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Development of dynamic traffic signal control based on Monte Carlo simulation approach

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ABSTRACT

This study addresses the maximum queue length problem arises when the arriving traffic exceeds the sensors' detection area, especially for the over-saturated condition. The paper presents an adaptive system working Monte Carlo based Signal Timing (MCaST) algorithm based on microscopic scale vehicle arrival. Since the algorithm works as an adaptive system, two new mathematical formulas for the delay and queue length calculations have been proposed instead for fixed time formulas. Based on both queue length and intersection delay, the intersection performance has been calculated and used as a slave method to optimize the cycle length. The suggested MCaST algorithm has been tested using real field data, and the delay is decreased by 30.2% less during peak hour traffic. Besides, the delay results from the algorithm have been compared with Webster's and Highway Capacity Manual 2010 delay formulas and VISSIM software. Results of the numerical experiments show the MCaST algorithm's validity.

1. Introduction

Traffic signal control is the most used traffic control application on intersections because the orders of the links are illustrated. However, traffic signal control of the intersection causes delays while ensuring intersection safety. Therefore, there are many studies about optimizing the signal timings [21;12,18,23] for more fluent traffic flow and less delay. The problem with traffic signal optimization is that there isn't just one solution. Since the flow is not uniform, vehicle speed varies over a wide range, and the human factor always plays an important role. There are tremendous combination probabilities for setting the cycle length ($C_{min} \leq C \leq C_{max}$) and green times ($g_{min} \leq g \leq C$) for each phase. The geometry of the road section affects the solution, and the formed queues make it harder when not discharged. The researchers should pick the best one among many others, which minimize the desired parameter like delay, fuel consumption, emission, etc. However, it is difficult because of the non-convex structure of the solution set [1]. One of the most important parameters that make it difficult to find the best solution for the intersection design is queue [7]. The reason queues are important could be explained via a high relationship between traffic delays [8,14,29], the number of stopping vehicles [14,29] and travel time [24,16,3]. An et al. [4] argued that traffic engineers could improve the system and give the right decision by using the queue length. The authors agree with An et al. [4].

Predicting the flow distributions precisely and estimating the queue lengths that occurred on the link depending on time could improve the system. Based on the two parameters, the needed green time to discharge the queue, delay of each vehicle on a microscopic scale, intersection performance based on extra fuel consumption, optimum cycle length of the intersection, and offset timings could be calculated. Therefore, the queue length and vehicle arrival times are the key parameters. Here, the queue length is defined as the number of fully stopped vehicles because of the red traffic light within the examined period. We note that queue length was studied broadly in the past. Most of the queue length is obtained based on two methods, such as cumulative input-output [38,31,37] and shockwave approach [22;30,13,11].

Cumulative input-output traffic flow could be calculated and the number of accumulated vehicles in the queue could be obtained between the desired period of t_0 - t_n when the functions of the input and output traffic flow are known by subtracting the departing vehicles from arriving vehicles. Although the calculation is that easy, the distribution functions of the arrival and departure flow are hard to determine precisely. Even if the function is obtained, the residue queue length should be known. There are equations developed to calculate the queue length

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like Miller's [26,27]. However, equations are assuming the initial queue as zero [10]. Later, the Transportation Research Board (TRB) suggested [34] a new queue length formula and afterward, revised it [35]. Like the "no initial queue" assumption made by Miller [26,27], equations which are given in the Highway Capacity Manual [34,34], are also neglecting the initial queue length. Cai et al [10], argued that the manual observation method is the only method that can determine the initial queue precisely.

The input and output approach is a deterministic method to calculate the average queue length for a period, for instance, one hour. However, traffic flow is more stochastic than deterministic. Also, the initial queue length should be known to determine the queue lengths precisely. The shockwave approach is widely used to obtain queue lengths at the intersection. Stephanopolos et al. [32] defined the shockwaves in their study as the movement or propagation of a sudden change in intensity. The shockwaves formed in traffic are defined as analytical solutions of the Lighthill-Whitham-Richards [22,30] traffic model by the characteristic method of partial differential equations (PDE). In simple terms, a shockwave is generated by a change in the characteristic curves of the flow. If both definitions, made by Stephanopolos et al. [32] and the change in the characteristic curves, are considered, it is expected that shockwaves occur many times at the signalized intersections. For example, when the traffic light turns red while a flow passes the signalized intersection, the density at the intersection entrance would increase and the vehicles would accumulate to the downstream direction. This accumulation will continue to increase in the form of the shockwave. So, the queueing will occur with the speed of the shockwave.

In the light of this information about the shockwave, the maximum queue length could be achieved with the velocity of accumulation to the downstream direction and the velocity of the section of both shockwaves, shockwave of the upstream moving vehicles after the light turns green and the shockwave of the downstream accumulating vehicles before the queue discharges completely. However, to calculate the maximum queue length analytically, the arrival vehicle information should be fully available. If the queue length was between the stop line and the sensor reading area, the method would work perfectly. However, if the queue length exceeds the sensor reading area, the method would fail. Therefore, Liu et al [25], used the sensor occupancy times and the gap between the vehicles to estimate the discontinuities on the characteristic curves. However, when the flow goes to a saturated condition, it is impossible to determine the discontinuities on the characteristic curves because the flow will discharge with a saturated flow rate. Liu et al. [23] studied on distributed implementation of switch-based adaptive dynamic programming to solve the saturated condition of the traffic. The adaptive dynamic programming choose the time and the mode.

As a conclusion of both approaches, the cumulative input and output approach is simple, but the arrival and departure vehicle times should be known exactly. The shockwave approach is more complex but can be obtained using stochastic processes. In this study, vehicle arrival times are estimated based on Monte Carlo simulation as a stochastic process. Based on the estimation, queue lengths that occurred on the link over time have been calculated. The selection of the Monte Carlo simulation could be explained by obtaining the results quickly and estimating the vehicle's arrival times from an enormous data set. The arrivals are affected by the upstream. For example, how the upstream signal operates, how platoon dispersed, how the traffic turns onto or leaves from the corridor etc. All these can affect the arrival flow which makes the problem uncertain. So, the Monte Carlo approach is needed to handle these uncertainties. For this purpose, first, a theoretical model has been put forth. The main parts of the theoretical model are (i) obtaining the time intervals between the arrival vehicles, (ii) determining the statistical distribution of the intervals and obtaining the flow profile, (iii) obtaining the queue length with the maximum probability according to the result of Monte Carlo simulation. In the model's design, the first-in-first-out (FIFO) approach is accepted. After suggesting the theoretical model, it is tested using proper field data obtained from an adaptive working intersection in Denizli, a city of the Aegean region in the west of Turkey. In the end, the model is used for a four-link / four-phase signalized intersection to show efficiency, and a comparison has been made by comparing the delay results obtained from Webster's and HCM2010 delay formulas. Besides, the delay results are compared with the results of widely used simulation software. VISSIM by the PTV group has been selected because of its frequent use around the world for traffic simulation and planning. The model is created by using MATLAB R2018a software and all the tests have been done using a 64-bit Intel® Core™ i7-7700 CPU, 3.60 GHz, and 16 GB RAM Windows 10 Pro OS desktop.

2. Theory of the proposed Monte Carlo based signal timing

As introduced above, the proposed model has three principal parts which are getting the headways, fitting to a statistical distribution, and obtaining the queue length with the maximum occurrence rate. Before estimating the queue length, the flow profile should be known as precisely as possible. There are many statistical distributions used to define the flow profile. It is needed to find out the best fitting statistical distribution to define the flow.

2.1. Obtaining the distribution function of arrival flow

All the flow control systems are based on the arrival flow data. The arrival flow is important data to build a flawless working system. However, especially for over-saturated traffic flow where sensors are inadequate, it is almost impossible to know exactly what the arrival flow's properties are. When this happens, estimation is the only way to determine the arrival flow precisely. In this study, the estimation of the arrival flow has been made using statistical distributions. First, the vehicle information is gathered using sensors such as a loop detector.

The arrival flow pattern is determined by using the time intervals between the incoming vehicles. The time intervals are calculated by subtracting the time of the two vehicles' occupancy trigger time from the detector. Afterward, frequency analysis has been done to determine the distribution function. Therefore, the intervals in seconds are rounded up to the nearest multiple of 0.5. It would be more precise to round the time interval into much smaller multipliers. However, the number of samples should be more for proper frequency analysis.

Frequency values obtained from the analysis have been fitted to the statistical distributions; exponential (exp), lognormal (logn), gamma, generalized Pareto (gp), inverse Gaussian (ig), generalized extreme value (gev), Birnbaum-Saunders (BS) and log-logistic (logl) where each distribution function has its shape, scale and location parameters. The parameters of the distribution functions are determined using the maximum likelihood estimation method using the real field data obtained from the last analysis period, for instance, one hour for this study. The best-fitting distribution is selected using correlation analysis between the real field data and predicted data which reproduces the data better.

2.2. Monte Carlo approach

Statistical distribution is insufficient even after modeling the flow with a high level of significance because of the trend of the data obtained from the statistical distribution. The reason is that the outcome changes for the analysis period with the determined statistical distribution function and the same parameters with each calculation. Thus, an iterative approach would reveal the outcomes. This can be handled using the Monte Carlo approach. The major role of the Monte Carlo simulation for this study could be explained as follows. The Monte Carlo approach uses iterations. Iteration is a friendly approach to analyze the possibilities and their probabilities. There are some studies in the literature, such as the iterative learning approach used for traffic signal control by Huang et al. [19]. We used Monte Carlo to see the results of each iteration. At the end of the total iterations, we get a solution set with different total vehicle counts. With the help of the simulation, we could show the arrival flow based on the statistical data for the analysis period. However, the outcoming result of total vehicle counts for each simulation varies. To achieve an accurate result, the number of iterations has been increased. But as a result, a result set with different values was obtained. It is very difficult to know which of the total number of vehicles and arrival intervals based on the total vehicle number from the result set is expected at the end of the analysis period. Here, the simulation of Monte Carlo has its role, allowing creating the solution set and determining the total number of vehicles with the highest frequency value in the set as it is most probable and outputting it. Therefore, the Monte Carlo simulation determines the key parameters like estimating the total vehicle count and giving the most possible time intervals.

First, the statistical distribution function has been obtained as explained in Section 2.1. Afterward, when the traffic flow profile is obtained, probabilities for the arrival vehicles could be simulated using Monte Carlo for a period, for instance, one hour. To do that, the distribution values are used to obtain the cumulative distribution values. This the objective function for Monte Carlo simulation. Then, on each iteration, the simulation generates a random number $x \in 0,1$ based on uniform distribution. The randomly generated number a variable in the inverse transform for CDF. So, the corresponding headway times are obtained. The process repeats until the sum of headways is equal to or greater than the period, one hour for this study. One more than the repeating number of the process is the count of arrival vehicles. Each simulated vehicle's cumulative headway is the intersection arrival time. The simulation runs for a predetermined number of iterations, 2000 times for this study. After all, frequency analysis of the count of arriving vehicles for each iteration gives the queue lengths and their possibilities of occurrence. The flowchart of the proposed algorithm is given in Fig. 1.

2.3. Queue formation

Yang et al. [40], used mesoscopic simulation to estimate the queue length for different demand-to-capacity scenarios. So, simulation is an available method to estimate the queue lengths. In this paper, microscopic simulation is used for the estimation. The queuing is expected to occur because the dynamic traffic assignment by the Monte Carlo simulation is determined by two basic principles. The first one is the queue of all vehicles that reach the intersection in red time, and the second one is, the vehicles depart the intersection following the FIFO approach during the green time. When these two basic principles are taken into consideration, it is enough to make two calculations on the link where the queuing analysis is intended. The Monte Carlo algorithm created for the first calculation will be run for the red period and the number of vehicles generated will be recorded as the queue length. The algorithm will then be run again during the green time and the difference between the sum of the initial queue and estimated arrival flow, and the flow that can depart from the intersection in the green time will be recorded as the queue length. If the difference is positive, the queue could not be discharged during the green time and there is a queue left. If the difference is negative, it means that there is no queue. In the design of the signal timings, which will vary depending on the flow size, the cycle length should be taken into consideration and the queue calculations should be made by the phase plans. For a phase that has been

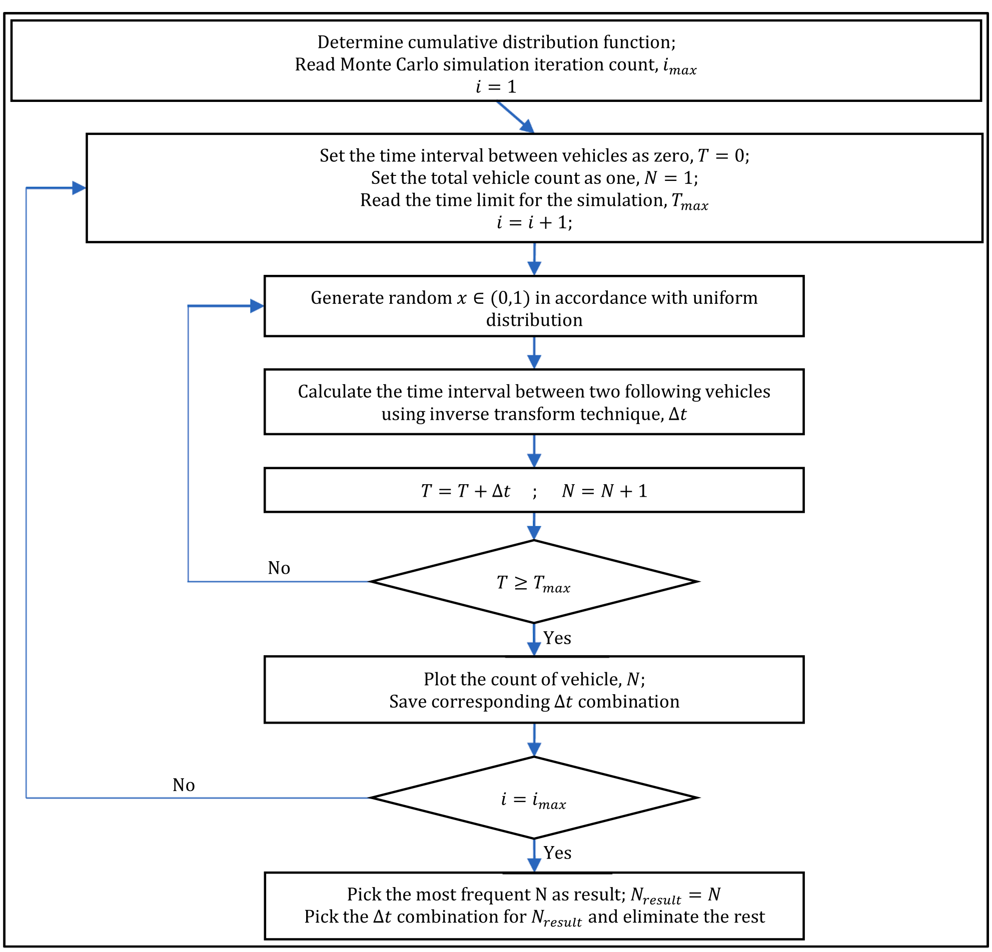

Fig. 1. Flowchart of Monte Carlo simulation part of proposed MCaST system proposed in this study.

formed with red, green, and red (typical four-phase four-link intersection), the queue calculation is obtained using Equation (1) [5].

1
$$Q_{total} = Q_{first} + V_{r_{end}} \tag{12}$$

2
$$Q_{first} = \begin{cases} 0 & if\left(Q_0 + V_{r_{first}} + V_g - s_g\right) \leq 0 \\ Q_0 + V_{r_{first}} + V_g - s_g & if\left(Q_0 + V_{r_{first}} + V_g - s_g\right). > 0 \end{cases} \tag{13}$$

3
$$s_g = t_g * \frac{s}{3600}, \tag{14}$$

where, Q_{total} is the total queue length at the end of the cycle (vehicle), Q_{first} is the initial queue (vehicle), Q_0 is the residue queue from the last cycle (vehicle), $V_{r_{first}}$ is the arrival flow at the first red time (vehicle), V_g is the arrival flow at the green time (vehicle), t_g is the green time (s), and s is the saturated flow rate (vehicle per hour). In a typical four-phase four-link intersection, there is a phase that starts with green. In such a situation, $V_{r_{first}}$ would be zero.

2.4. Vehicle delay

Delay is a very important parameter for traffic engineers. Although improving technology and more and more dynamic systems, delay modeling for adaptive working intersections has not been sufficiently studied. Some studies could be found in the literature, like modeling delay and emissions for signalized intersections by Zhu et al. [42]. We focus more on each vehicle individually to obtain the delay that occurred. Like queue formation, the delay calculation is based also on two principles; vehicles delayed according to their arrival time and departures at the green following FIFO. So, the time gaps (see Fig. 1, Δt) between arriving vehicles are the key parameters for the delay calculation. Here, the delay on both green and red times, should be calculated separately and then added cumulatively for the desired time interval. According to the two basic principles, the delay in red time is calculated using Equation (4) [5].

4
$$D_r = (Q^* t_r) + \left(\sum_{i=1}^{n}\left(t_r - \sum_{k=1}^{i}\Delta t_i\right)\right), \tag{15}$$

where, D_r is the delay occurred on red time (s), Q is the residue queue from previous green time (vehicle), t_r is the red duration (s), n is the arrival vehicle count during the red time, and Δt_i is the time gap between i^{th} and $(i-1)^{th}$ vehicles. For the first arriving vehicle after the light turns red, the Δt_i is the time gap between the vehicles' arrival time and redstart time. For the delay time that occurred on green time, Equation (5) could be used [5].

5
$$D_g = \begin{cases} \left(\sum_{i=1}^{k}\frac{i}{\frac{s}{3600}}\right) & if \;\; k = Q_0 + V_g \\ \left(\sum_{i=1}^{k}\frac{i}{\frac{s}{3600}}\right) + \left(\left(Q_0 + V_g - k\right)^* t_g\right) & if \;\; k = \frac{t_g{}^* s}{3600} \end{cases} \tag{16}$$

6
$$k = \min\left(\frac{t_g{}^* s}{3600}, Q_0 + V_g\right), \tag{17}$$

where, s is the saturated flow rate (vehicle per hour), Q_0 residue queue length (vehicle), V_g is the arrival flow on green time and t_g is the green time (s).

2.5. Intersection performance

Explanation of the flow could be done using the time metric since almost all the parameters depend on time such as the velocity of the vehicles, acceleration rate, flow rate, headways, time-based gaps, traffic signal timings, intersection clearance time, the arrival time of the flow. However, for a good design intersection, the time metric is not enough. There are some parameters, which create errors in time calculations such as a formed queue. Therefore, both time and queue must be taken into consideration to optimize the intersection. Two basic parameters could determine these parameters; delays as time and stopping count caused by an intersection as the queue. A joint examination of these two parameters will produce more accurate results, but because of the difference in units, a joint examination cannot be carried out with these states. For this reason, fuel consumption was examined to meet a common denominator in both parameters. When both intersection-induced delay and stopping counts are written on a fuel consumption basis, Equation (7), which measures the performance of the intersection, is obtained.

7
$$P_{intersection} = \alpha^* l_{departure}{}^* \sum_{i=1}^{n} Q + \beta^* l_{idle}{}^* \sum_{i=1}^{n} D, \tag{18}$$

where; $P_{intersection}$ is the intersection performance (l), $\sum Q$ is the total stopped vehicle count (vehicle), $\sum D$ is the total delay (vehicle-hour), $l_{departure}$ and l_{idle} are the consumption amount by accelerating after a full stop (l/vehicle) and while idling on the intersection (l/vehicle-hour), respectively. α and β are the weighting coefficients for vehicle types. The vehicle-based queue length has been taken into consideration because the entire vehicle at the queue makes a full stop. The fuel consumption value for an idle waiting passenger car is 0.6 l/h [6].

There is not a fixed fuel consumption amount for accelerating vehicles. This amount depends on the vehicle type, fuel type, engine size, driver behaviour, etc. To determine the fuel consumption amount for accelerating, the consumption of different velocities should be known. Nasir et al. [28] showed the consumption difference based on different velocities and gears for a manual transmission passenger car. Considering that a standard driver tries to minimize fuel consumption, it can be assumed that the driver will shift gears at optimum timings. Under these conditions, the consumption amount of the vehicle from 0 to 70 km/h which is the local speed limit can be calculated based on the acceleration rate. Bokare and Maurya [9] determined the maximum acceleration of the cars as 2.87 m/s^2 which is a value for aggressive drivers. Tanyel and Çalışkanelli [33] determined the acceleration rate as 2.09 m/s^2 in Turkey, which is close to Bokare and Maurya [9]. On the other side, Kraft et al. [20] suggested the acceleration rate as 1.5 m/s^2. Therefore, the authors calculated fuel consumption for different acceleration rates. The calculations are made based on Newton's motion law. In the first case, a car increases its speed per second by 2.87 m/s. With this information, the amount of fuel that a stationary vehicle will consume while reaching the local speed limit of 70 km/h is 2.56 cc and the vehicle needs about 66 m to move to reach the speed limit. If the vehicle would drive the 66 m with a constant speed of 70 km/h, only 1.36 cc fuel would be consumed. So, 1.2 cc fuel is over consumed. Afterward, different acceleration rates are taken into consideration as 1.5, 2, 2.5 m/s^{2,} and the extra fuel consumption values are calculated as 2.4, 1.9 and 1.4 ccs, respectively. However, the acceleration rate is accepted as 2.87 m/s^2 and extra fuel consumption as 1.2 ccs for this study, because it is observed that the drivers are more aggressive during peak hour traffic.

2.6. Signal timing optimization

The key value in the design of signal timings is the cycle length. By calculating the optimum cycle length, green times could be calculated depending on the arrival flow or the resulting queue length ratios. Finding cycle length is an optimization problem that needs to be solved. Zargari et al. [41], optimized the signal times by maximizing the incoming flow. In this study, the delay is minimized to optimize the cycle length. To minimize the delay, the green time must be increased. However, if the green time is over increased, the vehicles waiting at the other links would have an unnecessary delay. Therefore, the total intersection delay should be taken into consideration instead of the delay of a single link. Since the total intersection delay analysis considers all links, it will give more accurate results than the delay analysis of individual vehicles. However, another important point here is that the number of full-stopping vehicles. In a city with a speed limit of 70 km/h, as described under the “Intersection Performance” section, there is an additional 1.2 cc of fuel that vehicles consume while speeding up to their original speed after a full stop. They also consume 0.16 cc/s fuel during idle waiting. Therefore, it is not enough to examine the delay parameter alone. At the same time, it should be prevented that not any car full stops twice at the same intersection. For this reason, the minimization of the extra fuel consumption at the intersection is preferred in the cycle’s length optimization. The object function for the optimization process is shown as Equation (8), where the cycle length’s minimum and maximum values are the constraints. Therefore, both delay and stop-go counts would be taken into consideration.

8
$$C_{opt} = \min_{C_{min}<C<C_{max}} \left(\alpha^* l_{departure}{}^* \sum_{i=1}^{n} Q + \beta^* l_{idle}{}^* \sum\nolimits_{i=1}^{n} D\right) \tag{19}$$

subject to:

$$T_{max} = 3600s,$$

9
$$C_{min} = n_{phase}(g_{min} + g_{inter}), \tag{20}$$

where, C_{min} is the minimum cycle length and comprises phase count (n_{phase}) time, the minimum green time (g_{min}) and green intervals (g_{inter}). Minimum green time should not be less than 5–10 s interval [17]. It is selected as 7 s for the departure of the last arriving vehicle securely based on the intersection geometry. And the minimum green time is selected as 7 s. The objective function mentioned in Equation (8) is the intersection performance function (Equation (7)).

Once the optimum cycle length is obtained, the cycle length is distributed to the intersection links as green times Here, the authors suggest a novel approach to distribute the cycle length to green times based on the discharging time of the current queue (Equation (10)) instead of the flow rates (Equation (11)). The green times are calculated by using the ratio of the flow of the link to the total flow of the intersection. The distributable time, the subtraction of the total lost time from the cycle length, has been proportioned based on the calculated ratios (Equation (11)). However, it is not proper when all the links of the intersection do not have the same number of lanes. Queue size needs more time to discharge when the link has less lane. For example, the same queue size needs twice as much time to discharge on a link with half the number of the lane. So, the distributable time has been proportioned based on discharge time ratios (Equation (10)) rather than flow ratios [5].

10
$$g_j = \frac{\frac{q_i}{\frac{n_i * S}{3600}}}{\sum_{i=1}^{m} \frac{q_i}{\frac{n_i * S}{3600}}} * (C - L) \tag{21}$$

11
$$g_j = \frac{q_i}{\sum_{i=1}^{m} q_i} * (C - L), \tag{22}$$

where; g_j is the green time for j. phase, q_i is the critical flow for i. link, m is the link number, n is the number of lanes, S is the saturated flow rate, C is the cycle length and L is the total lost time.

3. Experimental research

3.1. Introduction of the sample intersection

To measure the efficiency of the Monte Carlo based signal timing (MCaST) proposed in the theory, real field data were collected and compared with the estimated results. For this aim, real field data from an adaptive working intersection in Denizli, a city of Aegean region in west Turkey with the coordinates 37.7830°N and 29.0963°E (Fig. 2), were used. An intersection with an adaptive signal control has been selected and the geometric form of the intersection is shown in Fig. 3. The data is obtained from the intersection at peak hour, 17.00–18.00 interval. The saturated flow rate is accepted as 1800 vehicle/h-lane as suggested by Webster and Cobbe [39].

Lane – Carril

3.2. Statistics of the sample intersection

With the help of the loop sensors, the arrival times of vehicles have been obtained. Hence MCaST algorithm is based on arrival flow at the link, not at the lane, the counts at the north side and south side points are merged into one and the timings are arranged from smallest to largest. Afterward, the arrival times are subtracted from the previous vehicle’s arrival time, and the time gap between vehicles is obtained. As mentioned above, the intersection works adaptively and regulates the green times and cycle lengths at the end of each cycle based on loop sensor data. The statistical parameters of the real field arrival flow are given in Table 1. The frequency analysis of the arriving vehicles is shown in Fig. 4. There, the headways in seconds are rounded up to the nearest multiple of 0.5 for the frequency analysis to make a more precise analysis.

After the first view of the data, the most sparsely arriving vehicles are at the east link of the intersection. The south link got traffic flow more often than the rest. To determine the most proper statistical distribution, corresponding parameters have been obtained by using the maximum likelihood approach. The parameter values are given in Table 2. The parameters’ statistical significance values were calculated using Chi-square ($\mathscr{X}^2$) and Kolmogorov-Smirnov (K-S) goodness-of-fit tests are given in Table 3. All the probability distribution graphs are shown in Fig. 5.

3.3. Selection of the statistical distribution

After the statistical simulations, the most appropriate distribution should be selected for each link. For this purpose, the relationship between empirical data and statistical distribution data was established. The statistical distribution with the strongest relationship was determined as the most appropriate distribution. Correlation analysis was performed to determine the strong relationship. In the correlation analysis, the probabilities of each statistical distribution were compared with the empirical data probabilities and the statistical distribution with the highest correlation coefficient was determined as the most appropriate distribution. When the correlation relationships were examined, the best fit has been obtained by generalized extreme values (R 92.5%), Birnbaum-Saunders (R 75.8%), generalized extreme values (R 90.9), and generalized extreme values (R 92.9%) distributions for North, East, South and West links, respectively. From this point, the study is continuing by using related distributions.

Fig. 2. Location of Denizli (Map Data: Google, Maxar Technologies, Basarsoft).

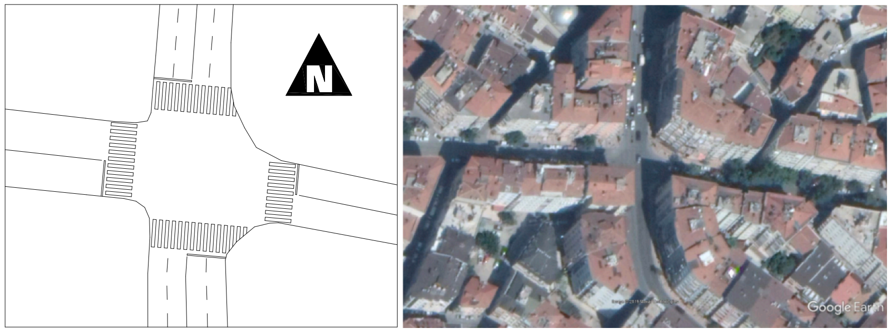

Fig. 3. Drawing (left) and google earth (right) view of the selected intersection for the study (Map Data: Google, Maxar Technologies).

Table 1
Statistical parameters calculated for the real field arrival flow's time intervals for one hour period.

Parameters	North	East	South	West
Count of Data (vehicle)	614	179	692	330
Peak Value (s)	2	6	2	2.5
Median Value (s)	3	11.5	3	5.5
Aritmetic Mean (s)	5.86	20.04	5.16	10.88
Standard Deviation	6.98	22.55	6.26	13.56
Variance	48.72	508.31	39.24	183.97
Skew Coefficient	2.97	2.10	3.53	2.65

After fitting the empirical data to one of the statistical distributions, the function of the statistical distribution becomes the objective function for the MCaST algorithm. The flow generated via the algorithm has been compared with the loop sensor data and high correlations have been found. However, the generated flow could concentrate either at the beginning or at the end of the simulation. So, while the total simulated system seems right, the calculated signal timings could be unnecessarily long or too short for most of the cycles. Therefore, a trend analysis has been conducted where the distribution of time intervals between simulated and real vehicles are compared. The trend analysis results are given in Fig. 6. The statistical parameters of the model simulated flow and the comparison are given in Table 4.

Trend analysis has also been done to determine whether the algorithm could generate vehicles close to real vehicles or not. Fig. 6 shows the trend analysis results, and as seen in the trendline equations, the coefficients are close to each other. After obtaining the high correlation and almost the same trend, it can be assumed that the generated vehicles using the simulation could be used for the intersection analysis by simulating the actual traffic conditions.

3.4. Analysis of the simulated traffic conditions

Queue analysis has been done to the Monte Carlo generated traffic flow data. To make the queue analysis, the saturated flow rates have been calculated as 3600 vehicle/h for North and South links while 1800 vehicle/h for East and West links. The cycle length has been selected as 140 s because the real cycle length of the traffic signal control runs for 137–140 s but mostly 140 s. When the MCaST is set as changing cycle length, the algorithm used 137 s for the whole analysis, so the cycle length is set as constant 140 s. However, the system changed the green times on the links at the end of all cycles. And when the calculated green time is shorter than the pre-given minimum green time, so the phase is skipped for the cycle. Here, with each phase omitted, the intergreen time is gained, and the times obtained are distributed to the other active phases in the ratios of the green time needed to discharge the queue. Besides 140 s as cycle length, the algorithm has also been run for different cycle lengths. The total stopping counts have been shown in Fig. 7. As seen in Fig. 7, the total stopping counts have been decreasing while the cycle length is increasing, which is an expected situation.

When the queue lengths have been calculated, the vehicle delay is the next parameter that is needed for the intersection performance. Therefore, the delay has been obtained for each vehicle individually.

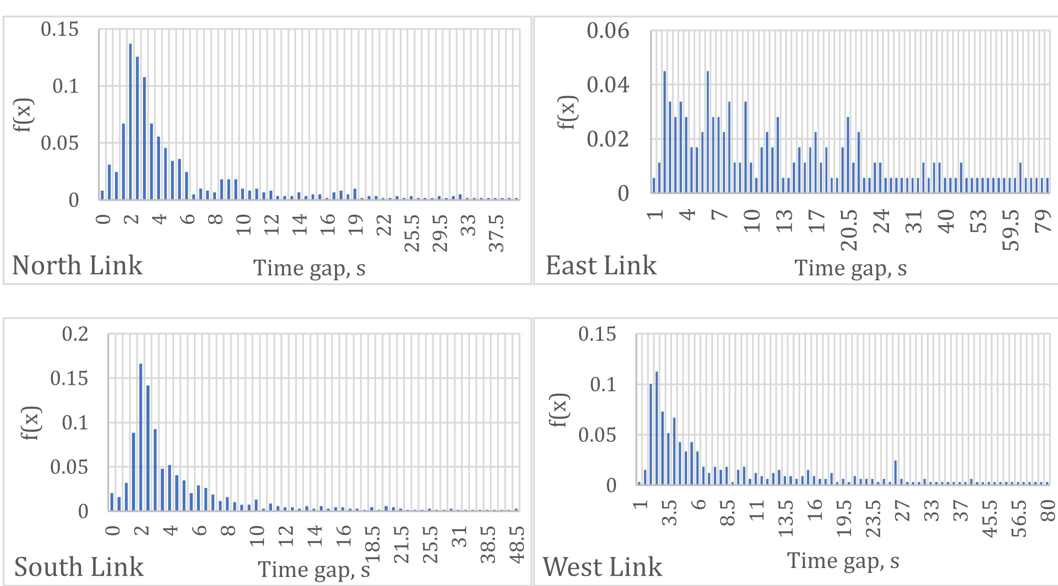

Fig. 4. Frequency analysis of the intersection.

Table 2
Distribution parameter values for each statistical distribution.

Distribution	Parameter	North	East	South	West
Exponential	μ	5.83469	18.0112	5.15824	10.7288
Lognormal	μ	1.29915	2.42734	1.10788	1.75899
	σ	1.02358	0.994468	1.40519	0.890215
Gamma	a	0.728251	1.21804	0.677991	1.11723
	b	8.01192	14.787	7.60812	9.60302
Generalized Pareto	k	0.12201	0.019675	0.11928	0.194281
	σ	5.1135	17.6573	4.5241	8.6548
	ζ	$-2.22\text{x}10^{-15}$	0	$-2.22\text{x}10^{-15}$	0
Inverse Gaussian	μ	5.83469	18.0112	5.15824	10.7288
	λ	0.001228	11.6146	0.049014	7.6552
Generalized Extreme Value	k	0.476206	0.620472	0.412844	0.864264 k
	σ	2.1753	7.00643	1.99457	3.01907 σ
	μ	2.76958	7.88696	2.55731	3.9742 μ
Birnbaum-Saunders	β Escale	0.009839	11.2647	0.167267	7.05109
	γ Chape	24.475	1.0928	5.68185	1.04875
Log-Logistic	μ	1.28967	2.43066	1.19269	1.7858
	σ	0.557241	0.581373	0.560298	0.560778

$$\frac{4.3712\,(19.13433)}{(9.9710)^{1.3}} \quad 31.485$$

Table 3
Significance values of the Chi-square ($\mathscr{X}^2$) and Kolmogorov-Smirnov (K-S) goodness of fit tests.

Distribution	North		East		South		West	
	K-S	$\mathscr{X}^2$	K-S	$\mathscr{X}^2$	K-S	$\mathscr{X}^2$	K-S	$\mathscr{X}^2$
Exponential	$3.4*10^{-13}$	$2.1*10^{-21}$	$2.0*10^{-3}$	$5.9*10^{-5}$	$1.0*10^{-13}$	$9.0*10^{-25}$	$7.5*10^{-9}$	$5.7*10^{-18}$
Lognormal	$1.7*10^{-14}$	$7.6*10^{-25}$	$6.5*10^{-5}$	$3.6*10^{-5}$	$3.6*10^{-12}$	$7.3*10^{-17}$	$3.1*10^{-15}$	$5.8*10^{-32}$
Gamma	$5.4*10^{-13}$	$4.9*10^{-27}$	$4.3*10^{-3}$	$7.4*10^{-6}$	$2.1*10^{-13}$	$2.3*10^{-22}$	$9.3*10^{-9}$	$1.8*10^{-19}$
Generalized Pareto	$1.0*10^{-13}$	$7.7*10^{-23}$	$1.6*10^{-3}$	$5.9*10^{-5}$	$3.5*10^{-14}$	$6.7*10^{-26}$	$1.8*10^{-10}$	$3.1*10^{-18}$
Inverse Gaussian	$2.1*10^{-48}$	0	$1.6*10^{-6}$	$9.4*10^{-6}$	$2.5*10^{-34}$	0	$1.8*10^{-11}$	$2.7*10^{-20}$
Generalized Extreme Value	$9.0*10^{-17}$	$1.5*10^{-34}$	$1.2*10^{-5}$	$7.9*10^{-6}$	$4.1*10^{-17}$	$2.6*10^{-40}$	$6.8*10^{-15}$	$1.1*10^{-22}$
Birnbaum-Saunders	$4.9*10^{-24}$	$1.4*10^{-39}$	$1.6*10^{-4}$	$1.4*10^{-4}$	$1.9*10^{-22}$	$9.6*10^{-37}$	$6.8*10^{-10}$	$2.8*10^{-18}$
Log- Logistic	$5.0*10^{-16}$	$1.0*10^{-30}$	$3.2*10^{-5}$	$1.6*10^{-5}$	$6.9*10^{-16}$	$8.8*10^{-32}$	$2.6*10^{-14}$	$5.8*10^{-26}$

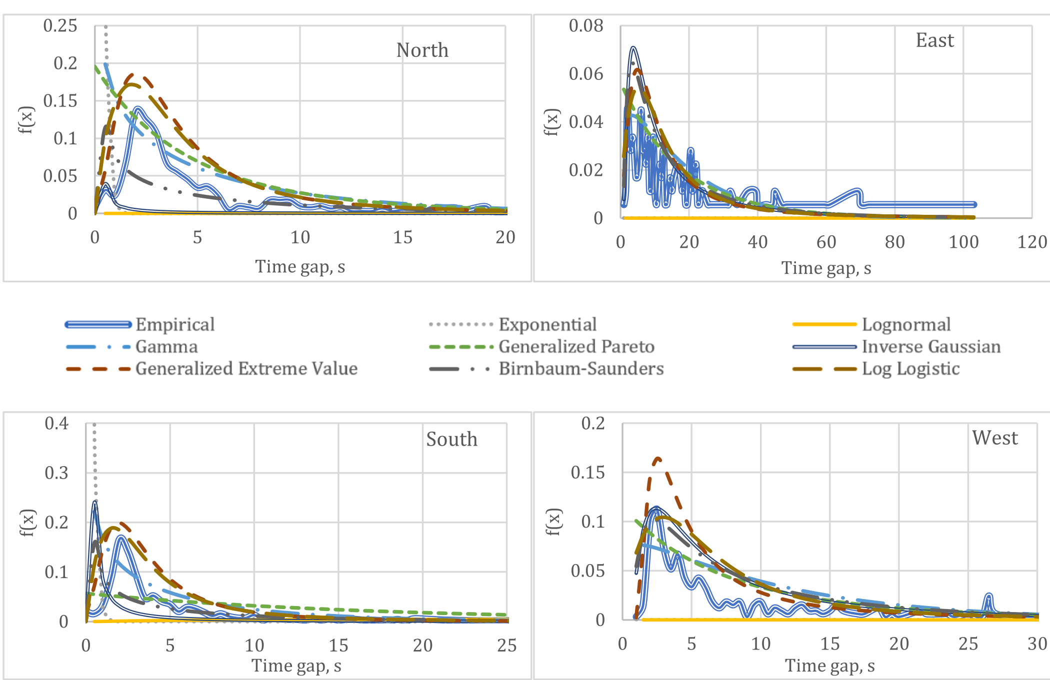

Fig. 5. Distribution graphs of the intersection links.

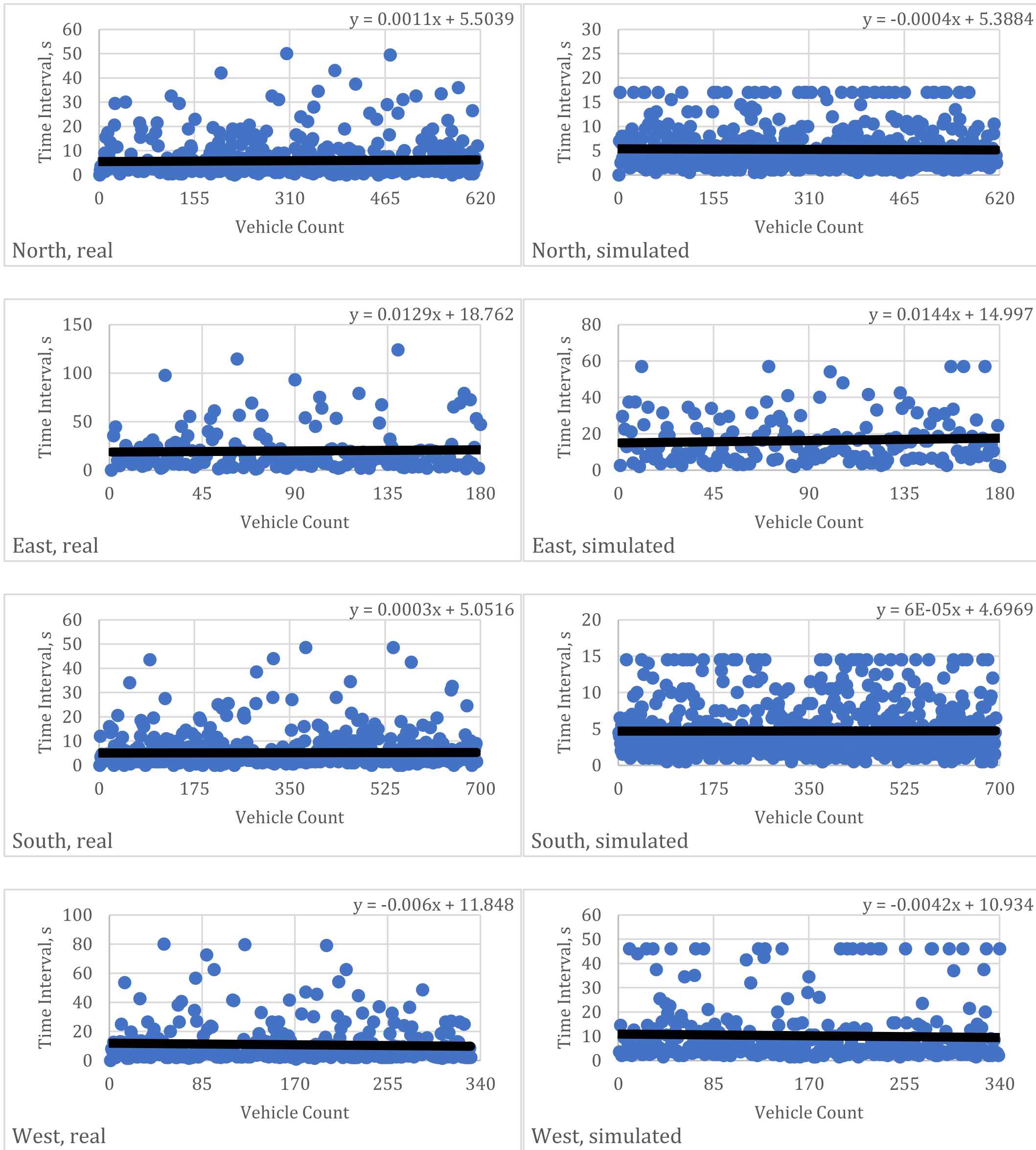

Fig. 6. Trend analysis of the real and simulated vehicle's time interval.

Table 4
Statistical parameters calculated for the Monte Carlo simulated flow's time intervals for the one hour and comparison with the real field data.

Parameters	North		East		South		West	
	S*	R*	S*	R*	S*	R*	S*	R*
Count of Data (vehicle)	614	614	179	179	692	692	330	330
Peak Value (s)	2.5	2	7.5	6	2	2	2.5	2.5
Median Value (s)	4	3	13	11.5	3.5	3	5	5.5
Aritmetic Mean (s)	5.27	5.86	16.30	20.04	4.72	5.16	10.21	10.88
Standard Deviation	4.14	6.98	12.82	22.55	3.57	6.26	12.05	13.56
Variance	17.12	48.72	164.25	508.31	12.78	39.24	145.13	183.97
Skew Coefficient	1.56	2.97	1.33	2.10	1.37	3.53	2.10	2.65

*S – simulated data; R – real data.

Then the delay values are added, and the total intersection delay has been obtained. However, the simulation generates different counts of vehicles each time when the algorithm runs. So, it is calculated the average delay time at the end of each simulation set. The average delay time is then calculated by dividing the sum of each vehicles' delay by the vehicle count. The change of the average delay time for different cycle lengths is given in Fig. 8.

As seen in Fig. 8, the average delay time for 60 s cycle length is shorter than the average delay time for 70 s cycle length. However, after 70 s, the average delay time is decreasing properly until about 120 s. Normally, the increase of the delay at the beginning is unexpected. However, since the algorithm regulates the green time at the end of each cycle the average delay value is smaller at 60 s. The situation explains that, at the end of each cycle, the calculated green time is obtained

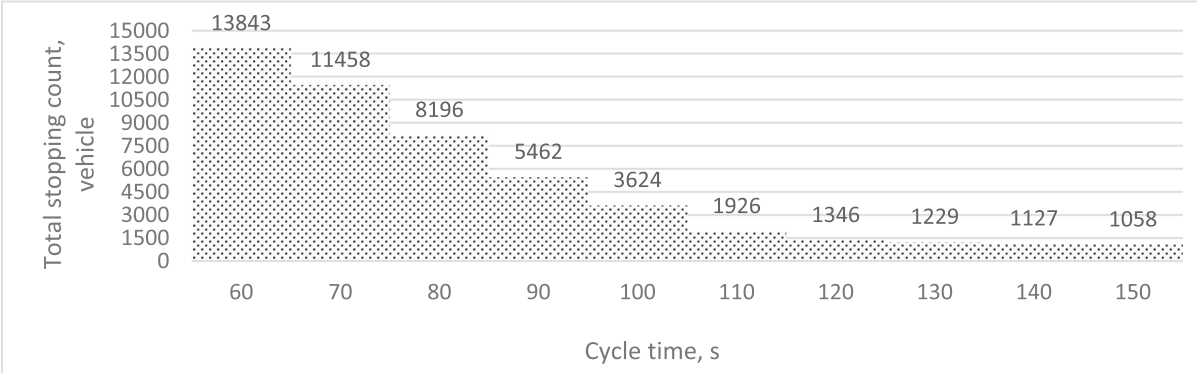

Fig. 7. Total stopping counts obtained using MCaST for different cycle lengths.

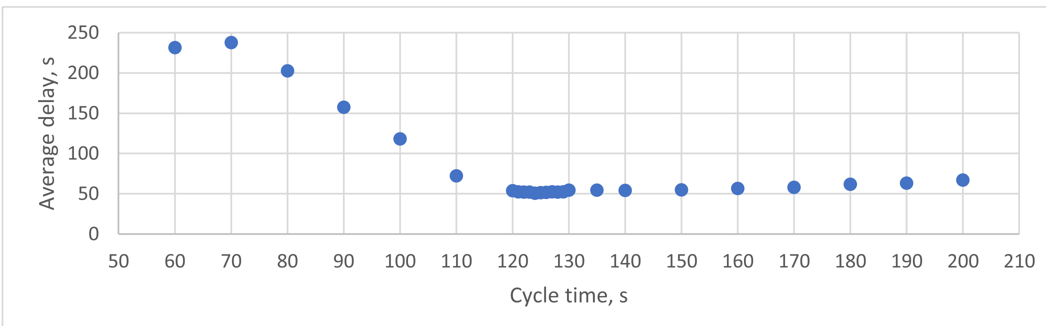

Fig. 8. The average delay for different cycle length for the one hour.

Table 5
Comparison of the delay results obtained from VISSIM, Webster, HCM2010 and proposed MCaST algorithm for different cycle length and flow/capacity ratio.

Cycle length, s	VISSIM Delay Results, Vehicle-h/h	Webster's Delay Formula, Vehicle-h/h	HCM2010 Delay Formula, Vehicle-h/h	MCaST Delay Results, Vehicle-h/h	Max Degree of Saturation, x_{max}	Average Degree of Saturation, x_{mean}
60	137.76 (%100)	N/A	236.26 (+%71.5)	160.63 (+%16.6)	1.28	1.22
70	97.15 (%100)	N/A	111.31 (+%14.58)	121.81 (+%25.38)	1.17	1.11
80	66.76 (%100)	N/A	72.41 (+%8.46)	96.47 (+%44.5)	1.02	0.99
90	63.01 (%100)	18.48 (-%70.67)	41.53 (-%34.09)	71.76 (+%13.89)	0.96	0.94
100	54.66 (%100)	20.27 (-%62.92)	36.08 (-%33.99)	58.93 (+%7.81)	0.92	0.91
110	22.76 (%100)	21.76 (-%4.39)	32.17 (+%41.34)	30.28 (+%33.04)	0.88	0.87
120	27.97 (%100)	23.57 (-%15.73)	33.08 (+%18.27)	31.43 (+%12.37)	0.86	0.85
130	23.39 (%100)	25.19 (+%7.7)	33.12 (+%41.6)	26.49 (+%13.25)	0.83	0.82
140	24.14 (%100)	27.31 (+%13.13)	35.4 (+%46.64)	28.07 (+%16.28)	0.82	0.81
150	28.08 (%100)	28.27 (+%0.68)	34.64 (+%23.36)	24.35 (-%13.28)	0.80	0.80
160	28.23 (%100)	30.41 (+%7.72)	36.65 (+%29.83)	29.49 (+%4.46)	0.80	0.79
170	28.95 (%100)	32.05 (+%10.71)	37.92 (+%30.98)	26.93 (-%6.98)	0.78	0.78
180	28.69 (%100)	33.79 (+%17.78)	39.38 (+%37.26)	28.11 (-%2.02)	0.78	0.77
190	29.16 (%100)	35.61 (+%22.12)	41.04 (+%40.74)	29.2 (+%0.14)	0.76	0.76
200	32.41 (%100)	37.19 (+%14.75)	42.35 (+%30.67)	30.41 (-%6.17)	0.76	0.75

shorter than the minimum green time. Therefore, the algorithm skips the link and gains the intergreen time, which is then distributed to the active phases. Therefore, the total green time is longer than the total green time which occurs at a cycle of 70 s.

The total delay at real field data has been calculated as 40.22 vehicle-hour/hour. This means, 40.22 vehicles of the flow are fully stopped for one hour. Vehicle sensor data and the green times start and end times are compared to calculate the real field delay. The total delay in the simulated vehicles at 140 s cycle length has been obtained as 28.07 vehicle-hour/hour. Therefore, MCaST arranged green times seem more efficient than the proper field data obtained from an adaptive working intersection, which decreases the total delay time by about 30.2%. Besides, the suggested delay formula is compared with commonly used two other delay formulas; Webster's delay formula and HCM2010′s delay formula. Both delay formulas could only be used when the signal-controlled system is running by predetermined green times, and not when the system is adaptive. So, to make a comparison, the algorithm has been run for predetermined green times for different cycle lengths.

The calculated delay times are shown in Table 5, where x is the degree of saturation. The results are compared not only with the delay formulas of Webster and HCM2010 but also with widely used software from the PTV group for traffic simulation and planning, VISSIM (Vehicles in the city – simulation). VISSIM has been introduced in 1994 and can analyze the traffic flows, integrate public and private transportation, and optimize the signal timings [15]. Besides the Webster's and HCM2010′s delay results, the same traffic condition such as traffic volume, green and red times are given to the software and the intersection has been analyzed for one hour. The obtained delay results are compared with each other. The mean error difference percentage of the delay results is shown in Fig. 9, as well. Therefore, the VISSIM delay is supposed as 100%, and the differences are written below the delay value. As seen in Table 5, the delay results obtained from MCaST seem more logical than the other two formulas, Webster and HCM2010. While the traffic is over-saturated, HCM2010 predicts greater delay values which are also mentioned previously by Akcelik [2] while Webster's formula underestimates the delay. However, for conditions with a lower flow/capacity ratio, Webster's formula predicts closer delay results than calculated by HCM2010. But for all the conditions, MCaST predicts more accurate delay results than the other two.

After obtaining the delay and queue values of the simulated vehicles, the intersection performance could be calculated. Here, it is assumed that all the vehicles are stationary passenger cars since only the loop sensor data are available. This section aims to test the consistency of the performance data to be obtained. Therefore, the one-hour delay of each vehicle consumes 0.6 l more fuel, and each vehicle consumes 1.2 cc more after a full stop. So, extra fuel consumption is shown in Table 6.

When all the values (total queue, total delay, average delay, consumed extra fuel) are obtained, the optimum cycle length could be determined. For example, when Table 6 is examined, the optimum cycle length is 150 s if the aim is to minimize the total queue. 120 s cycle length could minimize the delay, and 140 s minimize the consumed extra fuel at the intersection. As a result, the optimum cycle length changes based on the desired parameter.

Table 6
Consumed extra fuel at different cycle lengths.

Cycle length (s)	Generated Flow (vehicle/h)	Total Stop Count (vehicle)	Total Delay (vehicle-h/h)	Average Delay (s)	Consumed Extra Fuel (l)
60	1830	13,843	117.7726	231.6838	87.27516
70	1832	11,458	120.9703	237.7146	86.33178
80	1844	8196	103.8101	202.6661	72.12126
90	1830	5462	80.0878	157.5498	54.60708
100	1839	3624	60.3988	118.2358	40.58808
110	1833	1926	36.781	72.23764	24.3798
120	1858	1346	**27.7484**	**53.76439**	18.26424
130	1861	1229	28.2797	54.70549	18.44262
140	1859	1127	28.0678	54.354	**18.19308**
150	1862	**1058**	28.35019	54.8124	18.27971

Table 7
CPU processing time based on different cycle length and degree of saturation.

Cycle length (s)	x_{mean}	Q (vehicle/h)	CPU_{time} (s)
60	1.22	1819	4.45
70	1.11		4.40
80	0.99		4.35
90	0.94		4.31
100	0.91		4.28
110	0.87		4.27
120	0.85		4.31

CPU times used for solving the optimization problem for different cycle lengths and the degrees of saturation are given in Table 7. As seen in Table 7, the increase of the cycle length decreases the degree of saturation for the intersection while the flow rate is constant. The decrease in the degree of saturation values also decreases the CPU processing times.

4. Conclusion

In this study, a novel adaptively working traffic signal model has been developed based on the Monte Carlo simulation approach. Considering the findings, the following conclusions could be drawn:

- the developed MCaST algorithm can simulate real field flow with high similarity.
- Distributions such as exponential, normal, and gamma, which are traditionally used to model the gap values of vehicles in traffic, are not reproducing the data well. With these distributions, modeling cannot be performed at an adequate level. Instead, generalized

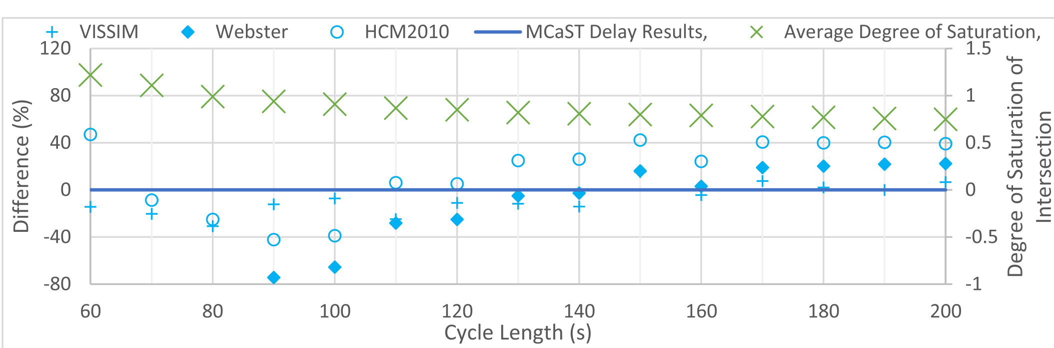

Fig. 9. Relative difference plot of the delay results.

extreme values, generalized Pareto and Birnbaum - Saunders distributions gave more appropriate results.
- With the Monte Carlo simulation, traffic flow can be reproduced, and realistic results can be obtained. Since the model adapts itself to the real field data, it can be claimed that the model can consider the local driver behaviors and make adaptive decisions. Therefore, the developed model can be used on a universal scale.
- The delay value of the vehicles on a microscopic scale and the queue could be calculated using Monte Carlo simulation.
- Developed adaptively working model skips phases if the green time needed is shorter than minimum green time and distribute the skipped green time to the active phases with the rate of the time needed to discharge the queue.
- Optimum cycle time depends on the parameter that the user aims to optimize.

It should be noted that all these conclusions are made based on the comparison made with delay formulas Webster and HCM2010 (5th Edition). However, Transportation Research Record updated the manual and published 6th Edition [36]. Although the main idea for calculating the delay is the same like the sum of the uniform delay, incremental delay and initial queue delay, a newly defined coefficient has been used to adjust the uniform delay. Based on this information, delay results calculated with HCM2016 delay formula would be calculated less than results calculated using HCM2010 delay formula. However, since the delay formula given with HCM2016 is used for predetermined times as mentioned before, it is expected that the existing delay difference may decrease but cannot be closed for an adaptive working intersection. But future works should be studied to reach a definite judgment.

As a result, the generated algorithm can calculate all the parameters needed for the design of a signalized intersection. In the sensitivity analysis, it was also possible to calculate delay and cycle length for different traffic volumes and different distribution functions. It can work under saturated conditions and over-saturated conditions. For future work, because the proposed algorithm could skip the unnecessary phases, sustainable intersections could be built using a weighted strategy to minimize queue delays and vehicular fuel consumption/emission. Also, the proposed model could be improved to develop an emergency priority intersection for emergency management.

CRediT authorship contribution statement

Ekinhan Eriskin: Conceptualization, Formal analysis, Investigation, Methodology, Software, Visualization, Writing – original draft, Writing – review & editing. **Serdal Terzi:** Conceptualization, Methodology, Supervision, Writing – review & editing. **Halim Ceylan:** Conceptualization, Resources, Supervision, Validation, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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