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Analysis of Travel Speed on 4-Lane Signalized Arterials

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Abstract

Signalized intersections are the major causes of delay to vehicular traffic along signalized arterials. Although a lot of research has been conducted about the optimization of the performance of signalized intersections, it is mostly applicable to the operations stage and involves changing signal phasing schemes. However, due to cost of roadways construction and the additional improvements after construction, the assessment of the potential performance of the roadway is very important even at the planning stage so that the plans do not call for some unfavorable design features along the arterial. The lack of performance oriented planning often leads to poor performance, such as very low travel speeds. In this work, we investigate the travel speed along a hypothesized 4-lane signalized arterial, and examine how it changes due to the signalized intersection density, cycle length, mean desired speed, and the function of the intersection. Results showed that cycle lengths up to 120 seconds were sufficient to yield high travel speed. At higher intersection densities, lower travel speeds will be achieved regardless of the mean desired speed. The function of the signalized intersection was also significant, showing the possibility of maintaining the traffic function of an arterial if it were crossed by a lower level road in the hierarchy. The importance of performance evaluation at the planning stage was further emphasized.

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Keywords: Travel speed ; Delay; Signalized intersections; Simulation analysis; Planning

1. Background and objectives

Delay due to signalized intersections is a major factor affecting the quality of service on signalized arterials. Many studies have been conducted about improving the performance of signalized intersections. Measures such as adaptive signal control, where traffic signal plans are varied with changing demand levels in order to minimize delay, have been proven effective countermeasures.

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Due to the costs of installing and operating real-time control systems, fixed –time control continues to be widely applied along signalized arterials. This makes the design of the signal settings more challenging because there is a need to forecast how the arterial performance might change if any of the historical information used in the design process changes.

When roads are designed solely based on the link capacity, without the consideration of the expected performance of the roadway, roads can be constructed with unfavorable features such as densely spaced signalized intersections, unnecessarily long cycle lengths, as well as improper offset settings. This lack of performance-based planning/design leads to the poor performance of the roads as witnessed by travel speeds being much lower than the speed limit.

In this research, by using microsimulation analysis software VISSIM on hypothetical arterials, we focus on the impacts of signalized intersections and clarify the mechanisms by which the travel speed falls below the posted speed limit. For simplicity, *only simultaneous offsets* are considered, *traffic is composed of 100% passenger cars* and there are *no pedestrians*. In particular, we investigate the relevance of the relationship between link length, speed, and cycle length towards delay minimization. The impacts of cycle length, mean desired speed, signalized intersection density, and function of the intersection on travel speed are also investigated in turn. Finally, we make suggestions towards improving the roadway performance by additional considerations during the planning/design phase of the roadway.

2. Literature review

Papageorgiou et al. (2003) undertook a comprehensive review of road traffic control strategies, which they classified into fixed time strategies that are based on historical demand and turning ratios; traffic responsive strategies in which real-time measurements are used to calculate suitable signal settings; isolated strategies and coordinated strategies that can consider an entire network of several intersections. They also discussed the shortcomings of fixed-time strategies, which included the variations in demand by time of day, and between different days, among others.

Fitzpatrick et al. (2003) investigated the roadway features that influence drivers' speed selection to enable design of roadways to better influence performance of drivers. Speed and site characteristics data from 79 locations was used to develop models for predicting the 85th percentile operating speed. Speed limit was found to be significant at the 95% level, while the access density was significant at the 80% level. Other factors that showed some influence on the 85th percentile operating speed were median type, parking along the street, and pedestrian activity level. Cluster analysis also showed that more signals per mile were associated with clusters with low operating speeds.

Lum et al. (1998) developed travel time-density models for radial and ring arterial roads in Singapore. In addition to calibrating the traditional speed-density and speed-flow models, they also formulated and calibrated a single arterial road model that included signalized intersection delay and frequency of intersections as parameters. From the field data collected, a speed-flow model was developed from the travel time-density relationship, and parameter values of minimum delay and number of intersections per kilometer were specified for ring arterials, radial arterials, and all arterial roads. Because of the ease of estimation of intersection density, the authors recommended that the actual values be used for planning purposes.

Tarko et al. (2006) developed a simple model for predicting travel speed on urban arterial streets. By using the delay formula used in the Highway Capacity Manual, they proposed and calibrated using microsimulation software CORSIM, an equation for travel speed estimation using the following inputs; one-way volumes, number of through-lanes, distance between intersections, and the cruise speed, which could be approximated to or just below the speed limit. Control delay was incorporated into the model by replacing factors typically unknown at planning such as cycle length, with model parameters that can be calibrated. Their work showed the possibility of developing a model for predicting travel speed that can be used by planners.

Koshi (1989) hypothesized a coordinated two-way link with common cycle lengths, green split, and saturation flow rate, no platoon dispersion and green time equal to 50% of the cycle length. He showed that it was possible to minimize delay by setting the cycle length as one n-th of the roundtrip link travel time T, where n is a positive integer.

Chapter 30 of the Highway Capacity Manual (HCM) 6th Edition (2016) provides a planning-level methodology for estimating the travel speed along a coordinated street segment bound by signalized intersections, and identifies several factors that can be used in the analysis. These include the speed limit, cycle length, effective green to cycle length ratio, mid-segment volume, number of through lanes, and saturation flow rate.

3. Methodology

3.1. Simulation model calibration

The procedure proposed by Park and Qi (2005) was followed in the calibration of the simulation model in this study. Their procedure is comprised of the following steps:

- Simulation model setup – defining the study scope and purpose, site selection, defining measures of effectiveness (MoE), collecting field data and coding the network.
- Initial evaluation – run simulation analysis using all default parameters, and compare result to field data. If a close match is obtained, the default parameters are deemed appropriate for further analysis. Otherwise, the rest of the steps are followed.
- Initial calibration – identify calibration parameters that have a relevant impact on the simulation results and using Latin Hypercube Sampling (LHS), generate scenarios including the parameter range.
- Feasibility test – undertake multiple runs of each scenario identified from LHS, and check whether the field data are reasonably covered by the distribution of the simulation results. Plots of the performance measure against each parameter are used to determine the key parameters and their acceptable ranges.
- Parameter calibration by use of a genetic algorithm (GA) – the genetic algorithm is applied to determine the optimal parameter values through a fitness function. The fitness function is defined as the relative error of the simulated MoE from the field measured value, and the calibration objective is to minimize it.
- Model evaluation – multiple runs are separately conducted for default parameters and the GA-based parameter set for comparison. The distribution of the MoE is compared with field measures. Simulation animations are also observed to ensure reasonability of model, and if satisfactory, the model calibration is considered complete.

Average travel time was selected as the measure of effectiveness for the simulation model calibration. Video data was collected at the boundary intersections of a 600-meter long road section in Nagoya City, Japan. The section has a posted speed limit of 50 km/h, and the video data was collected during the morning peak hour as determined from the road and traffic census data. The location of the video cameras allowed a view of the traffic signal indication, the stop line, and the vehicle number plate. Traffic signal settings (cycle length, splits and offsets) were also collected during the same period at all intersections along the section.

A 15-minute period was selected during which the traffic volume and turning ratios were directly measured from the video data. By matching the plate numbers of the vehicles entering and exiting the study section, the travel times of about forty vehicles were obtained and averaged per travel direction. The objective of the calibration was then set to the reproduction of these field measured average travel times in the simulation model.

The network was built in VISSIM 7, and the signal settings and field measured traffic volume inputted into the simulation model. The initial evaluation showed that the simulation model underestimated the field measured travel time along the section. For the study section selected, only the “desired speed distribution (DSD)” was found to be the key parameter in the estimation of travel time. For the parameter range, the default DSD for a 50km/h speed limit (38 – 48 km/h), as well as other DSD ranges up to 40 – 60 km/h were used. Not only a uniform distribution, but also normally distributed DSD values were used.

The fitness function was simply defined as the percent relative error between field measured travel time TT_{field} and simulated travel time, TT_{sim} i.e. $100*(TT_{field} - TT_{sim})/\ TT_{field}$, and the objective was to minimize this value.

The best fitness was obtained with normally distributed DSD of 38 – 48 km/h. However, the observations of the simulation animations showed that some vehicles were stopping close to the intersection waiting for lane changing, which is not realistic. The “waiting time before diffusion” parameter was therefore reduced from the default 60 seconds to 1.0 seconds. The right-turning movements were also unrealistic as only very small gaps were used from the default settings, so the front and rear gaps at right-turning conflict areas with through traffic were set to 1.0 and 2.0 seconds, respectively.

With these changes, additional five simulations runs were conducted with a 15-minute warm-up time and 1-hour data collection time. The average travel time obtained from the simulation in both travel directions was a close match to the field observed values, with an average fitness value of 2.0%. The field measured travel times also fell within the distribution of the simulated travel times, therefore the simulation model was deemed to be adequately calibrated.

3.2. Scenario design

Fig. 1 shows intersection layouts of both key and non-key intersections along the hypothesized signalized arterial. The non-key intersections were those at which the minor approaches mostly served left- and right-turning traffic, and whose approach volume was 10% of the major intersection approaches.

(a) Key intersection (b) Non-key intersection

Fig. 1. Intersection layouts for this study's hypothetical networks

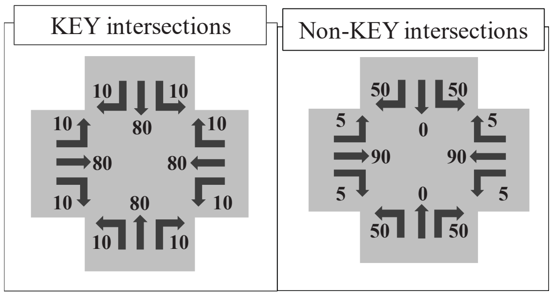


Fig. 2. Turning ratios

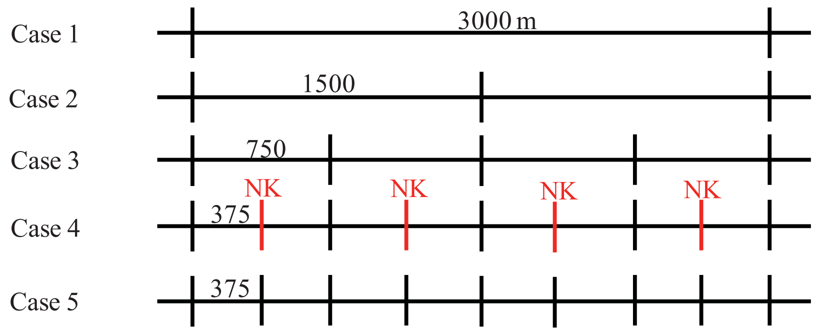


Fig. 3. Network layouts; in each case, adjacent intersections are equidistant

The turning ratios at each intersection were chosen so that the volume along the major direction (East-West) was kept constant along the entire section. For similar reasons, the input volumes at the minor approaches of non-key intersections were set to 10% of the approach volumes at key intersections. Turning ratios are shown in Fig. 2.

For each arterial, five different network layouts were considered. Starting with a 3-kilometer road section bound by two signalized intersections, an additional intersection is added equidistant from the intersections in the previous case to get the next one, until cases 4 and 5. The difference between cases 4 and 5 is that case 4 has four non-key intersections (labeled NK) in Fig. 3, which shows the schematic drawing of each of the five cases studied.

Next, traffic signal settings were designed for the hypothetical intersections following a modified version of the procedure in the Japan Manual on Traffic Signal Control, *Revised Edition* (2006) as shown in Fig. 4.

Key intersection approach volumes of 900 veh/h, and approach volumes on the minor streets of non-key intersections of 90 veh/h were used in the traffic signal setting procedure.

The vehicle approach speed assumed was 50 km/h, and based on the intersection layout shown in Fig. 1 (lane width = 3.0 meters, crosswalk width = 3.0 m), the clearance length was computed as no more than 30 meters. This information was used to set both the yellow change interval and the all red time to 3.0 seconds.

The minimum common cycle length determined by the procedure in Fig. 4 was found to be 80 seconds. In order to complete one of this work's objectives, it was important to investigate travel speed under various cycle lengths, so additional cycle length values were considered.

In Koshi's 1989 work, a two-way coordinated link was assumed. Additional assumptions were common cycle lengths, green split, and saturation flow rate, no platoon dispersion and green time equal to 50% of the cycle length. He showed that it was possible to minimize delay by setting the cycle length as one n-th of the roundtrip link travel time T, where n is a positive integer. Although all these simplifying assumptions are seldom achievable in the field, the theory provides a sound basis for delay minimization when designing traffic signal settings. In this work, we make use of this theory, and by defining a link of length L km, and assuming a *mean desired speed* V_{des} km/h, we can compute the delay minimizing cycle length C (s) for the link as shown in Eq. (1).

$$C = \frac{7200L}{nV_{des}} \tag{1}$$

To confirm the applicability of the theory to real world driving behavior (used for calibrating the hypothetical networks), four different mean desired speeds were assumed, and by applying Eq. (1) to the hypothesized networks, we obtained the values of cycle length that would minimize delay for each case. The results are shown in Table 1, and were kept in the range 80 – 160 seconds, and only to values common between at least two cases so that a comparison could be made. For cases 4 and 5, the cycle lengths from Eq. (1) were outside our range of consideration, but are shown with triangular brackets in Table 1.

Table 1. Delay minimizing cycle lengths (s) based on Eq. (1)

	Mean desired speed V_{des} (km/h)			
Case (Link length, km)	45	50	54	60
1 (3.0)	80, 120, 160	108	80, 100	90, 120
2 (1.5)	80, 120	108	100	90
3 (0.75)	120	108	100	90
4 (0.375)	<60>	<54>	<50>	<45>
5 (0.375)	<60>	<54>	<50>	<45>

From the results shown in Table 1, and considering the commonly used cycle lengths in Japanese cities like Nagoya, seven different cycle lengths were studied: 80, 90, 100, 108, 120, 150, and 160 seconds.

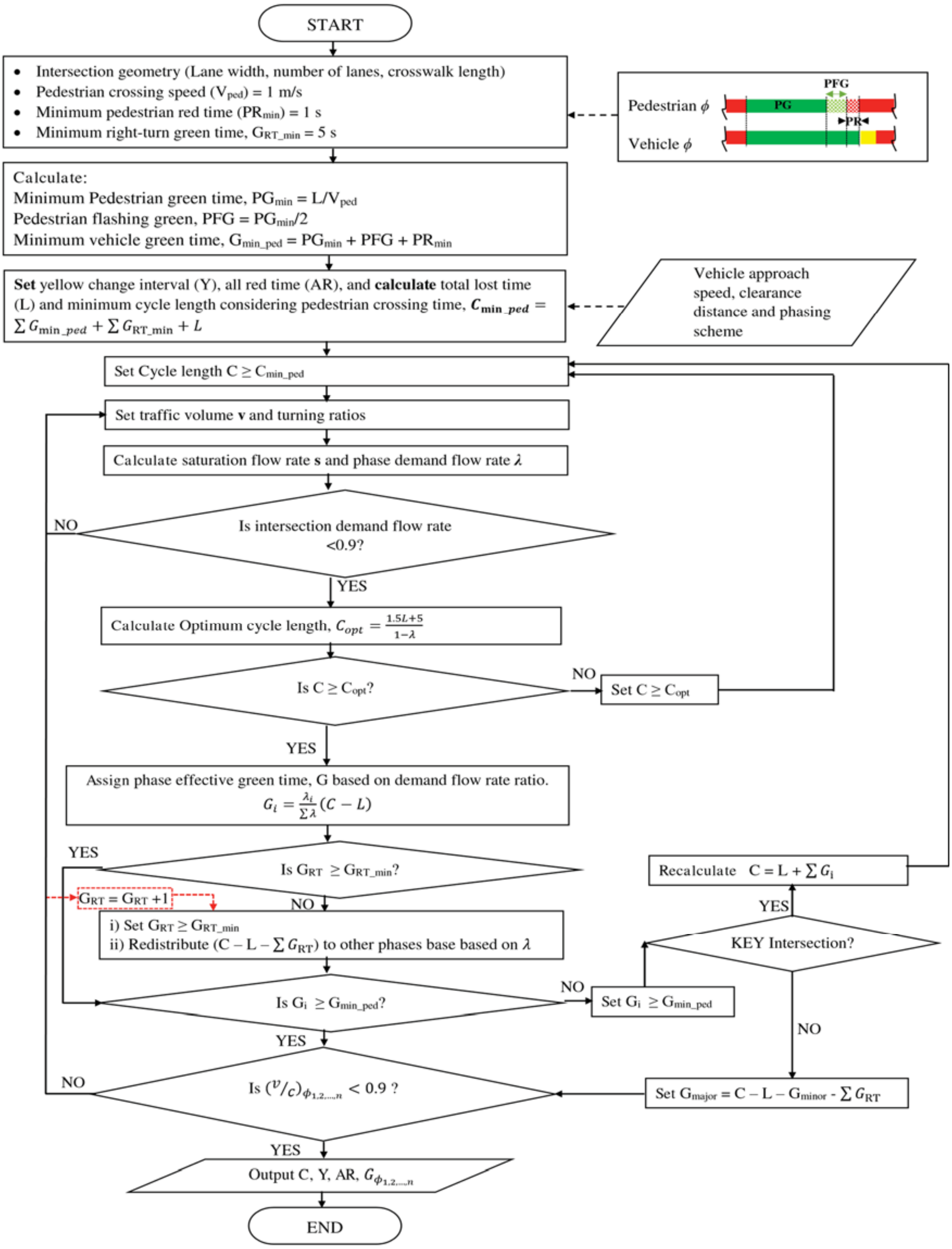
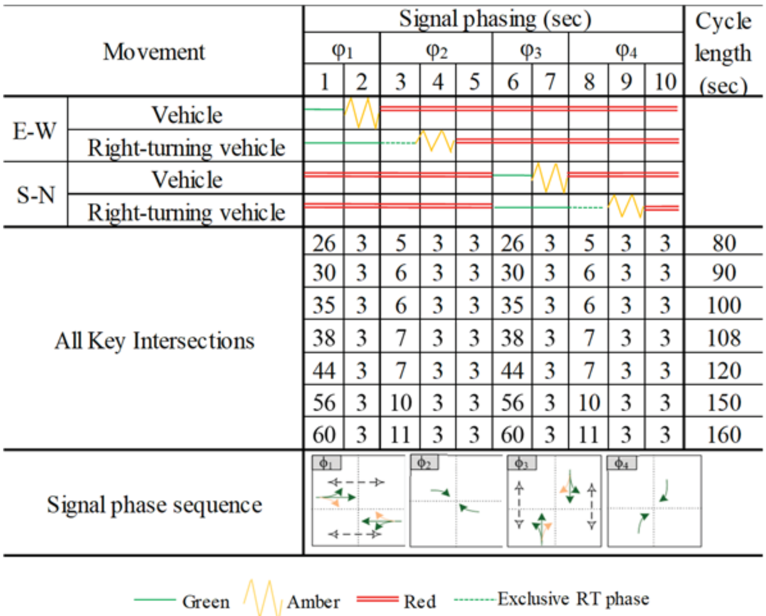


Fig. 4 Signal settings procedure adopted for this study

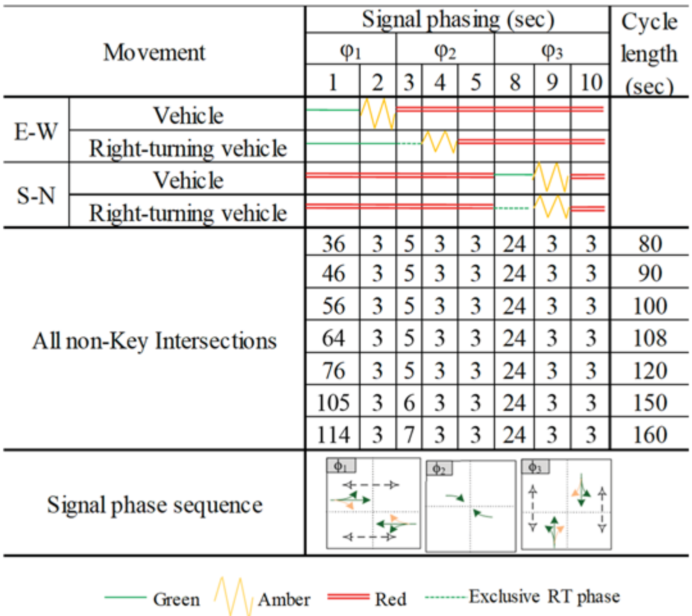
The signal settings procedure shown in Fig. 4 was then applied using each of these cycle lengths in turn and the effective green times for each intersection movement were allocated. Fig. 5 and Fig.6 show the signal settings of key intersections and non-key intersections, respectively.

Altogether, 140 scenarios (4 mean desired speeds x 7 cycle lengths x 5 cases) were developed for analysis.



Movement	φ1		φ2			φ3		φ4			Cycle length (sec)
	1	2	3	4	5	6	7	8	9	10	
E-W Vehicle											
E-W Right-turning vehicle											
S-N Vehicle											
S-N Right-turning vehicle											
All Key Intersections	26	3	5	3	3	26	3	5	3	3	80
	30	3	6	3	3	30	3	6	3	3	90
	35	3	6	3	3	35	3	6	3	3	100
	38	3	7	3	3	38	3	7	3	3	108
	44	3	7	3	3	44	3	7	3	3	120
	56	3	10	3	3	56	3	10	3	3	150
	60	3	11	3	3	60	3	11	3	3	160
Signal phase sequence	φ1		φ2			φ3		φ4			

Fig. 5 Signal settings for key intersections



Movement	φ1		φ2			φ3			Cycle length (sec)
	1	2	3	4	5	8	9	10	
E-W Vehicle									
E-W Right-turning vehicle									
S-N Vehicle									
S-N Right-turning vehicle									
All non-Key Intersections	36	3	5	3	3	24	3	3	80
	46	3	5	3	3	24	3	3	90
	56	3	5	3	3	24	3	3	100
	64	3	5	3	3	24	3	3	108
	76	3	5	3	3	24	3	3	120
	105	3	6	3	3	24	3	3	150
	114	3	7	3	3	24	3	3	160
Signal phase sequence	φ1		φ2			φ3			

Fig. 6 Signal settings for non-key intersections

For simplicity, this analysis was conducted on hypothetical networks with no pedestrians, 100% passenger cars, and simultaneous offsets only. For each mean desired speed V_{des}, the desired speed distribution in the simulation model was set as normally distributed between V_{des}±10 km/h, with mean = V_{des}. The simulation warm-up time was set to 15 minutes, and the travel time and traffic volumes collected in the following 30-minute period. Three simulation runs were conducted for each scenario. Travel time was collected for through vehicles over the entire section length (3.0 km), averaged for all vehicles in all three runs, and then converted to average travel speed. Traffic volume was converted to hourly flow simply by converting the three-run average 30-minute flow to hourly flow.

In the results section, the plots of average travel speed vs. 2-lane flow were made for each case.

4. Results and discussion

This chapter highlights the major findings of this work. The results are presented in the form of travel speed – flow curves. The chapter is concluded by highlighting how careful consideration of the relationship of the investigated factors can provide better arterial performance.

4.1. Impact of cycle length

For each case (1 – 5), we investigate the travel speed – flow relationship at each cycle length, and for each mean desired speed. Fig. 7 shows the variation with cycle length when V_{des} = 50 km/h for (a) case 1 and (b) case 2. It can be seen that cycle lengths of 150 and 160 seconds lead to the lowest average travel speeds along the section.

Although the combination of link length and mean desired speed will influence the travel speed achievable at each cycle length, this discussion is made later in the paper. Here, the focus is on showing that extremely long cycle lengths lead to lower travel speed.

For all cases 1 – 5, the cycle lengths 150 and 160 seconds consistently ranked among the three lowest performing cycle length settings. This trend was generally observed at all the mean desired speeds investigated in this study, with cycle lengths of 150 and 160 seconds leading to the lowest travel speed in most of the cases. This showed that at any mean desired speed, and regardless of the number of intersections on an arterial, longer cycle lengths always lead to lower travel speeds along an arterial. If the potential performance of the arterial were considered, such long cycle lengths would be less likely to be adopted. It also makes the case for considering arterial performance even at planning, so that any proposed designs do not call for such long cycle lengths.

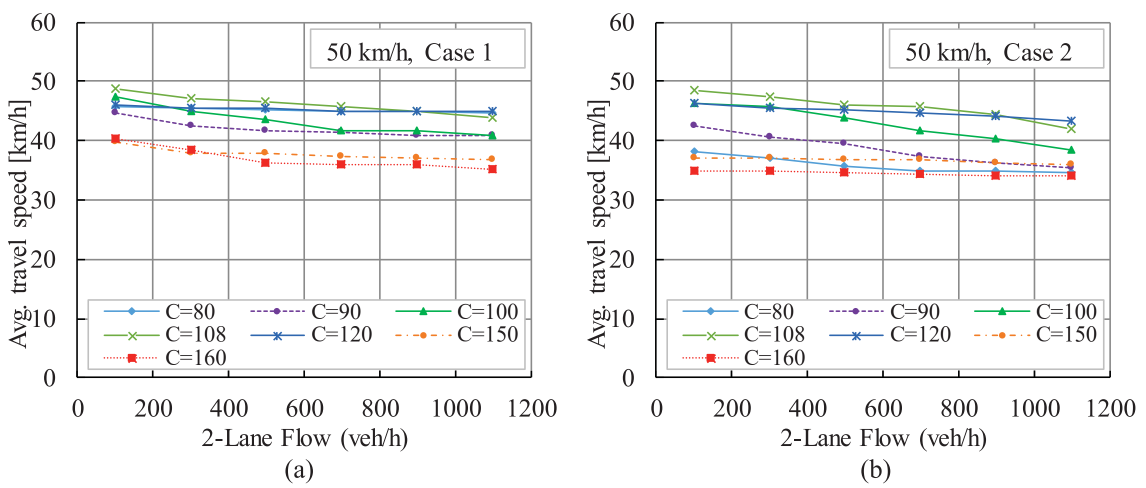


Fig. 7 Impact of cycle length at V_{des} = 50 km/h; (a) case 1, and (b) case 2

4.2. Impact of mean desired speed, V_{des}

The average travel speed – flow relationship was investigated for each case, at each cycle length, and at all four mean desired speeds. Fig. 8 (a) shows the impact of mean desired speed in case 1 when C=80s. The results are logical, since the mean desired speed is correlated to the desired speed distribution settings, and therefore the higher it is, the higher the travel speed is expected to be. For all the cases 1 – 5, this trend was generally observed.

An interesting observation was that as the number of intersections increased along the sections (case 1→Case5), the difference between the travel speeds at each mean desired speed reduced. Fig. 8 (b) shows the results for case 5 which has the highest number of key intersections, again at C=80s. It can be seen that at higher flows, the three lowest V_{des} have practically similar travel speeds. V_{des} = 60 km/h performs only marginally better (3.6 km/h) than the other mean desired speeds. This shows that at higher intersection densities and higher flows, mean desired speed has little impact on the travel speed.

Vehicles traveling below their desired speeds continuously check for opportunities to pass other vehicles. This is generally possible at low flow conditions, but the presence of other vehicles limits this possibility. In addition, at the same flow, the density of vehicles is much higher on links in case 5 than in case 1, making lane changing difficult.

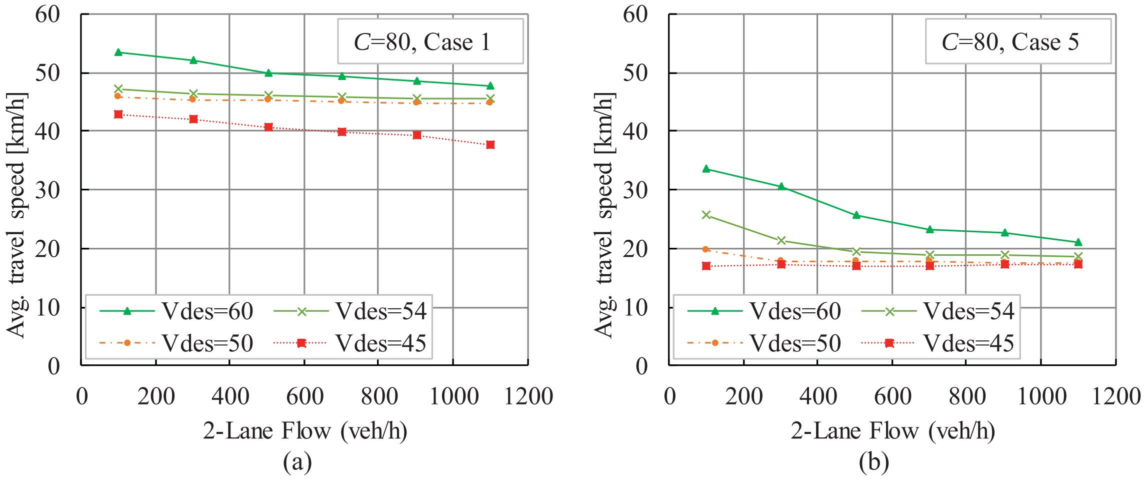


Fig. 8 Impact of mean desired speed for C = 80 s; (a) Case 1 and (b) Case 5

4.3. Impact of intersection density

In this section, we discuss how the average travel speed changes when we add more intersections onto a network. The base case is case 1, with only two intersections. Although cases 4 and 5 both have the same intersection density, case 4 includes non-key intersections, where green time for the major street is longer than minor street green time, while case 5 is composed of all key intersections, with an equal green time split. For purposes of discussing intersection density, we will consider only the cases that have all key intersections; that is cases 1, 2, 3, and 5.

We singled out the combinations of mean desired speed, V_{des} and cycle length, C that would minimize delay and maximize travel speed according to the discussion leading to the results in Table 1. For this analysis, the following pairs of [V_{des}, C] were selected: [45, 120], [50, 108], [54, 100], and [60,120].

The reason for this kind of analysis was to observe if the careful consideration of cycle length based on V_{des} and link length could counterbalance any effects of the intersection density on the average travel speed.

Because only the downstream intersections are considered to affect the travel speed along the section, the intersection density, D_{int} for each case is calculated as in Eq. (2) using the total number of signalized intersections in the arterial, N_{int} and the total arterial section length, $L_{section}$, which in this study is 3.0 km.

$$D_{\text{int}} = \frac{N_{\text{int}} - 1}{L_{section}} \tag{2}$$

Fig. 9 shows the impact of intersection density when C = 120 s, and V_{des} = (a) 45km/h, and (b) 60 km/h. For D_{int} = 0.33 and 0.67 signalized intersections/km, the travel speeds were very similar, until flow of 1100 veh/h when D_{int} = 0.67 had a 1.4km/h travel speed lower than D_{int} = 0.33. Similarly, travel speeds for cases with D_{int} = 1.33 and D_{int} = 2.67 were very similar. However, a large gap in travel speed was observed when D_{int} increased from 0.67 to 1.33. A possible reason for this observation is the arrival flow profile at the downstream intersection. A vehicle platoon departing the upstream intersection at the start of green will disperse if the distance to the downstream intersection is long enough, making the downstream arrival pattern more uniform. Bonneson et al. (2008) used delay data obtained using TRANSYT-7F to show that delay of a 1.6 km section was 10 – 14 seconds, compared to 4 – 21 seconds on a 0.43 km segment. They concluded that the longer the segment, the more uniform the arrival flow profile due to platoon dispersion, with the overall effect that delays approach values that could be found at an isolated intersection.

This tendency, including the large gap in travel speeds between case 2 and case 4 was the same at all the other investigated combinations of V_{des} and C.

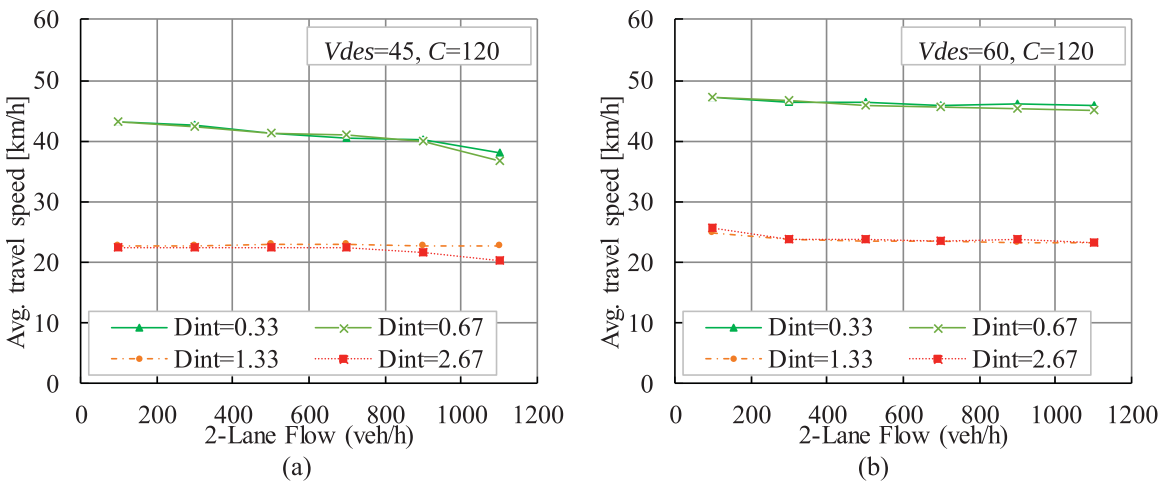


Fig. 9 Impact of intersection density for C = 120 s; (a) V_{des} = 45 km/h and (b) V_{des} = 60 km/h

These results show that despite considering delay minimizing cycle length based on link length and coordinating speed, the increased density of signalized intersections leads to drastic reductions in travel speed. However, since only four signalized intersection densities are considered in this study, to better understand the variation range, it would be worthwhile to investigate intermediate values of intersection density.

4.4. Impact of intersection type/function of intersection

In the previous section, the impact of intersection density was discussed for the cases with all key intersections. In this section, the discussion is focused on the difference between travel speeds in case 4 and case 5, which have the same intersection density, but case 4 has four non-key intersections, at which minor street flows are only 10% of the major street flow. The objective is to find out the impact of the constituent intersection types in the arterial on the average travel speed.

Fig. 10 shows the average travel speed for case 4 and case 5 when V_{des}=50km/h and (a) C=80s, (b) C=120s. It can be seen in Fig. 10(a) that travel speed in case 4 was higher than in case 5.

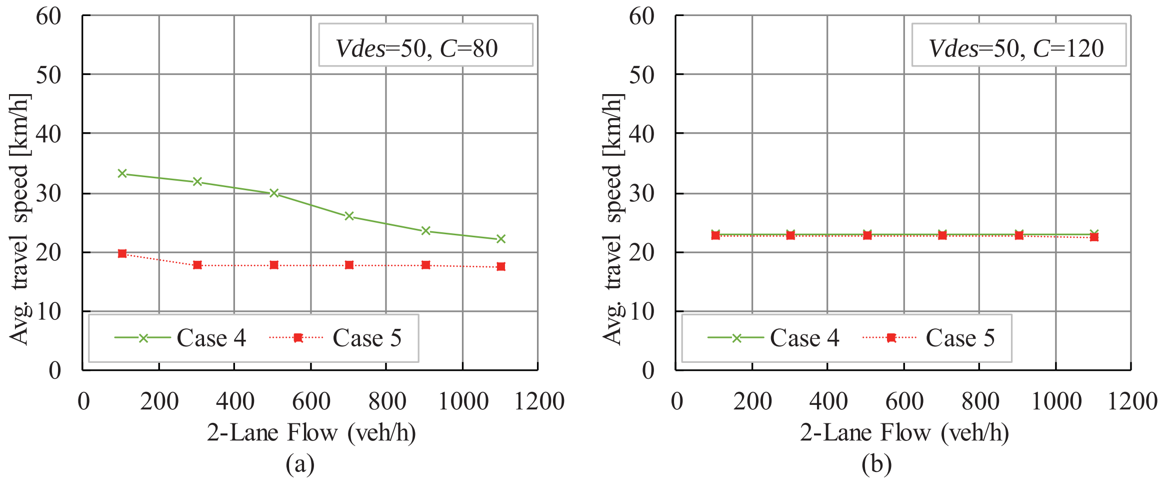


Fig. 10 Impact of intersection type/function at V_{des}=50km/h; (a) C=80s and (b) C=120s

This was observed in most of the cases because unlike in case 5 with all key intersections, 4 of the 9 intersections in case 4 were non-key intersections, at which larger green time ratio was allocated to the major street movements than at the key intersections, reducing their delay.

The major implications from this case lie in the concept of road hierarchy in which clear definitions of the function of surface streets is important in ensuring that the target speeds are met at each level in the hierarchy. The minor streets at the key intersections can be thought of as lower-level roads assigned a smaller portion of green than the higher-level road (major street), thereby maintaining the function of the higher-level road (i.e. higher travel speed).

However, as the cycle length increased from 80s, the difference between travel speed in case 4 and case 5 became smaller and smaller until it approached zero, as shown in Fig. 10 (b) when C = 120 s.

Naturally, cycle length has an important impact even in this scenario, in that the travel speed advantage was mostly achieved at lower cycle lengths, which inherently lead to lower delay. This result also indicates that although the presence of non-key intersections allowed for higher travel speeds along the major travel direction, care should be taken in selecting the cycle length to keep the speed advantage.

Furthermore, this result shows that in modeling travel speed by considering intersection density as an explanatory variable, it is important to clearly define the functions of each intersection in the arterial network, as this will change their impact.

4.5. Towards achieving the target travel speed – applicability of Eq. (1)

In this section, each mean desired speed was analyzed in turn. The objective was to examine each case in detail, and to observe which combination of factors was optimal for the achievement of an average travel speed closest to the target travel speed. Here, the mean desired speed is set as the target travel speed. A comparison was also made to the travel speeds obtained when cycle length was set to the values predicted from Eq. (1).

Eq. (1) is a reformulation of Koshi's 1989 work that hypothesized delay minimization by setting the cycle length as a function of link travel time. This equation works on the principle of the point queue theory, under which all queued vehicles leave the stop line of the upstream intersection, travel without platoon dispersion and cross the downstream intersection after the link travel time has elapsed. These are very simplifying assumptions, especially because queues actually have a certain length, and the vehicle speeds are not constant but vary within a distribution. Nonetheless, the theory gives a good baseline for vehicle delay, especially concerning the first vehicle in the queue.

By observing the travel speed – flow curves of all the cases at each cycle length and mean desired speed, we noted the cycle lengths at which the maximum travel speed was obtained for each case. A summary of these results is provided in Table 2. Values in square brackets show predictions from Eq. (1), and underlined values indicate a deviation from Eq. (1). For case 4 and case 5, values in triangular brackets show the Eq. (1) predictions, although these were not investigated in this study.

Table 2. Cycle lengths that actually yielded highest travel speed compared to [predictions from Eq. (1)]

	Mean desired speed V_{des} (km/h)			
Case (Link length, km)	45	50	54	60
1 (3.0)	120 [120]	108 [108]	100 [100]	90 [90]
2 (1.5)	120 [120]	108 [108]	100 [100]	90 [90]
3 (0.75)	80 [120]	80 [108]	80 [100]	80 [90]
4 (0.375)	90 <60>	90 <54>	80 <50>	80 <45>
5 (0.375)	100 <60>	100 <54>	90 <50>	80 <45>

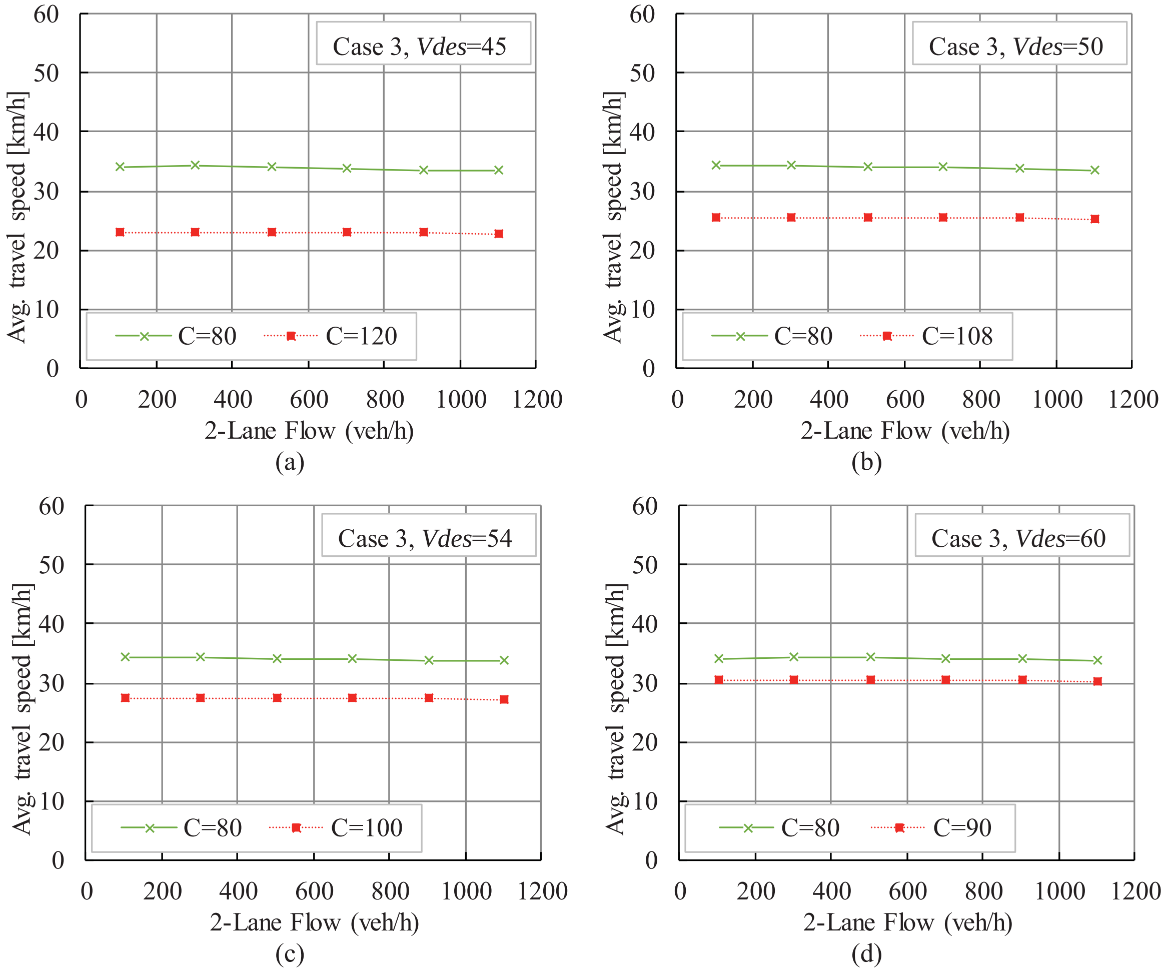


Fig. 11 Case 3; V_{des} = (a) 45, (b) 50, (c) 54, (d) 60 km/h

Results showed that Eq. (1) was accurate in predicting which cycle lengths would minimize delay and therefore yield higher travel speeds, in 8/12 scenarios (comparing only cases 1, 2, and 3 for which we investigated the predicted delay minimizing cycle lengths shown in Table 1).

Because the model in Eq. (1) is based on a single link, it was expected to be largely applicable to case 1, and to a certain extent, to case 2. This was confirmed from the simulation results, as the highest observed travel speeds were achieved at the cycle lengths obtained from Eq. (1) for case 1 and 2. However, as can be seen in Fig. 11, the cycle lengths predicted from Eq. (1) (shown by the square plots) did not yield the highest travel speed for case 3. A cycle length of 80 seconds yielded the highest travel speed at all mean desired speeds in case 3 as seen in Fig. 11 (a) – (d).

It should be mentioned that Koshi (1989) further undertook an experiment on an arterial with seven signalized intersections and two lanes per direction. By varying the cycle length and optimizing the offsets, he found that the predicted points of minimum delay were observed in some links, but not others, and this was attributed to the difference between the simplifying assumptions and the real traffic conditions.

Taking V_{des} as the target travel speeds, with the desired speed distributed between V_{des}±10 km/h, it was noted that at all V_{des}, the desired travel speed was only realized in case 1 and case 2. The more the intersection density increased, the lower the actual travel speed from the lower limit of the desired speed. These results could imply that in cases 1 and 2, the signalized intersections are sufficiently far apart for the intersections to effectively operate as isolated intersections, and therefore not be affected by the choice of offsets.

However, it was generally observed that as the intersection density increases, travel speeds fell below the target speed and its distribution. Because in this study the intersections were equally spaced, the discussion of intersection density can be extended to link lengths. The findings then show that lower travel speeds were observed on shorter links. This is consistent with the discussions in Bonneson et al. (2008), where they mention that platoon dispersion is negligible on shorter segments so that the platoon speed is limited to the leading vehicle's speed.

It should also be noted from Table 2 that in all cases the highest travel speed is achieved with cycle lengths no more than 120 seconds. This shows that at all investigated V_{des}, and the intersection density within the range studied in this work (0.33 – 2.67 signalized intersections per kilometer), a cycle length up to 120 seconds is likely to be sufficient to maximize the achievable travel speed.

For comparison, the section used to calibrate the simulation model in this study had an intersection density of 5 intersections/km, and a common cycle length of 150 seconds. According to the results of this study, this cycle length is extremely long and unnecessary, and better performance along the section could be achieved by considering shorter cycle lengths.

5. Conclusions

Average travel speed was analyzed along hypothetical 4-lane signalized arterials with varying intersection densities, cycle lengths, and mean desired speeds. The traffic was assumed to be made up of only passenger cars, no pedestrians, and only simultaneous offsets were considered.

Longer cycle lengths were found to lead to lower travel speeds along the arterials. Indeed, for all the cases analyzed, the highest travel speeds were obtained with cycle lengths of 120 seconds or less. Our objective with this analysis was to demonstrate the need for this kind of performance based analysis, which can be used by planners so that they do not adopt road designs that might call for long cycle lengths.

Higher mean desired speed led to higher travel speeds as expected. However, as the number of intersections along the arterial *and* the flow increased, the difference in the travel speed at different mean desired speeds reduced. In the case with the highest number of intersections, the difference was no more than 3.6 km/h. This showed that at high intersection densities and higher flow, the mean desired speed has little impact on the travel speed that can be attained along the arterial.

Increased signalized intersection density was also shown to reduce the average travel speed along the section due to the more frequent stops to the vehicles. A more interesting finding was that not simply the intersection density, but also the function of the intersection had a significant impact on the travel speed. For the arterial case with non-key intersections – those at which only low flows existed at the minor approaches, a higher travel speed was achieved compared to a case with same intersection density, but only key intersections. This distinction is not currently made in the literature where signalized intersection density is discussed.

The delay minimization model developed by Koshi (1989) was found to be mostly applicable to 1- or 2-link arterials, after which additional links led to noticeable deviations in delay-minimizing cycle lengths presumably due to the deviation of the actual field conditions from their hypothesized conditions. However, a more comprehensive validation of the model would require larger variation in cycle lengths and link lengths than considered in this study.

Simply considering the demand and capacity at an isolated signalized intersection will not always lead to the best network performance. For all the intersections hypothesized in this study, application of the signal settings procedure in Fig. 4 yielded an initial cycle length value of 80 seconds, which is simply based on the capacity of the individual intersection. However, it was shown that, based on the network link lengths and mean desired speeds, higher travel speeds could be achieved along the entire arterial by various cycle lengths such as 90, 100, 108, and 120 seconds.

Through this study, we showed that higher travel speed could be achieved by considering the entire network rather than individual intersections. As part of future research efforts, it would be beneficial to develop a model of travel speed that includes all the factors analyzed in this study. Such a model would serve as a relatively quick check for practitioners to get an idea of the expected performance of any planned roadways. To further ensure general applicability, it would also be necessary to consider the impact of vehicle compositions and the presence of pedestrians in the road network.

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