantly larger traffic flow were selected to observe the change of delay time. It shows, in Fig. 3, the geographical topology for the test system.

During the three time periods, morning peak (7:00–9:00), off-peak (15:00–17:00) and evening peak (17:00–19:00), on-site surveys were conducted to collect traffic data. Traffic volume data suggest four critical lines that contain most of the traffic over the planning horizon. We marked these three intersections from right to left as 1, 2 and 3. Line 1 contains the traffic flow from the northbound off-ramp through the major arterial path; line 2 passes through the major arterial path in opposite directions; line 3 contains the traffic flow from east to west at intersection 3; line 4 enters the major arterial path from the east entrance of intersection 2. It is worth noting that the existing phase design is quite reasonable and does not need to be changed. It shows, in Table 1, the phase design of the current timing scheme.

For numerical simulation, the multi-objective evolutionary algorithm NSGA-II was adopted to obtain the optimal solutions. The NSGA-II is one of the most advanced multi-objective optimization algorithms based on Pareto optimal solution currently.

For performance evaluations, we adopted VISSIM to simulate and record the total delay time of each path output, in unit of minutes. Then, we compared the path performance of the three timing schemes in three different time periods. It shows, in Table 2, the current scheme and the phase lengths resulting from M1 and M2.

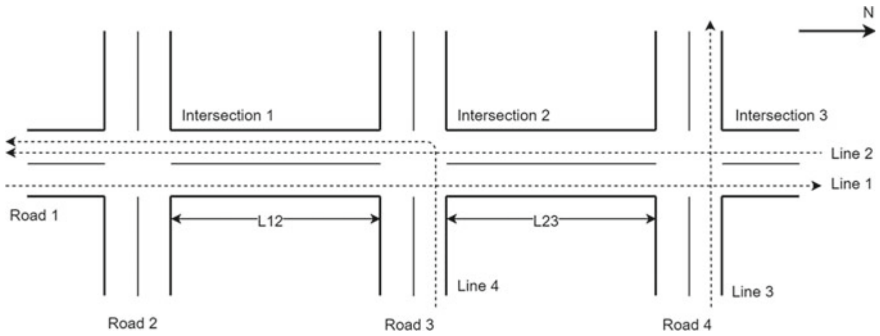


Fig. 3 Geographical topology for the test arterial line

Table 1 Design of phases of a cycle for the current timing scheme

Current scheme	Intersection 1	Intersection 2	Intersection 3
Phase 1			
Phase 2			
Phase 3			
Phase 4			

Table 2 Simulation results for three timing schemes

Scheme	Current $C = 150$	M1 $C = 160$	M2 $C = 140$
Intersection 1	(61, 46, 43)	(63, 51, 46)	(56, 48, 36)
$\varphi_{1,2}$	27	34	31
Intersection 2	(45, 46, 59)	(42, 55, 63)	(39, 48, 53)
$\varphi_{2,3}$	55	67	64
Intersection 3	(50, 28, 32, 40)	(47, 30, 36, 47)	(41, 28, 31, 40)

*Note that the phase lengths are arranged in the phase sequence shown in Table 1

It shows, in Fig. 4, the comparison of total delay time of three timing schemes. Obviously, the timing scheme obtained by M1 and M2 is better than current scheme, with an improvement of 21–24% in road performance.

It can be observed from Fig. 4, that even M2 has no obvious advantage over M1 in the branch (according to lines 3 and 4), from the perspective of the arterial road, M2 is a better model than M1 (according to Lines 1 and 2). That is to say, incorporation of the dispersion constraints does improve road performance, compared to free flow settings in other models. In fact, this is to be expected because the free flow speed is difficult to achieve, and using a constant travel time is an unrealistic ideal.

As can be seen from Fig. 4, the delays of line 1 in the morning peak and line 2 in the evening peak are significantly higher. This is because the traffic flow from south to north is relatively high in the morning and opposite in the afternoon. It can be seen from lines 1 and 2 that the performance of M2 is particularly improved in the

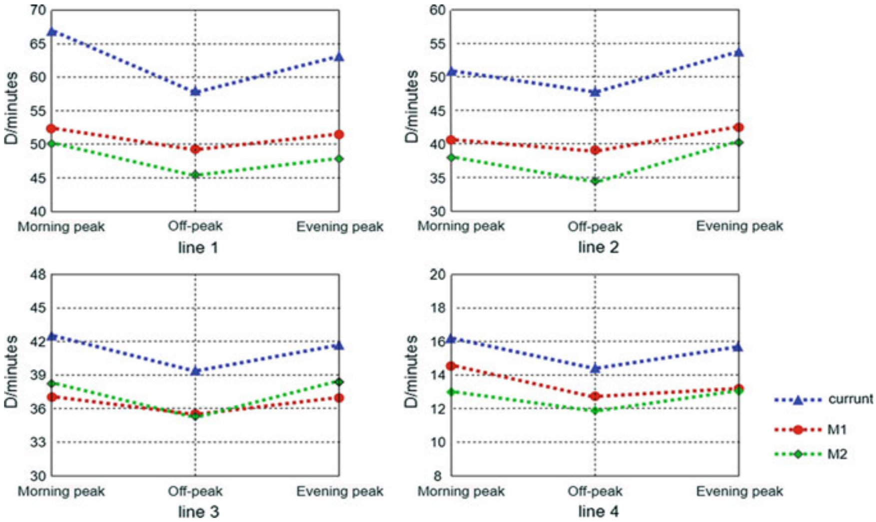


Fig. 4 Comparison of the control delay per line

off-peak, because the vehicle speed difference is larger in this period and the traffic dispersion module is more applicable. In line 3, current timing scheme adopts a fixed offset, which leads to the rise of delay, while M1 changes this situation. In line 4, subject to the left-turn phase time not changing, the delay time is not significantly improved, and the slight reduction is due to the improvement of the traffic capacity between intersection 1 and intersection 2.

5 Conclusion

In this paper, the traffic signal timing coordination on arterial road is considered as a multi-objective optimization problem, and models M1 and M2 can be used to generate signal timing schemes based on traffic flow. As a result, numerical simulation and result analysis have shown that models M1 and M2 are able to improve the performance compared with the current schemes. In addition, it can also be found that M2 is the best choice for optimizing the delay time of the main line, while M2 has no overall advantage over M1 in the lines that include branch traffic flows. Finally, the most potential development direction of traffic signal timing is to increase the scale of online control. With the rapid development of 5G communication and intelligent vehicles, the arterial systems or regional traffic signal systems can automatically change according to real-time traffic, which is the development direction in the future.

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