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Multi-Objective Optimization of Intersection Signal Time Based on Genetic Algorithm

by

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Abstract

A signal control intersection increases not only vehicle delay, but also capacity reduction in that area. Because more and more fuel and air pollution problems arise recently, an intersection signal control optimization method which aims at reducing vehicle delay, improving capacity is required heavily. This paper proposed a signal control multi-object optimization method to reduce vehicle delay and improve intersection capacity simultaneously at an intersection by using the Genetic Algorithm (GA). Data regarding traffic stream parameters, signal timing details and delay to vehicles are collected from an intersection in Shanghai, Hu Cheng Huan road. The result of the case study shows the optimal timing scheme obtained from this method is better than the observed one.

Keywords: Vehicle delay, Signal capacity, Signal control, Genetic algorithm, Multi-objective

1. Introduction

A favorable signal time scheme can increase the traffic capacity of entering or leaving arterial roads from minor road, and eliminate bottlenecks at connections. To find an optimal cycle and appropriate duration for green time in each phase, researchers often aimed at minimizing the delay or the queue length.

Traditional signal timing method is the Webster model, which is based on minimizing traffic delay to calculate the timing plan (Dion and Hellinga 2002). With the growth of automobile, the Webster model cannot satisfy actual situations. So, lots of researchers developed various signal timing methods to meet the demands (Ceylan and Bell 2004; Chen et al. 1997; Yang et al. 2001). Liu et.al (2005) made a classification of signalize intersection approach according to its traffic condition, namely unsaturation, critical saturation and oversaturation, and analyzed the delay at each class of approach; Ban et.al (2011) estimated real time queue lengths at signalized intersections using travel times.

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Many of the past research effort were conducted to examine various signal timing optimization methods with different single objective. For instance, Saka et al. (1986) investigated two innovative stochastic traffic signal optimization techniques on isolated intersections. The optimum cycle and green-phase lengths were determined by minimizing the average delay at the intersection within a given period of observation. Foy and Benekohal et al. (1992) implemented a GA to generate optimal or near-optimal intersection traffic signal timing strategies which yield the smoothest traffic flow with the least average automobile delay. Park et al. (1999) developed a GA-based signal optimizer and a mesoscopic traffic simulator to handle oversaturated signalized intersections. All of these signal optimization research used only one objective function, but provided a basis for investigating the implementation of multi-objective optimization technologies in traffic signal timing design.

Although single-objective optimization methods prevail in signal timing design, in most real-world problems, several goals must be satisfied simultaneously in order to obtain the preferred solution. A common difficult with the multi-objective optimization problem is the appearance of an objective conflict: none of the feasible solutions allow simultaneous optimality for all objectives. Signal timing planning is a typical multi-objective optimization problem, because for a signalized system, an optimal timing plan is usually required to meet some typical objectives (Leonard, 1998), such as:

- Minimizing delay;
- Minimizing stops;
- Minimizing fuel consumption;
- Maximizing progression.

A useful method which could balance different objective function should be proposed. Thus, a mathematically most favorable Pareto-optimum is the solution that offers the least objective conflict. Therefore, multi-objective problems are addressed to provide several Pareto optimal solutions, while decision makers are concerned with the selection of the most suitable solution from them.

Since GA search for the optimal solutions based on a population of points instead of a single point, multiple Pareto-optimal solutions can be found in a single run. A number of GA-based multi-objective optimization tools have been developed in recent years, including multi-objective optimization GA—MOGA (Shaffer, 1985), Niched Pareto GA—NPGA and non-dominated sorting GA—NSGA (Srinivas et.al, 1994), Strength Pareto Evolutionary Algorithm—SPEA (Zitzer et.al, 2001), Pareto-Archived Evolutionary Strategy—PAES (Knowles et.al, 1999), and Non-dominated Sorting GA II—NSGAII (Deb et.al, 2002) etc. All of these methods can be divided into two categories. The first category just converts a simple GA to a multi-objective GA by adding some new operators, such as MOGA, NPGA and NSGA. Nevertheless, these methods have been criticized due to their high computational complexity, non-elitist approach and their needs for setting an arbitrary sharing parameter. This results in the development of some new elitist MOEAs, including PAES, SPEA and NSGA II. In some recent studies, NSGAII has been proved to be one of the very promising members of MOEAs.

In this paper, a GA based intersection signal control multi-objective optimization is proposed by using NSGAII tool for heterogeneous traffic condition. This method can reduce vehicle delay and improve capacity simultaneously at an intersection.

This paper is organized as follows. Firstly an overview of some concerned research on intersection time design and multi-objective optimization is outlined. Secondly, the intersection used for case study is introduced. Thirdly, two problems of objective signal timing optimization with 7-constraint, which cover motor traffic types using Webster delay formulation, degree of saturation, and traffic capacity calculation equation, are designed and solved by NSGAII. The results including GA optimization and the observed result are discussed including regression function for Pareto-optimal solution set. Finally, conclusions of this study are presented.

2. Case Study

The study case is near Shanghai Ocean University, which is a T type signal intersection with motor and non-motor vehicles. The reason for choosing it is that the composition of traffic is complex (pedestrians (most part students), vehicle, E-bicycle, bicycle, truck and bus) and two bus stops are very close to the intersection, which led very bad traffic snarl and potential safety hazard. The investigation results include environmental conditions (obtained data by measuring road width, the number of lanes, et al.), traffic flow (number of each type of vehicle), and traffic signal conditions data.

2.1 Traffic signal condition

Traffic signal of research is arranged into 3 stages/3phases, total cycle time is 100s, as shown in Fig.1. It is obtained by counting the signal time at the location of intersection. There have no right turn signal indicator lamps to give right turn indications in this intersection.

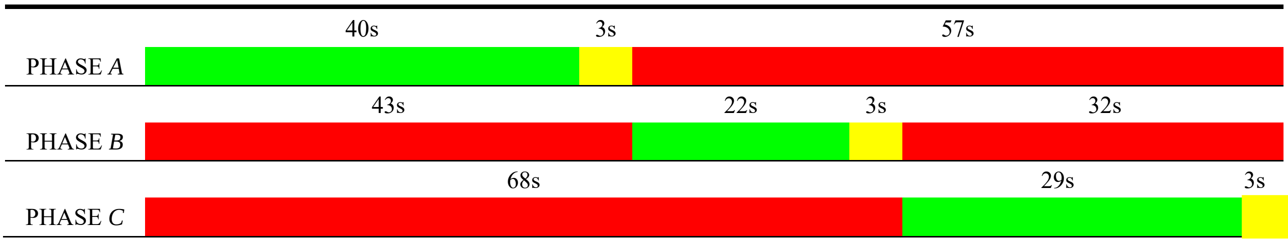

Fig. 1 Signal phase diagram

2.2 Geometric data

Results of the effective width measurement for each phase can be seen in Table 1 and Fig.2.

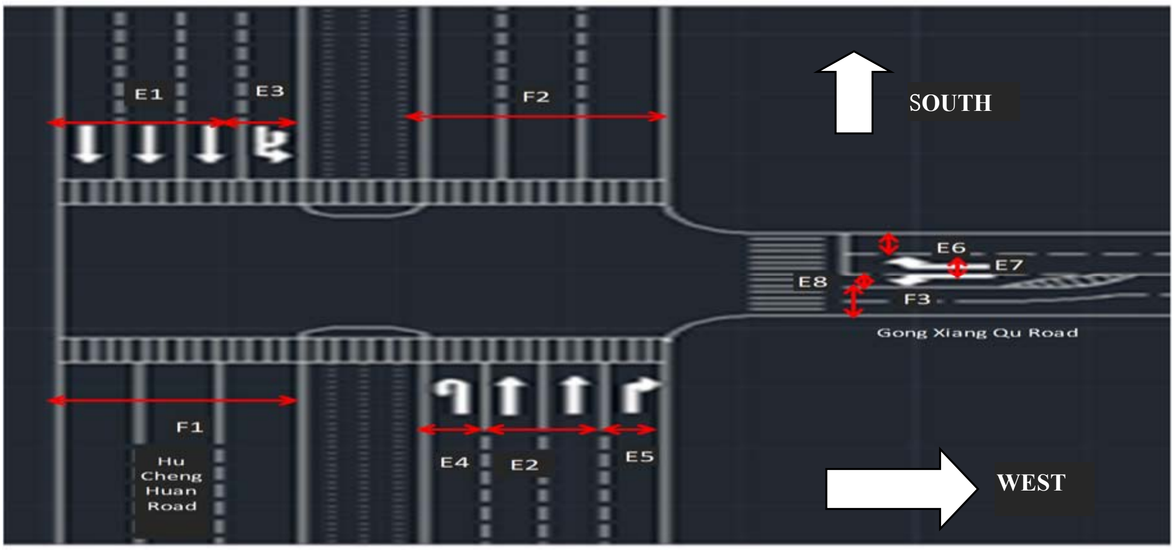

Fig. 2 Result of the effective width measurement

Table 1 Result of the effective width measurement

Phase	Entry			Exit		
	Approach	lanes	Width/lane (m)	Approach	lanes	Width/lane (m)
A	E_1	3	4.00	F1	3	4.00
	E_2	2	4.00	F2	3	4.50
B	E_3	1	3.75	F3	2	2.75
	E_4	1	3.75	—	—	—
C	E_8	1	3.00	—	—	—
Right turn	E_5	1	4.00	—	—	—
	E_7	1	3.00	—	—	—
	E_6(right turn bus lane)	1	3.50	—	—	—

2.3 Road and environment condition data

On either side of Gong Xiang Qu Road (Fig.2) there are one university and one bus stop. At the west end, there are some public facilities, so generally the activities around the intersection can be classified as commercial area. Based on visual observation, this location can be classified as an area on flat condition with 0% gradient.

2.4 Traffic flow condition

All data of traffic flow is presented in Table 2. This table shows the movement of each vehicle per phase at each approach. In this research we didn’t consider pedestrian and bicycle. Therefore, there are 4 categories of vehicle: bus, truck, ordinary car, and E-bicycle.

Table 2 Traffic flow recapitulation (Thursday, March 15th, 2017, PM4: 30-5:30, veh/h)

Phase	*A*		*B*		*C*	Right turn		
Vehicle type	E_1	E_2	E_3	E_4	E_8	E_5	E_7	E_6
Bus	12	16	12	0	20	30	0	24
Truck	80	38	9	0	27	46	48	0
Private Car	456	422	200	0	259	226	302	0
E-bicycle	29	49	52	0	66	57	86	0

In heterogeneous traffic condition, the traffic flow is accounted by converting each category of vehicle into equivalent passenger car units (PCU). Thus saturation flow rate is expressed in equivalent PCU per hour to accommodate for any possible mix of vehicles. In this study PCU values specified in Table 3 was adopted to convert each category of vehicles into its equivalent PCU values.

Table 3 Values of PCU

Vehicle type	Values
Private car (including taxis)	1
E-bicycle	0.5
Bus	1.5
Truck	1.5

After converting, the total investigated traffic flow according to different phase is denoted in

Table 4.

Table 4 Traffic flow after converting (pcu/h)

Phase	*A*			*B*	*C*	Right turn		
Vehicle type	E_1	E_2	E_3	E_4	E_8	E_5	E_7	E_6
Bus	18	24	18	0	30	45	0	36
Truck	120	57	14	0	40	69	72	0
Ordinary Car	456	422	200	0	259	226	302	0
E-bicycle	15	25	26	0	33	29	43	0
Sum	609	528	258	0	362	369	417	36

3. Methodologies

3.1 Estimation of saturation flow rate and degree of saturation

Saturation flow rate is an important factor in the estimation. It is a measure of the maximum rate of flow of traffic which could be obtained if hundred percentage green times was awarded to a particular approach.

In this research, saturation flow rate was obtained by qualified method based upon method recommended in 2000 Highway Capacity Manual (HCM2000) which is a publication of the Transportation Research Board of the National Academies of Science in the United States. The result of saturation flow rate and the adjustment factors are denoted in Table 5.

In Table 5, S_0: the basic saturation flow rate; f_w: lane width adjustment factor; f_{LT} : left-turn adjustment factor; f_{RT}: right turn adjustment factor; N: number of lanes. Therefore, adjusted saturation flow rate $S(\text{pcu/h})$ is denoted in Eq. 1.

$$S(\text{pcu/h}) = S_0 \times N \times f_w \times f_{RT} \times f_{LT} \quad (1)$$

Cycle time that will be used in capacity calculation is obtained by counting the average cycle time setting plan data that is 100 seconds. Calculation results of degree of saturation rate (DS) are in Table 6.

Based on the calculation result, it shows that the maximum DS is 0.16, 0.17 and 0.27 respectively, in phase *A*, *B* and *C*. The sum of maximum DSs in three phases is 0.6.

Table 5 Saturation flow rate S

Phase	Approach	S_0	Adjustment factor			S
		pcu/lane/h	f_w	f_{RT}	f_{LT}	pcu/h
A	E_1	1650	1.025	1	1	5073
	E_2	1650	1.025	1	1	3382
B	E_3	1550	1.0125	1	0.95	1490
	E_4	1550	1.0125	1	0.95	1490
C	E_8	1550	1.0	1	0.87	1349
Right turn	E_5	1550	1.025	0.85	1	1350
	E_7	1550	1.0	0.85	1	1317
	E_6 (right turn bus lane)	1550	1.0	0.85	1	1317

Table 6 Calculation result of degree of saturation (DS)

Phase	Approach	Q (pcu/h)	S (pcu/h)	DS
A	E_1	609	5073	609/5073=0.12
	E_2	528	3382	528/3382=0.16
B	E_3	258	1490	258/1490=0.17
	E_4	0	1490	0
C	E_8	362	1349	362/1349=0.27

3.2 Delay

Delay is an important parameter that is used in the performance evaluation of signalized intersections. Delay is influenced by many variables and hence its determination is a complex task. Webster's classical delay formula is the oldest and the most popular one among the models developed to estimate average delay per vehicle at signalized intersections for strict lane disciplined traffic.

Webster (1958) using the deterministic queuing analysis developed a model for the estimation of delay incurred by the vehicles at under saturated or saturated signalized intersections. The mathematical form of the Webster's delay model is shown in Eq. 2.

$$d = \frac{(Ct-g)^2}{2Ct(1-\lambda \times DS)} + \frac{DS^2}{2q(1-DS)} - 0.65(\frac{Ct}{q^2})^{\frac{1}{3}} DS^{(2+5\lambda)} \tag{2}$$

where, d is the average delay per vehicle on the approach (s/veh); g is the effective green time (s); Ct is the cycle time (s); q is the flow rate (veh/s); DS is the degree of saturation; $\lambda = g/Ct$ is the proportion of the cycle which is effectively green for the phase under consideration.

The first term represents the average delay to vehicles assuming uniform arrivals. The second term represents the additional delay due to the randomness of vehicle arrivals. This additional delay is attributed to the probability that sudden surges in vehicle arrival may cause the temporary over saturation of the signal operation. The third term is a semi-empirical correction term that was introduced in the model to account for specific field conditions (Preethi P et al., 2016). In this study, we just consider the first and the second terms when calculation the vehicle delay.

The equation of effective green time g_i for phase i in this research is denoted in Eq. 3.

$$g_i = G_i + A_i - L \tag{3}$$

where, g_i (s) is the effective green time for phase i; G_i (s) is the real green time for phase i; A_i is the yellow time for phase i; L =4s is the loss time which is the sum of start loss time=2s and clearance loss time=2s .

The minimization of average delay for each vehicle crossing the signal is used as the first objective equation, which is shown in Eq.4.

$$\text{Min}(\frac{\sum_{i=1}^{n}(d_i Flow_i)}{\sum_{i=1}^{n} Flow_i}) \tag{4}$$

where, d_i (s) is average delay per vehicle in phase i; $Flow_i$ (pcu) is one hour traffic flow for phase i; n is the phase number.

3.3 Traffic capacity at intersection

Traffic capacity in effective green time is calculated with Eq. 5.

$$CAP_A = \frac{(5073+3382)g_1}{Ct};$$

$$CAP_B = \frac{1490g_2}{Ct}; \quad (5)$$

$$CAP_C = \frac{1349g_3}{Ct};$$

where, CAP_A (pcu/h): traffic capacity in phase *A* (pcu/h); CAP_B: traffic capacity in phase *B* ; CAP_C (pcu/h): traffic capacity in phase *C*; g_1(s): effective green time for phase *A*; g_2 (s): effective green time for phase *B*; g_3 (s): effective green time for phase *C*; Ct (s) : total cycle time.

3.4 Multi-objective GA optimization

GA is a stochastic search algorithm based on theory of evolution and population genetics (Xi et al. 1996). The elements of GA contain encoding, fitness function and operators. In this paper, they are set as follows.

Each individual in GA stands for a timing scheme of the signalized intersection, as G_i={G_1, G_2, G_3}, in which G_i represents real green time of phase i in the scheme and it is the variable remaining to be optimized.

Typical GA follows a sequence of decisions that can be summarized as follows:

Step 1: Problem encoding;

Step 2: Random generation of initial population;

Step 3: Evaluation of the fitness of each chromosome in the population;

Step 4: Selection for reproduction;

Step 5: Crossover and mutation;

Step 6: Test for stopping criteria. Return the solution, if satisfied; repeat from Step 3 onward, if not satisfied.

This work focused on capturing a cycle length design and green time split which take into account the minimization of average vehicle delay and maximum intersection traffic capacity using NSGAII (Non-dominated Sorting GA).

The average delay per unit vehicle and the traffic capacity are considered vital in the evaluation of the traffic signal timing plan. A generic 2-objective traffic signal timing optimization problem for a three—phase control strategy can be formulated as Eq.6 :

Object 1: (delay)

Min $f_1(G_i)$=

$$\Bigg[(609+528)\left(\frac{(G_2+G_3+10)^2}{2(G_1+G_2+G_3+9)\left(1-0.16\frac{(G_1-1)}{(G_1+G_2+G_3+9)}\right)}+\frac{0.16^2}{\frac{(609+528)}{3600}(1-0.16)}\right)$$

$$+258\left(\frac{(G_1+G_3+10)^2}{(G_1+G_2+G_3+9)\left(1-0.17\frac{(G_2-1)}{(G_1+G_2+G_3+9)}\right)}+\frac{0.17^2}{\frac{(258)}{3600}(1-0.17)}\right)$$

$$+362\left(\frac{(G_1+G_2+10)^2}{(G_1+G_2+G_3+9)\left(1-0.27\frac{(G_3-1)}{(G_1+G_2+G_3+9)}\right)}+\frac{0.27^2}{\frac{(362)}{3600}(1-0.27)}\right)\Bigg]/(609+528+258+362)$$

Object 2: (traffic capacity)

Min $f_2(G_i)$= $\frac{-(5073+3382)(G_1-1)}{G_1+G_2+G_3+9} - \frac{1490(G_2-1)}{G_1+G_2+G_3+9} - \frac{1349(G_3-1)}{G_1+G_2+G_3+9}$

Subject to

$$30\text{s} \le G_1 + G_2 + G_3 \le 120\text{s};$$
$$30\text{s} \le G_1 \le 80\text{s};$$
$$15\text{s} \le G_2 \le 80\text{s};$$
$$20\text{s} \le G_3 \le 80\text{s};$$
$$\frac{(609+528)}{(5073+3382)\frac{G_1}{(G_1+G_2+G_3+9)}} < 0.9;$$
$$\frac{258}{1490\frac{G_2}{(G_1+G_2+G_3+9)}} < 0.9;$$
$$\frac{362}{1349\frac{G_3}{(G_1+G_2+G_3+9)}} < 0.9;$$

(6)

where, G_1: real green time of phase *A;* G_2: real green time for phase *B*; G_3: real green time for phase *C*. .

To optimize the traffic signal time, signal design problems are defined to minimize average delay per vehicle and maximize intersection capacity, using the real green time at each signal phase as the design variable. Such objective consideration is conflicting in traffic signal design, because minimizing delay leads to short cycle length while maximize traffic capacity leads to longer cycle time. These objectives are also non-commensurable. The average traffic capacity usually has larger value, while the average delay is generally a smaller one. The designed scenario is a three-phase isolated intersection with permissive right turn. The flow ratios are 0.16, 0.17 and 0.27. Table 7 shows the GA parameters used in these experiments as follows:

Table 7 GA parameters used in signal optimization

Parameter	Value	Parameter	Value
Population size	60	Lower limits on 1st variable	30
Selection strategy	Tournament	Upper limits on 1st variable	80
Probability Cross-over	0.75	Lower limits on 2nd variable	15
Probability mutation	0.02	Upper limits on 2nd variable	80
No. of Functions	2	Lower limits on 3rd variable	20
No. of Constrains	7	Upper limits on 3rd variable	80
No. of variables	3		

3.5 Optimization result

The curve in Fig. 3 shows the non-dominated boundary Pareto-optimal solutions in first front F_1 (the state of Pareto Optimality) obtained using NSGA-II, after 800 times iterations. The regression equation of first objective value and second objective value is denoted in Eq. 7. The regress value reached 0.998.

We choose one result from the Pareto-optimal solutions (30, -4017) as the final solution, with the accordance integer result G_1=36s, G_2=17s and G_3=26s. The results are shown in Table.8.

In order to analyze the effectiveness of the multi-objective optimization, a comparison between the proposed method and observed signal time was carried out. Then two kinds of objective cost of the observed scheme were calculated by the evaluation model. Their values are also shown in Table 8.

The comparison result shows that the delay of the optimal scheme (30s) is lower than that of observed scheme (34s). And the traffic capacity of the optimal scheme (4017 pcu/h) is higher than that of observed scheme (3988 pcu/h). It indicates that the proposed optimization method has an effect on reducing vehicle delay and improving traffic capacity of this signal intersection simultaneously.

Table 8 Optimization result and observed result.

	GA	Observed
Phase *A*	36s	40s
Phase *B*	17s	22s
Phase *C*	26s	29s
Delay	30s	34s
Capacity	4017 pcu/h	3988 pcu/h
Cycle	88s	100s

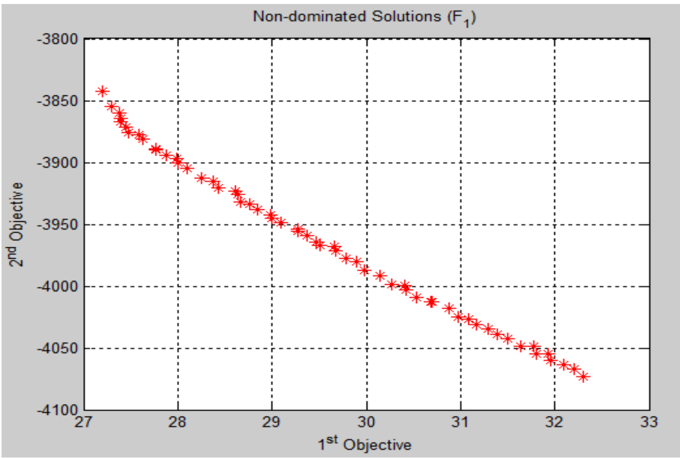

Fig. 3 Non-dominated solutions obtained using NSGAII.

$$y = 0.51x^3 + 47.43x^2 - 1511.68x + 12406.29, \qquad R^2=0.998, \qquad (7)$$

where, x is object 1's value; y is object 2's value, R^2 is regression value.

4. Conclusion

This paper proposed a GA based intersection signal control multi-objective optimization method. It aims at finding an optimal timing scheme that generates less vehicle delay and more traffic capacity. The optimization was applied to an intersection in Shanghai and obtained an optimal scheme. Because this method minimized vehicle delay and maximized the traffic capacity, this result indicates that the GA based multi-objective optimization is effective for this case.

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