Design of an Efficient Vehicle-Actuated Signal Control Logic for Signalized Intersections

Ziya CAKICI\textsuperscript{a,*}, Yetis Sazi MURAT\textsuperscript{b}, Metin Mutlu AYDIN\textsuperscript{c}

\textsuperscript{a} Bayburt University, Faculty of Engineering, Department of Civil Engineering, Bayburt 69000, Turkey
\textsuperscript{b} Pamukkale University, Faculty of Engineering, Department of Civil Engineering, Denizli 20160, Turkey
\textsuperscript{c} Ondokuz Mayis University, Faculty of Engineering, Department of Civil Engineering, Samsun 55000, Turkey

* Corresponding author:
E-mail: civilengineerziyacakici@gmail.com; Phone no: +90 554 204 5924

Abstract
The effectiveness of fixed-time management systems dramatically decreases in case of fluctuated traffic demands at signalized intersection approaches. This leads to the waste of time in traffic and may cause material, psychological and ecological problems. Especially in recent years, to minimize the negative impacts of these problems, many researchers focus on Intelligent Transportation Systems (ITS). As it is known, one of the Intelligent Transportation Systems applications is called vehicle-actuated (traffic-actuated) management systems. Because the success of these types of applications is directly related to the created control logic, the selection of the most proper control parameters for the created control logic is an important issue. This study aims to create an effective control logic and flow chart for the vehicle-actuated management system. At the end of the analyses, it is seen that the created vehicle-actuated management system can adapt to fluctuations in traffic demands at signalized intersection approaches. Average vehicle delays, fuel consumptions and exhaust emissions can be reduced significantly by the created system. Especially, in case of fluctuations in traffic demands at
intersection approaches exist, it is concluded that the performance of the intersection can
increase enormously with the created vehicle-actuated management system.

**Keywords:** Fluctuated traffic demand; Vehicle-actuated management, Delay, Fixed-time
management, Simulation.

1. **Introduction**

Economic and technological developments all over the world make life easier. However, this
situation may cause several negative effects in many areas. Highways are undoubtedly one of
the areas where these problems are mostly seen. In parallel with the increase in demand for
passenger and freight transportation, the number of vehicles has increased. This increase can
cause many economic and environmental problems [1-3]. To minimize or eliminate the
negative effects of these problems on humans and the environment, geometric-topographic and
operational arrangements at road networks can be pointed out as important parts of
environment-economy-friendly measures and reasonable solutions. At this stage, scientific-
based analysis and solution approaches should be preferred. Minimizing the stop numbers and
delays is an important step to solve this problem at signalized intersections [4]. The
minimization of delays and stops at signalized intersections can be achieved in two different
ways: with most proper designing of intersection geometry (geometric and topographic
arrangements) and with determining of signal timings considering traffic demands at
intersection approaches (operational arrangement) [5-8]. Hence, effective signal management
is a quite important factor to reduce the delay, fuel consumption, noise pollution and exhaust
emissions and to increase the intersection capacity and level of service of intersection [9-12].
In the absence of the fluctuations in traffic demands at intersection approaches, the optimum
fixed-time management system, which is widely used today, maybe the right and appropriate
choice to manage the traffic flows at intersections [13]. But sometimes sports events, concerts,
traffic accidents, bad weather conditions can cause short or long-term fluctuations in traffic
demands at some intersection approaches. In such cases, the operating efficiency of the optimum fixed-time management system decreases especially at intersection approaches where the fluctuations in traffic demands exist. This situation may cause long-term delays, long queues, excessive fuel consumption and exhaust emissions [13-16]. It has been demonstrated in many studies by researchers that in the event of fluctuations in traffic demands, optimum fixed-time signal management systems cannot provide efficient results [17, 18].

For the last 60 years, a lot of studies have been conducted on signal control systems. And, the studies have continued today to improve their performances [19-21]. Since the last quarter of the 20th century, it has been also aimed to make these systems more effective for high traffic demands at signalized intersections and thus it has been aimed to increase the operational efficiency further [22-26]. Especially in recent years, there has been a significant increase in the number of studies on the signal management systems and control algorithms that take the instantaneous traffic flows into account. These systems may appear in different ways either optimization or created control algorithm based [27, 28].

Traffic management systems that are operated based on any created control algorithm are referred to as vehicle-actuated (traffic-actuated) signal management systems. Vehicle-actuated management systems determine the green and red signal timings according to the current dynamics of vehicle arrivals and queuing information obtained by detectors placed on the lanes at intersection approaches [28]. Because vehicle-actuated signal management systems generally have positive effects on the intersection performance, many researchers have worked on this issue [26]. Some of these studies can be summarized as follows:

Akçelik [22] proposed an approach to estimate the average green times and cycle time for vehicle-actuated management systems. He made an evaluation on the prediction of vehicle arrival headways and tried to determine the optimum cycle time. Trabia et al. [29] worked on fuzzy logic-based vehicle-actuated management systems. As a result of the study, they
determined that delays can be reduced by 10% with a fuzzy-based approach. Kim and Courage [30] proposed a maximum green time design method for a vehicle-actuated management system that minimizes delay. Guo et al. [31] developed the two analytical models to optimize the maximum green times which are applied on vehicle-actuated management systems. They specified that these models could adjust maximum green time systematically and cyclically by adapting to the changes in traffic demands at intersections. Zheng et al. [32] worked on a control model which aims to provide adaptive functionality of vehicle-actuated management systems. As a result of the study, it was found that the model has a great potential to improve the performance of the signalized network. Jiang et al. [33] developed a method to optimize the parameters of the vehicle-actuated signal management systems. In another study, Grether et al. [13] evaluated the vehicle-actuated signal control strategy. As a result, they concluded that travel times can be significantly reduced with the vehicle-actuated management system, especially in the event of excessive increases in traffic demands. Swaminathan et al. [34] developed a vehicle-actuated management model. According to analysis results, it was determined that the average delays can be reduced by 28% compared to the current situation with the developed control model. In a similar study, Guo and Ma [35] aimed to develop a new vehicle-actuated signal control model, which resulted in lower average vehicle delays compared to both fixed-time and standard actuated control systems. Ribeiro and Simoes [36] used global optimization and complementarity to determine the green and red signal timings at each cycle for vehicle-actuated management systems. At the end of the analysis, it was concluded that the suggested methodology can be used for the determination of the green and red signal timings, as well as cycle lengths. Lee and Maleki [37] studied on maximum green time settings for vehicle-actuated signal control at isolated intersections. They aimed to determine maximum green times using fuzzy logic approach, dynamically. As a result, it was determined that fuzzy logic is a useful approach to overcome the difficulty of setting suitable maximum green time
for any phase. In a different study, Wang et al. [38] investigated the green time extension for vehicle-actuated signal control. As a result, they found that the optimum critical headway value, which is an important parameter for green time extension, decreases with an increase in traffic demand and in some cases, this value may be more than 2-3 seconds as a common assumption. Promraksa et al. [39] studied on vehicle-actuated signal management system for reducing CO$_2$ emission. At the end of the analyses, it was concluded that a fully-actuated management system can provide better and more encouraging results than the fixed-time and semi-actuated management systems.

As can be seen from the previous studies, the selection of the most proper design parameters (such as minimum critical headway, placement of detectors, minimum green signal timings, maximum green signal timings) for the vehicle-actuated management systems has a direct and significant effect on the system performance. This study guides for designing an efficient vehicle-actuated signal control logic for signalized intersections. In the scope of the study, the selection of the most proper design parameters was evaluated, separately. Then, a new vehicle-actuated management system for a four-leg signalized intersection was developed in **VISSIM Vehicle Actuated Programming (VISVAP)**. The efficiency of the developed system was tested on 20 different traffic scenarios considering the different performance criteria (vehicle delay, fuel consumption and exhaust emission) in VISSIM. To determine the success of the developed system, unlike the current studies in the literature, short and long-term fluctuations in traffic demands at the signalized intersection approaches were also taken into account in the analysis. It is seen that the developed system can adapt to the fluctuations in traffic demands and enormously reduce the average vehicle delay, fuel consumption and exhaust emission at the signalized intersection. Thus, it is determined that the developed system can be used for improving the performance of signalized intersections.

2. Method
2.1. Vehicle-Actuated Management Model for Four-Leg Signalized Intersections: VAM4

In vehicle-actuated management, traffic flows at an intersection are managed using data and information from the detectors which are located at the intersection approaches [18]. Vehicle-actuated management systems can be classified in two ways as semi-actuated and fully-actuated. In semi-actuated management, only one or a few of the intersection approaches have detectors. In fully-actuated management, all intersection approaches have detectors and traffic flow data and information are provided from all approaches. In this type of management, the green signal timings and the cycle times are determined by considering the traffic demands detected. In fully-actuated management systems, the order of phases can be fixed or flexible. Furthermore, if there is no demand for a phase, that phase can be skipped directly. In vehicle-actuated management systems, the effective operation of an intersection generally depends on the following factors: minimum green times (sec.), critical arrival headways of traffic flow, placement of detectors (detector locations), maximum green times (sec.). Therefore, choosing reasonable and appropriate values for these factors is crucial to improve signalized intersections’ performance [14, 35]. In this study, the performance of the vehicle-actuated signal management system (VAM4) developed for a four-leg signalized intersection which includes scenarios, where short or long-term fluctuations exist in traffic demands at intersection approaches, was examined. For this purpose, firstly, the control parameters and criteria related to the control algorithm of the proposed VAM4 were determined. In the analysis, considering the high traffic volumes, the minimum critical arrival headway is selected as approximately 2 seconds. In this case, as long as the critical arrival headway determined by the detectors does not exceed 2 seconds, the green signal timing of that phase is extended by 2 seconds. When the current studies in the literature are examined, it can be seen that this value is generally selected between 1.5 and 4 seconds. [14, 40]. Therefore, the selected green extension value (2 seconds) was appropriate for the analysis. Besides, when the previous studies are examined carefully,
there is no clear and precise information about the placement of detectors for vehicle-actuated management systems. However, the placement of the detectors was directly related to the traffic volume as a common practice in the literature [14, 15, 40]. According to previous studies, the detectors were placed generally 15 m to 45 m before the stop lines. However, there is no any precise information about the placement of the detectors in the literature. In this study, the locations of the detectors were determined by trial and error method for each traffic scenario. According to the results obtained by trial and error, the relationship between the distance of the detectors to the stop line at the intersection approaches and the average delay was determined as given in Fig. 1. This relationship is similar to the relationship depicted by Bullen [14].

When the previous studies are examined, the minimum green time is generally selected as 4-5 seconds or above depending on the distance of the detector to the stop line [14, 15]. As mentioned before, especially when the traffic volumes at intersection approaches are high, the distance of the detectors to the stop line can be up to 45 meters. For this reason, the minimum green time value must be determined considering locations (placements) of the detectors at the intersection approaches. Determined minimum green time must allow to clearing of the queues at intersection approaches. Depending on the locations of the detectors, the minimum green times for the vehicle-actuated management systems can be calculated by using Eq. (1) [15]:

\[
G_{\text{min}} = t_L + \left[ h \times \text{integer} \left( \frac{l}{e} \right) \right] \\
\]

Where;

\( G_{\text{min}} \) : Minimum green time (sec),

\( t_L \) : Assumed start-up lost time (sec),

\( h \) : Assumed saturation headway (sec),

\( l \) : Distance between detectors and stop line (m),

\( e \) : Distance between storaged vehicles (average 6-7 m).
In the scope of this study, minimum green times are computed by using Eq. (1). Depending on the distance between detectors and stop line, the obtained minimum green times are presented in Table 1.

As can be seen from the previous studies, the maximum green time for vehicle-actuated management systems is generally selected between 45 and 60 seconds [41]. In this study, the maximum green time of each phase is selected as 60 seconds and this value is used in the analysis. General information about the proposed VAM4 for the four-leg signalized intersection model is shown in Fig. 2.

As can be seen from Fig. 2, the modeled four-leg signalized intersection has a total of 14 detectors for 14 lanes (one detector for each lane). These detectors are used for both detecting the queuing at intersection approaches and determining the vehicle arrival headways. In the developed VAM4 model, it is assumed that the modeled four-leg signalized intersection is operated with four phases. Right of way for each intersection approach is provided in separate phases. Right of ways for the West, North, East and South intersection approaches are provided in Phase I, Phase II, Phase III and Phase IV, respectively. Besides, in the developed VAM4 model, when excitation is not detected from the detectors which are at one of the intersection approaches, phase transition can be actualized (the order of phases can be changed). The simulation period for the analysis is determined as 3600 sec. The flow chart of the developed VAM4 control logic is presented in Fig. 3.

2.2. Traffic Scenarios and Fluctuations in Traffic Demands

In the scope of this study, firstly, a movement direction-based reference demand matrix has been created considering the four-leg signalized intersection model shown in Fig. 2. This created reference demand matrix for the analysis is given in Table 2. In the next step, a total of 20 different traffic scenarios are created by using the reference demand matrix given in Table 2 for the modeled four-leg signalized intersection. For each
created scenario, movement direction-based traffic volumes showed an increase or decrease based on the reference demand matrix. Besides, some of the movement direction-based traffic volumes are constant in some scenarios. The increase and decrease rates of the movement direction-based traffic volumes for all scenarios are shown in Fig. 4.

As can be seen from Fig. 4, when the scenario number increases, general increase trends are seen in movement direction-based traffic volumes. Therefore, the total traffic volumes at the intersection increase when the scenario number increases as shown in Fig. 5. As can be seen in Fig. 5, created traffic scenarios are divided into four groups considering total traffic volumes at the intersection. According to their demand levels, these groups are named as low, moderate, high and very high. When the hourly total traffic demand is between 2000 veh. and 2500 veh., the demand is considered as low. If the hourly total traffic demand is between 2500 veh. and 3000 veh., it can be said that the demand is moderate. When the hourly total traffic demand is between 3000 veh. and 3500 veh., the demand is regarded as high. Finally, if the hourly total traffic demand is between 3500 veh. and 4000 veh., it can be said that the demand is very high. Thus, the performance of the VAM4 model developed for a four-leg signalized intersection could be evaluated for the cases of low, moderate, high and very high traffic demands, separately.

As it is known, in the VISSIM simulation program, the time intervals when the fluctuations in traffic demands occur and the quantities (proportions) of the fluctuations in these time intervals can be determined by the users. Thus, as a result of the short or long-term fluctuations that may be seen due to the reasons such as concerts, sports events, traffic accidents, adverse weather conditions, changes that occur in the performance of intersection can be analyzed in VISSIM. As mentioned above, in this study, it is aimed to examine the performance and effectiveness of the developed VAM4 in case of fluctuations with variable quantities in variable time intervals for traffic demands at different intersection approaches. For this aim, sample cases related to
the quantities of the fluctuations in traffic demands and the time intervals when the fluctuations occur in traffic demands are created for each traffic scenario. The created sample cases are presented in Fig. 6. In Fig. 6, for each traffic scenario, the time intervals when the fluctuations occur in traffic demands at different intersection approaches and the quantities of fluctuations during 60 minutes of the analysis period can be seen in detail. As an example; for Scenario 6, it is assumed that the fluctuations in traffic demands occur twice at the West and North approaches and only once at the East approach during 60 minutes of the analysis period. Besides, for Scenario 6, the following matters can be said about the quantities of fluctuations, and the time intervals when the fluctuations occur in traffic demands:

- It is assumed that approximately 33% of the vehicles at the west approach arrive to the intersection between the 20th and 33rd minutes, and approximately 28% arrive to the intersection between the 47th and 60th minutes.
- It is assumed that approximately 22% of the vehicles at the north approach arrive to the intersection between the 20th and 27th minutes and 22% arrive to the intersection between the 40th and 47th minutes.
- It is assumed that approximately 50% of the vehicles at the east approach arrive to the intersection between the 13th and 33rd minutes.

3. Analysis

Webster, Highway Capacity Manual (HCM) and Akcelik methods are the most used delay calculation approaches for predicting the average vehicle delay which occurs at signalized intersections [20, 40, 42]. While the Webster and HCM methods are based on the phase-related design, the Akcelik method is based on the movement-related design. In the scope of the study, when the modeled intersection was examined carefully, it was seen that all lanes at intersection approaches were designed based on the traffic flows’ movement directions. Thus, the Akcelik delay model which aims movement-related design was used for optimizing the signal timings.
in this study. According to the Akcelik method, the approximate value of total delay for a movement can be calculated by using Eq. (2) [42]:

\[
D = \frac{qC \times (1-u)^2}{2 \times (1-y)} + N_o x \tag{2}
\]

Where:

- \( D \): Total Delay (sec),
- \( q \): Flow (veh/sec),
- \( C \): Cycle time (sec),
- \( qC \): Average number of arrivals in vehicles per cycle,
- \( u \): Green time ratio (\( g/C \)),
- \( y \): Flow ratio (\( q/s \)),
- \( N_o \): Average overflow queue (veh)
- \( x \): Degree of saturation (\( q/Q \)) (veh).

At a signalized intersection approach, to predict the overflow queues for both undersaturated (\( x < 1 \)) and oversaturated (\( x > 1 \)) conditions, the following Eq. (3) which was developed by Akcelik can be used:

\[
N_o = \begin{cases} 
\frac{QT_f}{4} \left( z + \sqrt{z^2 + \frac{12(x-x_o)}{QT_f}} \right), & x \geq x_o \\
0, & x < x_o 
\end{cases} \tag{3}
\]

Where:

- \( N_o \): Average overflow queue (veh),
- \( Q \): Capacity in vehicle in per hour (veh/hr),
- \( T_f \): Flow period,
- \( QT_f \): Maximum number of vehicles which can be discharged during the interval \( T_f \).
\( x \): Degree of saturation \((q/Q)\),

\( z \): \( x-1\),

\( x_o \): The degree of saturation below which the average overflow queue is approximately zero.

The degree of saturation below which the average overflow queue is approximately zero \((x_o)\) can be calculated by Eq. (4):

\[ x_o = 0.67 + \frac{sg}{600} \] (4)

Where;

\( s \): Saturation flow (veh/sec),

\( g \): Effective green time (sec).

Average vehicle delay can be calculated by Eq. (5):

\[ d = \frac{D}{q} \] (5)

Where;

\( D \): Total Delay (sec),

\( q \): Flow (veh).

With the analysis studies, the determination of the effectiveness of VAM4 which was created in VISVAP was aimed. Therefore, firstly, it was assumed that the modeled signalized intersection was managed with an optimum fixed-time signal control system and it was operated with four phases. The signal timings were optimized by minimizing of average vehicle delay at the intersection. The objective function \( f(x) \), the decision variables and the set of constraints for the signal timing optimization problem are shown in Table 3.

As can be seen from Table 3, for the modeled intersection, the optimization problem consists of 1 objective function, 4 decision variables and 18 constraints. When the previous studies were
investigated carefully, it was seen that green signal timings for each phase should be within a certain range [43]. Thus, green signal timings were constrained between 7 seconds and 45 seconds considering the previous studies in the literature [43, 44]. In addition to this, the degree of saturation for each lane was constrained with a maximum of 1.2. Differential Evolution (DE) algorithm which is a powerful and simple meta-heuristic optimization algorithm is preferred to solve the signal timing optimization problem [45-49]. A signal timing optimization program was developed in MATLAB for this purpose. For created traffic scenarios, obtained signal timings and average vehicle delays are given in Table 4 in detail.

Because all of the scenarios which are created in the scope of the study are analyzed in VISSIM, Akcelik average vehicle delay results and average vehicle delay results obtained from VISSIM should be similar to each other for the same scenarios. Rational and suitable evaluation cannot be possible, otherwise. Due to differences between the results, firstly, an evaluation area was created considering the points which are 100 meters away from all approaches stop lines. Then, driving behaviors and the safety factors in VISSIM software were revised considering the results which are obtained from Akcelik delay equation. Thus, average vehicle delay results for each scenario are likened to each other. For created scenarios, the comparison of the average vehicle delay values is shown in Table 5.

As can be seen from Table 5, for created scenarios, the differences are below about 5% in generally. The accuracy of the simulation is determined by using the Sum of Squared Error value [50]. This value is calculated as 33.56. These results show that the VISSIM can provide similar average delay values with the Akcelik delay equation and can be used for analysis studies. The average vehicle delay values which are obtained from VISSIM and by using the Akcelik delay equation are shown in Fig. 7(a). The percentage differences between Akcelik and VISSIM are also shown in Fig. 7(b).
After the completion of the revision process in the VISSIM simulation program, to determine the effectiveness of VAM4, created vehicle actuated signal management control algorithm is tested on all traffic scenarios. At this stage, firstly, the most proper placements of detectors for each scenario were determined with the trial and error method. For Scenario 5 and Scenario 17, average vehicle delay values which were obtained depending on the detector placement are as shown in Fig. 8.

As seen in Fig. 8, a comprehensive study was carried out to determine the most proper placement of the detectors. Because the distance between detectors and stop lines is varied from 15 meters to 45 meters, detectors must be placed within this range at all intersection approaches. At this stage, created scenarios were analyzed considering different distances between detectors and stop lines. At the end of the analyses, the distance which corresponds to the minimum average vehicle delay value was considered as the most proper distance between detectors and stop lines. Near optimum detector placements (distances) which are obtained from the analysis for all created scenarios are presented in Table 6.

As can be seen from Table 6, when the total traffic volume at the intersection increases, the distance between detectors and stop line also shows an increase. While the total traffic demand at the intersection is low or moderate, near optimum distances between detectors and stop line vary in the range of 15 meters and 25 meters in generally. Besides, while the total traffic demand at the intersection is high or very high, near optimum distances between detectors and stop line vary in the range of between 30 meters and 45 meters.

4. Results

After the determination of the most proper placements of detectors for all created scenarios, two different types of traffic management approaches were evaluated separately considering whether the fluctuations in traffic demands exist or not for each scenario. These four different cases which are taken into account for the analysis can be summarized as follows:
• Optimum Fixed-Time Management / Fluctuations do not exist in traffic demands (OFTM / Not-Fluctuated): In this case, fluctuations do not exist in traffic demands. Movement-based traffic volumes and optimum signal timings are transferred to VISSIM. At the end of the analysis, average vehicle delay values for each scenario were obtained. Obtained results can be seen in Table 5.

• Vehicle-Actuated Management / Fluctuations do not exist in traffic demands (VAM4 / Not-Fluctuated): In the second case, fluctuations do not exist in traffic demands. All scenarios were analyzed considering the VAM4 model which is created in VISVAP.

• Optimum Fixed-Time Management / Fluctuations exist in traffic demands (OFTM / Fluctuated): In the third case, fluctuations exist in traffic demands for each scenario (in Fig. 6). The only difference of this case from the first case is existing fluctuations in traffic demands. Movement-based traffic volumes and total traffic volumes are the same for the first case and third case. In this case, the investigation of the success of optimum fixed-time management (OFTM) was aimed for fluctuated traffic demands.

• Vehicle-Actuated Management / Fluctuations exist in traffic demands (VAM4 / Fluctuated): In the fourth case, fluctuations exist in traffic demands for each scenario (in Fig. 6). The only difference of this case from the second case is existing fluctuations in demands. Movement-based traffic volumes and total traffic volumes are the same for the second case and fourth case. In this case, the investigation of the success of VAM4 was aimed for fluctuated traffic demands.

At the end of the analysis which is made considering four different cases described above, average vehicle delay-fuel consumption and exhaust emission (CO+NOx) values were obtained for each scenario. Obtained average vehicle delay, fuel consumption and exhaust emission values are shown in Fig. 9(a), Fig. 9(b) and Fig. 9(c), respectively.
As seen in Fig. 9, especially in case of fluctuated traffic demands, the effectiveness of the optimum fixed-time signal management system decreases dramatically. Besides, the average vehicle delays, fuel consumptions and exhaust emissions can be significantly reduced by implementing vehicle-actuated signal control at the intersection. For Scenario 3, in the time interval when the fluctuations occur in traffic demands, sample intersection views which are obtained from VISSIM in case of implementing the optimum fixed-time management or/and vehicle actuated management are presented in Fig. 10.

In Fig. 10, for Scenario 3, in the time interval when the fluctuations occur in the traffic demands, sample intersection views in case of implementing the different types of management approaches are shown. As shown in Fig. 6, the fluctuations in the traffic demands for Scenario 3 occur in between the 12th and 36th minutes. Therefore, simulation views are taken in the 25th minutes (1475 sec.) of the simulation. As can be seen from Fig. 10, in case of implementing of optimum-fixed time traffic management at modeled signalized intersection, excessive queuing (queues) are seen at the intersection approaches where fluctuations in traffic demands occur (Fig. 10 (a)). When Fig. 10(b) is examined carefully, in case of implementing vehicle-actuated management instead of optimum fixed-time management at the modeled intersection, queuing (queues) at intersection approaches where fluctuations in traffic demands occur can be significantly reduced. Thus, the performance of the intersection can also be improved in this way.

When the fluctuations do not exist in traffic demands, although vehicle-actuated signal management systems have more advantages than optimum fixed-time signal management systems, obtained success rates become low. For the sample scenario (Scenario 3) which is shown in Fig. 10, sample intersection views in the same time intervals in case of implementing the optimum fixed-time management and vehicle-actuated management are presented in Fig. 11.
In Fig. 11, for Scenario 3, when the fluctuations do not exist in the traffic demands, sample intersection views in the same time interval in case of implementing the different types of management approaches are shown. As can be seen from Fig. 11, when the fluctuations do not exist in traffic demands, excessive queuing is not seen at the intersection approaches in case of implementing of both optimum fixed-time management and vehicle-actuated management. But, more queuing occurs in case of implementing of optimum fixed-time management (in Fig. 11(a)) than in case of implementing of vehicle-actuated management (in Fig. 11(b)). When Fig. 11 is investigated carefully, the queuing at intersection approaches can be reduced in case of implementing of vehicle-actuated management. As a result, it was concluded that simulation views are compatible with obtained performance results. The simulation views support and verify the obtained results.

For created traffic scenarios, reduction rates (success rates) were evaluated separately considering whether fluctuations in traffic demands exist or not. In case of implementing of vehicle-actuated management instead of optimum fixed-time management, obtained reduction rates (success rates) for each performance criteria are presented in Fig. 12.

When Fig. 12 is examined carefully, the obtained results can be summarized as follows:

- For the fluctuated traffic demands, when the total traffic demand at the intersection is low, moderate or high, average vehicle delay can be reduced between about 20% and 60%, fuel consumption and exhaust emission can be reduced between about 15% and 40%.

- For the fluctuated traffic demands, when the total traffic demand at the intersection is very high, the reduction rates for average vehicle delay decrease down to 15%, reduction rates for fuel consumption and exhaust emission decrease down to 10%. These results show that the vehicle-actuated management systems are not efficient systems for specified traffic demand conditions.
For not-fluctuated traffic demands, when the total traffic demand at the intersection is low, moderate or high, average vehicle delay can be reduced between about 5% and 20%, fuel consumption and exhaust emission can be reduced between about 3% and 9%.

For the not-fluctuated traffic demands, when the total traffic demand at the intersection is very high, it can be seen that the reduction rates for average vehicle delay increase up to 30%, reduction rates for fuel consumption and exhaust emission increase up to 18%.

These results show that the vehicle-actuated management systems are very effective systems for specified traffic demand conditions.

When the cases of whether the fluctuations in traffic demands exist or not are investigated separately, the obtained results are presented in Table 7 in detail.

As seen in Table 7, for the created scenarios:

- For not-fluctuated traffic demands, in case of implementing of VAM4 instead of OFTM, the obtained reduction rates for average vehicle delay are about 4.5% of minimum, 29.2% of maximum and 14.2% of average. For the fuel consumption and exhaust emission, these rates are about 2.8% of minimum, 17.7% of maximum and 7.3% of average.

- For fluctuated traffic demands, in case of implementing of VAM4 instead of OFTM, the obtained reduction rates for average vehicle delay are about 15.6% of minimum, 58.6% of maximum and 32.5% of average. For the fuel consumption and exhaust emission, these rates are about 9.6% of minimum, 38.4% of maximum and 21.8% of average.

As can be seen from the obtained results, developed VAM4 is more effective in case of fluctuated traffic demands. For fluctuated traffic demands, reduction rates for all performance criteria increase about three times compared to not-fluctuated traffic demands in case of implementing of VAM4 instead of OFTM.

5. Discussion and Conclusion
In this study, a VAM4 that can adapt to fluctuations in traffic demands was developed. The effectiveness of the developed system was tested considering whether the fluctuations in traffic demands exist or not. Obtained results from this study can be summarized as follows:

- When the fluctuations in traffic demands did not exist, lower average vehicle delay, fuel consumption and exhaust emission values were obtained with VAM4 compared to OFTM. When Fig. 12 is examined carefully, especially in case of traffic demand is very high at the intersection, reduction rates for all performance criteria increase. One of the main reasons for this situation is that the constraint of maximum green time of 45 seconds for OFTM is lower than the constraint of maximum green time of 60 seconds for VAM4. The increase of maximum green time length for VAM4 enabled to lower waste of time for vehicles at intersection approaches where the traffic demand is very high. Thus, average vehicle delays, fuel consumption and exhaust emissions were significantly reduced.

- When the fluctuations in traffic demands existed, the effectiveness of OFTM decreased and average vehicle delay, fuel consumptions and exhaust emissions significantly increased. Average vehicle delay, fuel consumption and exhaust emissions were significantly reduced with the implementation of VAM4, especially in case of traffic demand at the intersection was low, moderate and high. In case of traffic demand at the intersection was very high, reduction rates decreased. This situation has arisen from the increase of the usage of maximum green time by different intersection approaches. It was determined that the density of the usage of maximum green time was an important factor that encourages the increase of delay, fuel consumption and exhaust emission.

- For created scenarios, when the fluctuations in traffic demands did not exist, in case of implementation of VAM4 instead of OFTM, the average vehicle delay was reduced about the average of 14.2%. Besides, fuel consumption and exhaust emissions were reduced by
about an average of 7.3%. When the fluctuations in traffic demand existed, these rates increased 2.5-3 times.

When the fluctuations in traffic demands did not exist, in case of implementation of VAM4 instead of OFTM, the level of service of only five scenarios was improved. When the fluctuations in traffic demands existed, the level of service of fourteen scenarios was improved.

All obtained results show that VAM4 can effectively adapt to fluctuations in traffic demands. In addition to this, it is thought that the effectiveness and the performance of VAM4 can be more improved with the different computational approaches.

Acknowledgments

The authors of this article would like to thank to PTV (Planung Transport Verkehr AG) for providing of VISSIM simulation software.

Funding

The authors of this article received no financial support for the research, authorship, and publication of this article.

Availability of Data and Materials

The data used to support the findings of this study are available from the corresponding author upon request.

6. References


Figure and Table Captions

List of Figures

Fig. 1 The relation between average vehicle delay and the placement of detectors

Fig. 2 Vehicle-actuated management application at a four-leg signalized intersection in VISSIM

Fig. 3 The flow chart of vehicle-actuated control logic created with VAP

Fig. 4 The change rates of movement direction-based traffic volumes for created scenarios

Fig. 5 Hourly total traffic volumes for all traffic scenarios

Fig. 6 Time intervals and quantities of fluctuations in traffic demands for each traffic scenario

Fig. 7 For created scenarios (a) the comparison of average vehicle delay values and (b) the percentage (%) differences of average vehicle delay values.

Fig. 8 The effect of detector placements on average vehicle delay (a) for Scenario 5 and (b) Scenario 17.

Fig. 9 At the end of the analysis, for created scenarios (a) obtained average vehicle delay results; (b) obtained fuel consumption results; (c) obtained exhaust emission results.

Fig. 10 For the Scenario 3, in time interval when the fluctuations occur in traffic demands, sample intersection view in case of implementing (a) the optimum-fixed time management (OFTM) (b) vehicle-actuated management (VAM4)

Fig. 11 For Scenario 3, when the fluctuations do not exist in traffic demands, sample intersection view in time same time interval in case of implementing (a) the optimum fixed-time management (OFTM), (b) vehicle-actuated management (VAM4)

Fig. 12 Obtained reduction rates in case of implementing of vehicle-actuated management instead of optimum fixed-time management for each performance criteria

List of Tables

Table 1 Minimum green times for different detector locations

Table 2 Reference demand matrix created for analysis studies
Table 3 The objective function, the decision variables and the set of constraints for the signal timing optimization problem

Table 4 Optimum signal timings and average vehicle delays for created traffic scenarios

Table 5 Scenario-based comparison of average vehicle delay values (sec/veh)

Table 6 Optimum detector placements for created scenarios

Table 7 For the cases of fluctuated and not-fluctuated traffic demands, obtained reduction rates (success rates)
Fig. 1 The relation between average vehicle delay and the placement of detectors

Fig. 2 Vehicle-actuated management application at a four-leg signalized intersection in VISSIM
Fig. 3 The flow chart of vehicle-actuated control logic created with VAP
Fig. 4 The change rates of movement direction-based traffic volumes for created scenarios

Fig. 5 Hourly total traffic volumes for all traffic scenarios
<table>
<thead>
<tr>
<th>Scenario No</th>
<th>Intersection Approach</th>
<th>Analysis Period (60 minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>West</td>
<td>44%</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>West</td>
<td>33%</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>West</td>
<td>60%</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>West</td>
<td>56%</td>
</tr>
<tr>
<td>Scenario 5</td>
<td>North</td>
<td>33%</td>
</tr>
<tr>
<td>Scenario 6</td>
<td>West</td>
<td>22%</td>
</tr>
<tr>
<td>Scenario 7</td>
<td>West</td>
<td>50%</td>
</tr>
<tr>
<td>Scenario 8</td>
<td>North</td>
<td>38%</td>
</tr>
<tr>
<td>Scenario 9</td>
<td>West</td>
<td>50%</td>
</tr>
<tr>
<td>Scenario 10</td>
<td>West</td>
<td>50%</td>
</tr>
<tr>
<td>Scenario 11</td>
<td>North</td>
<td>67%</td>
</tr>
<tr>
<td>Scenario 12</td>
<td>West</td>
<td>75%</td>
</tr>
<tr>
<td>Scenario 13</td>
<td>West</td>
<td>42%</td>
</tr>
<tr>
<td>Scenario 14</td>
<td>North</td>
<td>33%</td>
</tr>
<tr>
<td>Scenario 15</td>
<td>West</td>
<td>67%</td>
</tr>
<tr>
<td>Scenario 16</td>
<td>West</td>
<td>33%</td>
</tr>
<tr>
<td>Scenario 17</td>
<td>North</td>
<td>38%</td>
</tr>
<tr>
<td>Scenario 18</td>
<td>North</td>
<td>50%</td>
</tr>
<tr>
<td>Scenario 19</td>
<td>North</td>
<td>44%</td>
</tr>
<tr>
<td>Scenario 20</td>
<td>East</td>
<td>50%</td>
</tr>
</tbody>
</table>

Fig. 6 Time intervals and quantities of fluctuations in traffic demands for each traffic scenario
Fig. 7 For created scenarios (a) the comparison of average vehicle delay values and (b) the percentage (%) differences of average vehicle delay values.

Fig. 8 The effect of detector placements on average vehicle delay for (a) Scenario 5 and (b) Scenario 17.
Average Vehicle Delay - Demand: Low

Average Vehicle Delay - Demand: Moderate

Average Vehicle Delay - Demand: High

Average Vehicle Delay - Demand: Very High

(a)
Fuel Consumption - Demand: Low

Fuel Consumption - Demand: Moderate

Fuel Consumption - Demand: High

Fuel Consumption - Demand: Very High

(b)
Fig. 9 At the end of the analysis, for created scenarios (a) obtained average vehicle delay results; (b) obtained fuel consumption results; (c) obtained exhaust emission results.
Fig. 10 For the Scenario 3, in time interval when the fluctuations occur in traffic demands, sample intersection view in case of implementing (a) the optimum fixed-time management (OFTM) (b) vehicle-actuated management (VAM4).

Fig. 11 For Scenario 3, when the fluctuations do not exist in traffic demands, sample intersection view in time same time interval in case of implementing (a) the optimum fixed-time management (OFTM), (b) vehicle-actuated management (VAM4).
Fig. 12 Obtained reduction rates in case of implementing of vehicle-actuated management instead of optimum fixed-time management for each performance criteria.
Table 1 Minimum green times for different detector locations

<table>
<thead>
<tr>
<th>Option</th>
<th>O-1</th>
<th>O-2</th>
<th>O-3</th>
<th>O-4</th>
<th>O-5</th>
<th>O-6</th>
<th>O-7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance between detectors and stop line (m)</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>45</td>
</tr>
<tr>
<td>Minimum green times (sec)</td>
<td>5</td>
<td>7</td>
<td>9</td>
<td>11</td>
<td>13</td>
<td>15</td>
<td>17</td>
</tr>
</tbody>
</table>

*O-1, 2, ...7 shows the options of distance between detector and stop line.

Table 2 Reference demand matrix created for analysis studies.

<table>
<thead>
<tr>
<th>Volumes (veh/hr)</th>
<th>Flow Rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intersection Approach (O-D)</td>
<td>West</td>
</tr>
<tr>
<td>West</td>
<td>-</td>
</tr>
<tr>
<td>North</td>
<td>150</td>
</tr>
<tr>
<td>East</td>
<td>600</td>
</tr>
<tr>
<td>South</td>
<td>200</td>
</tr>
</tbody>
</table>

West Intersection Approach: 1000 veh/hr
North Intersection Approach: 400 veh/hr
East Intersection Approach: 850 veh/hr
South Intersection: 650 veh/hr

Total Traffic Flow at Intersection: 2900

Table 3 The objective function, the decision variables and the set of constraints for the signal timing optimization problem.

<table>
<thead>
<tr>
<th>Objective Function ( f(x) )</th>
<th>( n = 1, 2, \ldots, 14 ) (( n = ) Total number of lanes at intersection)</th>
<th>( f(x) = \min \left( d = \sum_{i=1}^{n} D_i / \sum_{i=1}^{n} q_i \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Decision Variables ( i = 1, 2, \ldots, 4 ) (( i = ) Phase no)</td>
<td>( g_i ) : Green signal timing (green split) for phase ( i ) (sec)</td>
<td>( 7 \leq g_i \leq 45 )</td>
</tr>
</tbody>
</table>

Constraints for minimum and maximum green signal timing (\( g_i \))

Constraints for degree of saturation (\( x_n \))

\( n = 1, 2, \ldots, 14 \) (\( n = \) Total number of lanes at intersection)

\( 0 \leq x_n \leq 1.2 \)
Table 4 Optimum signal timings and average vehicle delays for created traffic scenarios

<table>
<thead>
<tr>
<th>Demand</th>
<th>Scenario No</th>
<th>Phase I (sec)</th>
<th>Phase II (sec)</th>
<th>Phase III (sec)</th>
<th>Phase IV (sec)</th>
<th>Cycle Time (sec)</th>
<th>Average Delay (sec/veh)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>1</td>
<td>10</td>
<td>9</td>
<td>9</td>
<td>12</td>
<td>60</td>
<td>25.00</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>9</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>53</td>
<td>20.82</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>19</td>
<td>8</td>
<td>16</td>
<td>13</td>
<td>76</td>
<td>33.25</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>13</td>
<td>9</td>
<td>11</td>
<td>11</td>
<td>64</td>
<td>26.89</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>12</td>
<td>7</td>
<td>10</td>
<td>9</td>
<td>58</td>
<td>23.60</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>14</td>
<td>8</td>
<td>12</td>
<td>11</td>
<td>65</td>
<td>28.32</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>19</td>
<td>8</td>
<td>16</td>
<td>13</td>
<td>76</td>
<td>32.84</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>19</td>
<td>9</td>
<td>17</td>
<td>13</td>
<td>78</td>
<td>33.30</td>
</tr>
<tr>
<td></td>
<td>9 (R)</td>
<td>19</td>
<td>9</td>
<td>16</td>
<td>13</td>
<td>77</td>
<td>33.20</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>18</td>
<td>9</td>
<td>16</td>
<td>13</td>
<td>76</td>
<td>34.51</td>
</tr>
<tr>
<td>High</td>
<td>11</td>
<td>19</td>
<td>9</td>
<td>17</td>
<td>14</td>
<td>17</td>
<td>79.00</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>25</td>
<td>11</td>
<td>22</td>
<td>17</td>
<td>95</td>
<td>43.82</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>24</td>
<td>11</td>
<td>21</td>
<td>17</td>
<td>93</td>
<td>44.63</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>25</td>
<td>12</td>
<td>22</td>
<td>18</td>
<td>97</td>
<td>45.55</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>31</td>
<td>14</td>
<td>27</td>
<td>22</td>
<td>114</td>
<td>55.59</td>
</tr>
<tr>
<td>Very High</td>
<td>16</td>
<td>45</td>
<td>19</td>
<td>39</td>
<td>31</td>
<td>154</td>
<td>78.83</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>45</td>
<td>20</td>
<td>39</td>
<td>31</td>
<td>155</td>
<td>82.37</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>31</td>
<td>15</td>
<td>27</td>
<td>22</td>
<td>115</td>
<td>57.56</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>45</td>
<td>20</td>
<td>39</td>
<td>31</td>
<td>155</td>
<td>81.21</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>45</td>
<td>20</td>
<td>39</td>
<td>31</td>
<td>155</td>
<td>80.26</td>
</tr>
</tbody>
</table>

Phase I: West ; Phase II: North ; Phase III: East ; Phase IV: South
Inter Green Time: 5 sec.
(R): Reference Demand

Table 5 Scenario-based comparison of average vehicle delay values (sec/veh)

<table>
<thead>
<tr>
<th>Demand</th>
<th>Scenario No</th>
<th>Scenario No</th>
<th>Scenario No</th>
<th>Scenario No</th>
<th>Scenario No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>25.00</td>
<td>20.82</td>
<td>33.25</td>
<td>26.89</td>
<td>23.60</td>
</tr>
<tr>
<td>Moderate</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>28.32</td>
<td>32.84</td>
<td>33.30</td>
<td>33.20</td>
<td>34.51</td>
</tr>
<tr>
<td>High</td>
<td>11</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>27.52</td>
<td>33.38</td>
<td>31.74</td>
<td>32.23</td>
<td>34.94</td>
</tr>
<tr>
<td>Very High</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>78.83</td>
<td>82.37</td>
<td>57.56</td>
<td>81.21</td>
<td>80.26</td>
</tr>
</tbody>
</table>

Table 6 Optimum detector placements for created scenarios

<table>
<thead>
<tr>
<th>Scenario No</th>
<th>Distance between Detectors and Stop Line (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>West Approach</td>
</tr>
<tr>
<td>1</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>15</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
</tr>
<tr>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>9</td>
<td>25</td>
</tr>
<tr>
<td>10</td>
<td>20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Scenario No</th>
<th>Distance between Detectors and Stop Line (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>West Approach</td>
</tr>
<tr>
<td>11</td>
<td>20</td>
</tr>
<tr>
<td>12</td>
<td>30</td>
</tr>
<tr>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>14</td>
<td>35</td>
</tr>
<tr>
<td>15</td>
<td>45</td>
</tr>
<tr>
<td>16</td>
<td>45</td>
</tr>
<tr>
<td>17</td>
<td>45</td>
</tr>
<tr>
<td>18</td>
<td>45</td>
</tr>
<tr>
<td>19</td>
<td>45</td>
</tr>
<tr>
<td>20</td>
<td>45</td>
</tr>
</tbody>
</table>
Table 7 For the cases of fluctuated and not-fluctuated traffic demands, obtained reduction rates (success rates)

<table>
<thead>
<tr>
<th>Performance Criteria</th>
<th>Demand: Not - Fluctuated</th>
<th>Demand: Fluctuated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Average vehicle delay</td>
<td>4.51</td>
<td>29.16</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>2.80</td>
<td>17.68</td>
</tr>
<tr>
<td>Exhaust Emission</td>
<td>2.79</td>
<td>17.68</td>
</tr>
</tbody>
</table>

A Brief Technical Biography of Authors

Ziya CAKICI received B.Sc. degree from Civil Engineering Department of Celal Bayar University, Manisa, Turkey in 2010. He received the M.Sc. and Ph.D. degrees in transportation engineering from Graduate School of Natural and Applied Sciences at Pamukkale University (Turkey), in 2014 and 2020, respectively. He has been working as Research Assistant in Department of Civil Engineering at Bayburt University. He studies in the fields of traffic management (especially Intelligent Transportation Systems), traffic and transportation planning and traffic optimization. He has published several papers in different journals and international conference proceedings.

Yetis Sazi MURAT received B.Sc. degree from Civil Engineering Department of Dokuz Eylul University, Izmir, Turkey in 1992. He has received M.Sc. degree from Pamukkale University in 1996 and Ph.D. degree from Istanbul Technical University in 2001. He has been working as Professor in Department of Civil Engineering at Pamukkale University. He worked as visiting scholar at Virginia Tech. University, Falls Church, USA in 2006 (6 months) and University of Nevada, Reno, USA in 2017. He has published many technical-scientific research papers and conducted many projects on different subjects of Transportation Engineering. His main research interests include fuzzy logic and neural network based traffic management approaches. He studies in the fields of traffic management (especially Intelligent Transportation Systems), traffic and transportation planning, traffic accidents and etc.

Metin Mutlu AYDIN received B.Sc. degree from Civil Engineering Department of Dokuz Eylul University, Izmir, Turkey in 2010. He received the M.Sc. degree from Dokuz Eylul University in 2012 and Ph.D. degree from Akdeniz University, Antalya, Turkey in 2017. He
has been working Associate Professor in Department of Civil Engineering at Ondokuz Mayıs University. He studies in the fields of traffic and transportation engineering. His research interests include modelling of the lane discipline, optimizing of the intersection geometry, simulating of driver behaviours, in generally. He has published many research papers in different journals and international conference proceedings.