# Analysis of Traffic Operations at All-Way Stop-Controlled Intersections by Simulation

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All-way stop-controlled (AWSC) intersections are one of the common intersection types in the United States. Although significant research has been completed on signalized intersections and two-way stop-controlled intersections, a limited number of studies have been made of traffic operations at AWSC intersections. In addition, although a general analytical procedure now exists for AWSC intersections, it is not capable of handling the wide variety of conditions typically faced by the practicing traffic engineer. Further, it may not be computationally possible for an analytical model to handle the large number of vehicle interaction combinations that exist at AWSC intersections. This study presents a new AWSC simulation model and some of the results from that simulation. The model was tested against the field data collected during NCHRP Project 3-46, Capacity and Level of Service at Unsignalized Intersections. The model can be used to predict vehicle delay, queue length, and saturation headways, which have shown good correlation with the field data. The simulation model predicted the same level of service for about 66 percent of the cases tested, and for 100 percent of the cases the model predicted level of service within one level difference.

An intersection is an important element of a transportation system. It must be appropriately designed to optimize operations on the basis of traffic flow conditions and to minimize delay for those vehicles passing through the intersection. For a given set of traffic flow and geometric conditions, a control type must be selected on the basis of which of the three standard control types would result in optimal intersection performance.

All-way stop-controlled (AWSC) intersections are one of the common intersection types in the United States. Although significant research has been completed on signalized intersections and two-way stop-controlled (TWSC) intersections, a limited number of studies have been made of traffic operations at AWSC intersections. In addition, although a general analytical procedure now exists for AWSC intersections, it is not capable of handling the wide variety of conditions typically faced by the practicing traffic engineer. Further, it may not be computationally possible for an analytical model to handle the large number of vehicle interaction combinations that exist at AWSC intersections. The following issues must be resolved if the practicing traffic engineer is to have the tools to analyze all of the intersection control types and conditions needed.

- A theoretical (analytical) model is unable to consider all of the operational scenarios found at AWSC intersections. Although there is the possibility of extending Richardson's model by considering more vehicle interaction cases (*1*), this may not be computationally practical.
- An empirical model is limited to the traffic volume and geometric conditions of the sites for which it is calibrated; thus, it cannot be applied to a wide range of traffic conditions.
- Simulation techniques provide an efficient tool for estimating capacity and delay at AWSC intersections. However, no model currently available provides satisfactory results for capacity and delay forecasts for these intersections.

The main objective of this paper is to present a new simulation model that can be used to analyze traffic operations at AWSC intersections. The following tasks are documented in the paper:

- Driver behavior and vehicle characteristics data were collected and used in the development of the simulation model. Collected data included hesitation time (time for a driver to judge the traffic situation when he or she arrives in the first position at the stop line), vehicle move-up time (time needed for a vehicle to reach the first position at the stop line after the previous vehicle leaves the stop line), and passing time (time needed for a vehicle pass through the intersection conflict point after leaving the stop line).
- The effects of volume distribution on delay and capacity were analyzed for each approach and turning movement (left turn, through, and right turn).
- The simulation model was tested against field data collected as part of NCHRP Project 3-46, a recent national study of unsignalized intersections.
- Comparisons were made between the simulation results and other available models and procedures.

## PREVIOUS WORK

Hebert (*2*) conducted a study of AWSC intersections in 1963. He collected data at three intersections in Chicago with single lanes on each approach. He investigated the average departure headway when a vehicle on the subject approach faces (*a*) only a vehicle on the conflicting approach and (*b*) no opposing or conflicting vehicles. The capacity was determined on the basis of these headways. Hebert found that the volume split on both streets affects the saturation headway and the capacity, with maximum capacity occurring when the volume split is 50/50; that left-turn vehicles have no effect on capacity; and that capacity is increased by 0.2 percent for each 1 percent of right-turn traffic.

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Richardson (*3*) developed a queueing model to calculate capacity and delay for AWSC intersections based on Hebert's field measurements of saturation headway. He calculated capacity based on the service time, or the time that each vehicle occupies the server (i.e., is present at the stop line). The calculation of service time was based on Hebert's two cases: (*a*) the service time is 4.0 sec when there is no vehicle on the opposing and conflicting approaches, and (*b*) the service time is 7.6 sec when there is a vehicle on either the opposing or conflicting approach or both. His delay model was developed on the basis of the Pollaczek-Kintchine equation for an M/G/1 queueing system.

Current guidance for computing capacity and level of service for AWSC intersections is covered elsewhere (*4*). This procedure has recently been included in the 1994 update of the *Highway Capacity Manual* (HCM) (*5*). The capacity and delay models were developed from empirical field data collected by Kyte and Marek (*6*). Traffic volume and delay data were collected at intersections where significant queueing conditions were observed. Saturation headways, the minimum time interval between two consecutive vehicles on the same approach passing through the intersection under continuous queueing conditions, were measured. The capacity, the maximum traffic volume an intersection approach can handle under prevailing conditions, was then calculated on the basis of the average saturation headway for each approach. Regression methods were used to develop the capacity and delay models. Because the data collected were usually within a limited range, the models cannot be applied to the wide range of traffic conditions found in the real world.

Simulation models often are based on a more complex description of the world than are analytical models. Although simulation techniques have been widely used in studying traffic operations at signalized intersections, none are yet available that will accurately model AWSC intersections. Only two models, TEXAS (*7*) and STOP-4 (*8*), specifically model AWSC intersections. The TEXAS model uses kinematic calculations to track vehicles through an intersection. It was found that the quality of the simulation results from the TEXAS model varies widely, depending on traffic volume conditions. With high traffic volumes (more than 280 vehicles per hour on an approach), TEXAS significantly overestimates vehicle delay; with low traffic volumes (fewer than 100 vehicles per hour on an approach), TEXAS generally underestimates vehicle delay. The TEXAS model is also limited to a maximum of 1 hr simulation, with no variation in traffic volume allowed.

STOP-4 was developed by Chan et al. (*8*). They developed a delay model based on simulation results and field data. One of their conclusions is that intersection capacity decreases as the volume split approaches 50/50, which contradicts the findings of Hebert and Richardson and is not consistent with field data. Zion et al. (*9*) tested Chan's model and found that it always predicts higher delays than those measured in the field.

Brilon and Bondzio (*10*) studied several simulation models for TWSC intersections, including NETSIM and TEXAS, but found that only the KNOSIMO model produced realistic results.

## TRAFFIC OPERATIONS AT AWSC INTERSECTIONS

AWSC intersections require drivers on all approaches to stop before passing into the intersection. Although it is a recognized rule in some areas that the driver on the right has priority it is not a sufficient descriptor of intersection operations. What in fact happens is the development of a *right-of-way consensus* that alternates between the drivers on the intersection approaches, a consensus that is dependent primarily on the intersection geometry and the arrival patterns at the stop line.

Consider an intersection composed of two one-way streets. Here drivers alternately proceed into the intersection, one vehicle from one approach followed by one vehicle from the other approach. This same two-phase pattern is observed at a standard four-leg AWSC intersection where drivers from opposing approaches enter the intersection at roughly the same time during capacity operations. Some interruption of this pattern occurs when there are conflicts between certain turning maneuvers (such as a northbound left-turn vehicle and a southbound through vehicle), but in general the north-south streams alternate right-of-way with the east-west streams. A four-phase pattern emerges at multilane four-leg intersections, where the development of the right-of-way consensus is more difficult. Here drivers from each approach enter the intersection together as right-of-way passes from one approach to the next and each is served in turn.

Although these patterns are useful to describe the overall intersection operation, it is still necessary to consider how the patterns affect the capacity of an approach, described here as the *subject approach*. At the intersection of two one-way streets, the headways of vehicles departing from the subject approach fall into one of two cases. If no vehicles are on the other (or conflicting) approach, subject approach vehicles can enter the intersection immediately after stopping. However, if vehicles are waiting on the conflicting approach, a vehicle from the subject approach cannot enter the intersection immediately after the previous subject vehicle but must wait for the next conflicting vehicle to enter and clear the intersection. The headways between consecutively departing subject approach vehicles will be shorter for the first case than for the second. Thus the headway for a departing subject approach vehicle is dependent on the *degree of conflict* experienced in interacting with vehicles on the other intersection approaches. This degree of conflict increases with two factors: the number of vehicles on the other approaches and the complexity of the intersection geometry.

This description can be extended to the operation of an individual vehicle and the parameters needed to account for the movement of the vehicle through the intersection. A vehicle approaching the intersection has to join a queue if other vehicles are in front of it. As a vehicle in front departs from the stop line, the following queued vehicle needs a time of $t_{mv}$ (move-up time) to move to the stop line. A vehicle arriving at the intersection must stop for a minimum time $t_{hst}$, or hesitation time, before it can pass through the intersection. The time a driver needs to stay at the stop line depends on what kind of traffic movements exist on the other approaches. For example, the following traffic movements conflict with the northbound left-turn movement: southbound through, southbound right turn, eastbound left turn, eastbound through, westbound left turn, and westbound through. The conflict points are shown in Figure 1. As can be seen, there is also a conflict area in which most of the conflict points are located. Note that the paths of both the southbound left-turn movement and the northbound left-turn movement are covered by the conflict area. Although these two movements may not necessarily result in a physical conflict, field observations have shown that these two movements affect each other, especially at intersections with single lanes where space is limited. Therefore, the left-turn movements on two opposite approaches are considered conflicting movements for purposes of the simulation model described here.

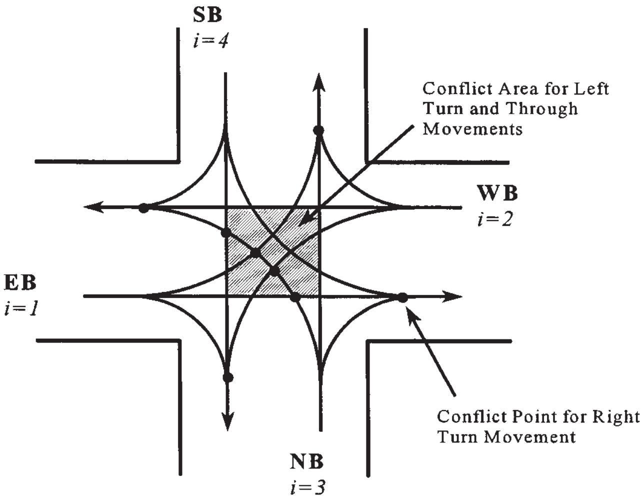


**FIGURE 1 Traffic conflicts at AWSC intersection.**

## SIMULATION MODEL DEVELOPMENT

A simulation model named AWSIM was developed for analyzing traffic operations at single-lane approach AWSC intersections using the theory and background described earlier. AWSIM was constructed with a main module and several submodules using the Quick Basic programming language. A time scan interval of 0.5 sec was used in the model to provide a sufficiently fine grain for the model to track key events. Figure 2 is a flowchart of the simulation process.

The system is assumed to be empty at the beginning of each simulation. Statistical data are collected 5 min after the simulation starts to allow for equilibrium conditions to develop.

The following data are needed for the input file: volume by each movement; vehicle mix present in the traffic stream—percentage of passenger cars, heavy trucks, light trucks, and motorcycles; the peak-hour factor and length of peak period; and the distance to any upstream signals.

Cowan's M3 headway distribution (*11*) is used to generate a series of vehicle headways $t_t$. This headway distribution takes into account the effect of platoon arrivals, which is related to the distance from upstream signalized intersections.

The status of the system is updated every 0.5 sec. The updated events include a vehicle arriving at the intersection and joining the back of a queue, a vehicle reaching the first position at the stop line, a vehicle leaving the stop line, and a vehicle passing through the conflict area. All related measures of effectiveness are collected, including queue delay, front delay, total delay, queue length, vehicle type, and turning movement type. Statistical results are calculated at the end of each simulation run.

It is crucial that the model correctly describe the driver's behavior and vehicule characteristics when he or she reaches the first position at the stop line. Unlike TWSC intersections, there is no clear priority rule. Rather, vehicles proceed into the intersection on a first-come, first-served basis. However, the vehicle's departure from the stop line also depends on its turning movement type, traffic conditions on the other approaches, and the status of the conflict area. As shown in Figure 1, each movement has a unique set of conflict points. However, the differences between the times when a vehicle passes these conflict points are very small. Therefore, the single parameter $P_{ij}$ is used to represent the passing times through these conflict points. The approach number is denoted by $i$ (1 = EB, 2 = WB, 3 = NB, 4 = SB), and $j$ denotes the movement type (1 = LT, 2 = TH, 3 = RT). The status of the conflict area is recorded by assigning a value to the parameter $P_{ij}$. A zero value implies that the conflict area is not occupied by a vehicle of movement $j$ on approach $i$. Otherwise, the time when the conflict area is cleared by a vehicle of movement $j$ on approach $i$ is stored in parameter $P_{ij}$.

A vehicle can move to the stop line position only $t_{mv}$ (move-up time) seconds after the previous vehicle leaves the stop line. Once a vehicle moves to the stop line, the first-in-queue time is recorded for this vehicle, and all other vehicles in the queue move one position ahead. A vehicle can exit from the stop line only when it has been there for a minimum time ($t_{hst}$) and no other conflicting vehicles occupy the conflict area.

Consider a left-turn vehicle on the eastbound approach as an example ($i = 1, j = 1$). Whether this vehicle can leave the stop line or not is subject to the following conditions:

$$T_{clk} \geq P_{2,1}$$
$$T_{clk} \geq P_{2,2}$$
$$T_{clk} \geq P_{2,3}$$
$$T_{clk} \geq P_{3,1}$$
$$T_{clk} \geq P_{2,2}$$
$$T_{clk} \geq P_{4,1}$$
$$T_{clk} \geq P_{4,2}$$
$$T_{clk} - t_{fq} \geq t_{hst}$$

where

$T_{clk}$ = simulation clock time,
$t_{fq}$ = first-in-queue time, and
$t_{hst}$ = hesitation time.

Once a vehicle leaves the stop line, the delay experienced by the vehicle is recorded, and the time the vehicle passes through or out of the conflict area is calculated by $P_{ij} = T_{clk} + p_{ij}$. $T_{clk}$ is the simulation clock time, and $p_{ij}$ is the passing time, the time needed for a vehicle to travel from stop line to conflict area.

The simulation model produces the following information: arrival flow rate and departure flow rate on each approach, average queue delay and total delay on each approach, average queue length and maximum queue length on each approach, and average intersection delay. The simulation model also generates two disk files: one is a queue length distribution file; the other is a history file of the simulation in which all the time events, the vehicle type, and movement type are recorded for each vehicle processed during the simulation.

## DETERMINATION OF PARAMETER VALUES OF SIMULATION MODEL

### Peak-Period Volume

The model considers traffic volume variations during the simulation period. A minimum of 1 hr is generally required in a simulation run to yield more stable results. The model assumes that the peak period is reached 15 min after simulation begins. The peak-period volume is

$$V_p(i) = \frac{V(i)}{PHF} \tag{1}$$

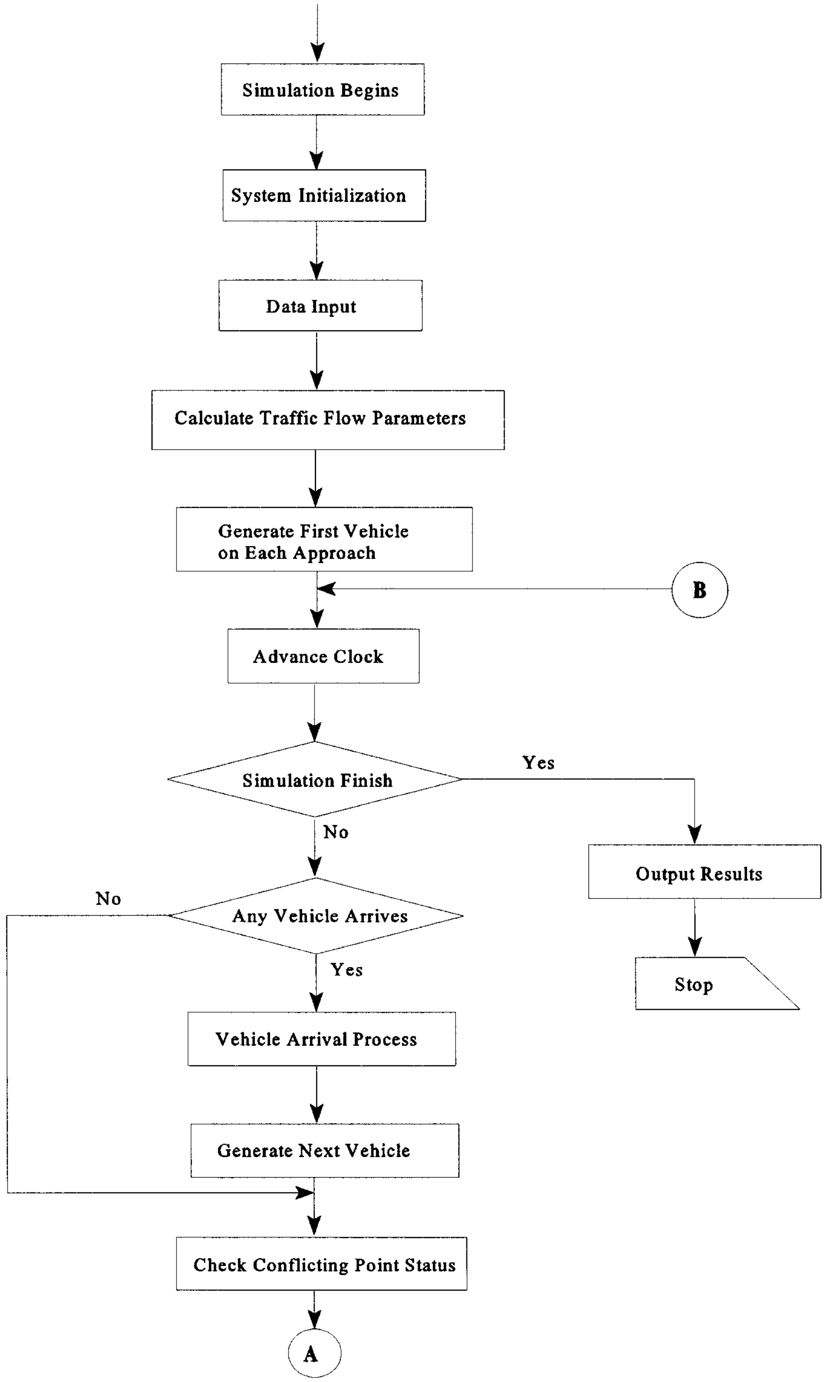


**FIGURE 2 Flowchart of simulation model.**

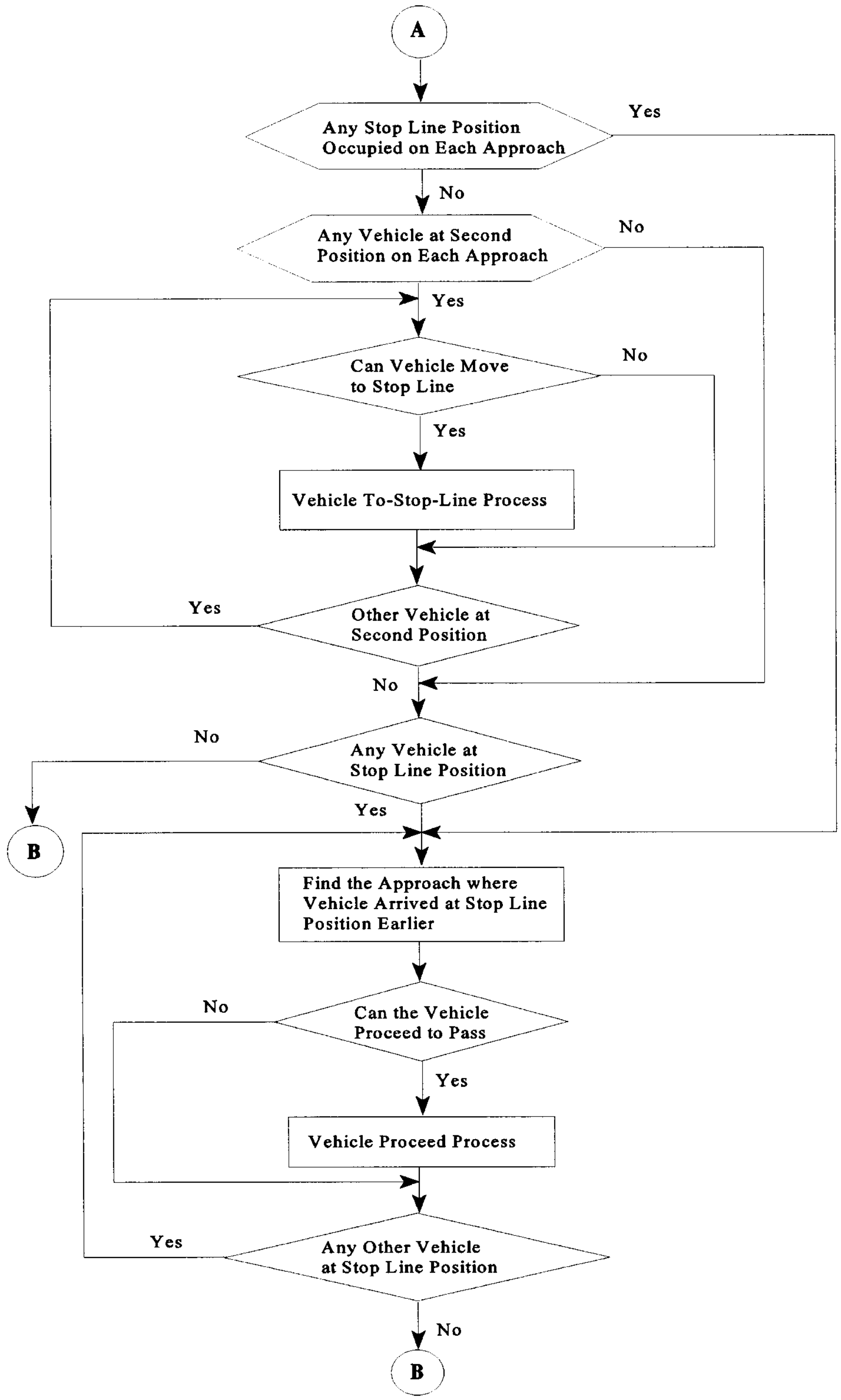


**FIGURE 2** *(continued)*

where

$V_p(i)$ = peak-period volume on approach $i$ (veh/hr),
$V(i)$ = average volume on approach $i$ (veh/hr), and
$PHF$ = peak-period factor.

Traffic volumes during the nonpeak period are

$$V_{np}(i) = \frac{T \cdot V(i) - \frac{T_p}{60} \cdot V_p(i)}{T - \frac{T_p}{60}} \tag{2}$$

where

$V_{np}(i)$ = nonpeak traffic volume on approach $i$ (veh/hr),
$T$ = simulation time (hr), and
$T_p$ = length of peak period (min).

### Proportion of Free Vehicles and Minimum Headway

Cowan's M3 model has been widely used in Australia (*12*) to study traffic operations at unsignalized intersections. The cumulative distribution function for this model is

$$F(t) = 1 - \alpha e^{-\lambda(t-t_m)} \tag{3}$$

where

$\alpha$ = proportion of free vehicles,
$t_m$ = minimum headway (sec), and
$\lambda = \alpha V/(3600 - t_m V)$

If $\alpha = 1$ and $t_m = 0$, Cowan's M3 distribution becomes the negative exponential distribution. Cowan's M3 model introduces a platoon, or bunching, factor into the model. Bunching is a natural occurrence, caused either by a slower vehicle forcing the vehicles traveling behind it to travel at its speed or by the discharge of a queue at a signalized intersection. Sullivan and Troutbeck (*13*) found that $\alpha$ can be calculated based on Equation 4 under an equilibrium state:

$$\alpha = e^{-AV} \tag{4}$$

where $A$ is a coefficient ranging between 3.7 to 7.5, which is related to the lane width and lane type, and $V$ is the traffic flow rate in vehicles per hour.

In AWSIM, a negative exponential distribution is used when the distance to the upstream signal is greater than 3.22 km (2 mi). When the distance is between 1.61 and 3.22 km (1.0 and 2.0 mi), 6.5 is used for $A$, and 1.5 is used for $t_m$. When the distance is less than 1.0 mi, the calculated $\alpha(A = 6.5, t_m = 1.5)$ is adjusted by multiplying by 0.9, resulting in more vehicles traveling within platoons.

### Hesitation Time and Move-Up Time

Hesitation time is difficult to measure directly. However, this parameter can be derived from other measurable parameters. Figure 3 illustrates the relationship between move-up time, hesitation time, and minimum headway. The relationship between these parameters is

$$t_{hst} = h_s - t_{mv} \tag{5}$$

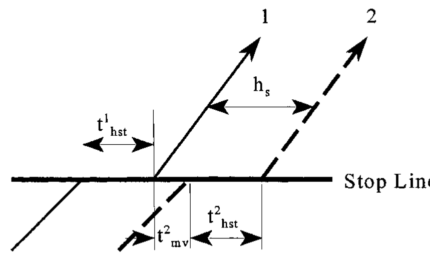


**FIGURE 3 Relationships between move-up time, hesitation time, and minimum headway.**

The minimum headway $h_s$ is referred to as saturation headway of Case 1 in Transportation Research Circular (TRC) 373 (*4*) Kyte et al. (*1*) measured $t_{mv}$ and $h_s$ based on field data collected from several intersections. The values of $t_{mv}$ used in the simulation model are as follows: $t_{mv} = 1.8$ for passenger cars, 2.2 for light trucks, 3.0 for heavy trucks, and 1.5 for motorcycles.

The values of $t_{hst}$ used in the simulation model are based on traffic conditions on the other approaches and are as follows: $t_{hst} = 2.0$ for no vehicle on other approaches, 2.2 for vehicles on any one of the other approaches, and 2.5 for vehicles on more than one of the other approaches. For a passenger car, the minimum headway value would be 1.8 + 2.0 = 3.8 sec.

### Passing Time

Passing time ($p_{ij}$) is the time required for a vehicle moving from the stop line to the conflict area. This parameter is used to predict the time a vehicle passed through the conflict area. It is assumed to be a function of the vehicle type and movement type and is given by Table 1. The passing times given by Table 1 were iteratively derived from the simulation results, which have provided the best fit to the field data.

## TESTING SIMULATION RESULTS

The simulation model was tested against the field data collected from 10 single-lane approach intersections. Data were extracted from videotapes using the Traffic Data Input Program (TDIP) (*14*). The following events were recorded and assembled into a single data file for each vehicle: directional movement, vehicle type, the time a vehicle passed through the conflict area, the time a vehicle entered a queue, the time a vehicle reached the stop line, and the time a vehicle left the stop line. Additional computer software was developed to calculate other required traffic flow parameters including volume, vehicle delay, average queue length, percentage of turning movements, and percentage of vehicle types. Data were aggregated using 5- and 15-min intervals. The difference between the times a vehicle entered a queue and reached the stop line is defined as *queue delay*. The difference between the times a vehicle reached the stop line and left the stop line is defined as *front delay*. Total delay is the sum of queue delay and front delay. The average delay for a specific time interval is calculated from the delays experienced by the vehicles passing through the intersection during that time interval. Queue length was collected from time slices. The number of vehicles in a queue was counted for every 20 sec, and the average queue length was calculated from these counts.

**TABLE 1 Passing Times Used for Simulation Model**

| Vehicle Type | Movement Type | | |
|---|---|---|---|
| | Left Turn | Through | Right Turn |
| Passenger Car | 3.0 | 3.0 | 2.8 |
| Light Truck | 3.5 | 3.5 | 3.2 |
| Heavy Truck | 5.0 | 5.0 | 5.0 |
| Motorcycle | 2.5 | 2.5 | 2.3 |

## Analysis of Vehicle Delay

Delay was collected in a similar manner in the simulation model. Testing of the delay results was based on 15-min interval data. The traffic flow parameters from each 15-min interval were used for the simulation. A 2-hr simulation time was used for each simulation run. A PHF of 1.0 was used for the simulation. With the same data set, the TRC 373 procedure was also tested against the field data. Figure 4 shows the comparisons of the simulation results and TRC 373 results with the field data. The $R^2$-values are 0.71 and 0.61 for AWSIM and TRC 373, respectively. Two other statistical parameters were also calculated: mean absolute error (MAE) as defined in Equation 6, and mean absolute percent error (MAPE) as defined in Equation 7:

$$MAE = \frac{1}{n}\sum_{i=1}^{n}\left|d_s^i - d_f^i\right| \tag{6}$$

$$MAPE = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{d_s^i - d_f^i}{d_f^i}\right| \tag{7}$$

where

$n$ = number of data points,
$d_s^i$ = delay from simulation (sec/veh), and
$d_f^i$ = delay from the field (sec/veh).

An *MAE* of 2.4 sec/veh and an *MAPE* of 25.5 percent were obtained for AWSIM, and an *MAE* of 3.1 sec/veh and an *MAPE* of 31.5 percent were obtained for TRC 373.

Comparisons were also made in terms of level-of-service (LOS) forecasts by the two models. The LOS calculation was based on the same criteria suggested in the HCM (*5*). For about 66 percent of the cases, the simulation model predicts the same level of service, and for 100 percent of the cases, the simulation predicts a level of service within one level difference. The TRC 373 gives similar range of LOS forecasts: about 65 percent with the same level of service, and 99 percent within one level difference. However, it is of interest to note that of the 203 cases tested, 30 exceeded the valid volume ranges for the TRC 373 procedure.

## Analysis of Queue Length

Average queue length was compared between the field data and the simulation results. Similar to the delay data, 15-min interval data from the field were used. Figure 5 shows the comparison between the field data and the simulation results.

Figure 5 shows that the queue length data from simulation are consistently, but not significantly, higher than the field data. The $R^2$ is 0.77 based on regression results, which means that there is good correlation between the simulation results and the field data. The reason for the difference is probably due to the methods used to collect the queue data from the simulation and the field. In the simulation model, every time a queue changes, the queue length is recorded, and the average queue length is calculated from these recorded queue lengths. However, the queue length from the field was collected based on time slices: the number of vehicles in a queue was counted every 20 sec, and the average queue length was

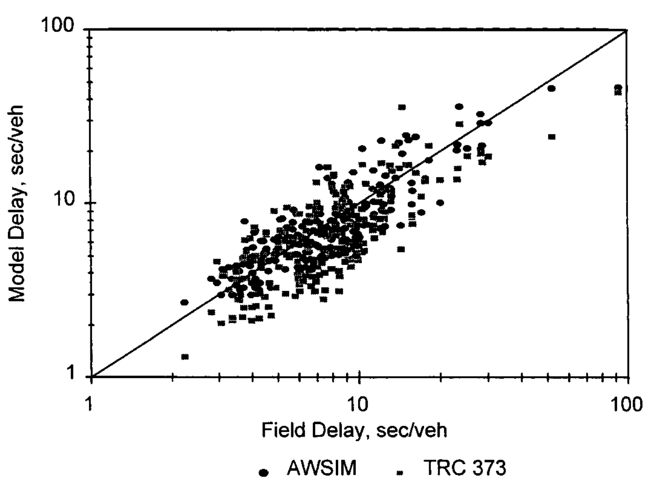


**FIGURE 4 Comparisons of delay forecasts by AWSIM and TRC 373.**

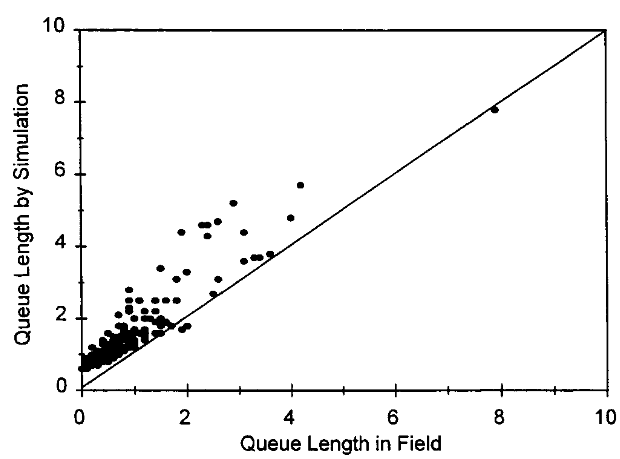


**FIGURE 5 Comparison of queue length between simulation and field data.**

obtained by averaging these queue length values. Therefore, the queue length from simulation is generally larger than the field data because in many cases the queue length is zero, and these zero queue cases are not taken into account in calculating the queue length in the simulation model.

### Analysis of Saturation Headways

The saturation headway determines the capacity for AWSC intersections. However, the saturation headway at an AWSC intersection is defined in a different manner from that at signalized intersections. On the basis of the traffic flow conditions faced by a vehicle at the stop line (subject vehicles), the saturation headway can be classified into different cases. Kyte et al. (*1*) classified the saturation headway into eight cases depending on the situation a subject vehicle faces at the stop line. As an example, saturation headway of Case 6, which was defined for the situation in which a subject vehicle faces one vehicle on the opposing approach and one vehicle on the conflicting approach from the left side, equals 7.3 sec.

Such a classification of the saturation headway cases at AWSC intersections is probably the most detailed that can be handled by an analytical model. However, it must be noted that these measurements are based on normal traffic volume compositions. With different proportions of turning movements and vehicle types, the saturation headway can be significantly different even for the same case. For example, the headway for a through vehicle facing a left-turn vehicle on the opposing approach can be very different from the headway for a vehicle facing a through or right turn on the opposing approach. By analyzing the saturation headways from the simulation results, more insight can be gained on whether the model correctly simulates real-world conditions.

Saturation headways were compared for two intersections in which a significant number of observations of saturation headways exists. Using the traffic volumes at the intersections, simulation runs were made, and saturation headways were collected from the simulation results. Table 2 shows the saturation headway data from both field measurement and simulation. It can be seen that the simulation results are well correlated with the field data. The difference is generally within 0.5 sec. The results indicate that the simulation model adequately models the operational characteristics at AWSC intersections.

### Effect of Volume Distribution on Capacity and Delay

Research has been done by Hebert (*2*), Richardson (*3*), and Chan (*8*) on the effect of volume split on intersection capacity. Different conclusions were obtained by these researchers. The effect of volume split on intersection capacity was investigated here using the AWSIM model. However, it is difficult to judge by simulation when the intersection reaches capacity conditions because increasing the demand traffic volume usually causes the departure flow rates to increase. Therefore, a delay criterion was set for obtaining the capacity for AWSC intersections. The intersection capacity was determined by the maximum departure flow rate of the intersection, while the average intersection delay was not more than 45 sec/veh, the break between LOS E and F. Only passenger cars were used for the traffic volumes. Table 3 summarizes the results using these four models.

The results from AWSIM showed that with a 50/50 demand volume split, maximum intersection capacity was obtained. This is consistent with the field data and values forecast by Hebert and Richardson. However, Chan's model forecasts values that are not consistent with the other models. With a 100/0 demand split, the minimum intersection capacity was obtained. With the increase of left-turn movements on each approach, the capacity decreases compared with the case in which only through movements exist on each approach.

## CONCLUSIONS

A simulation model, AWSIM, was developed for analyzing traffic operations at AWSC intersections. The model was tested against the field data collected as part of NCHRP Project 3-46. Comparisons were made between the simulation and field data for vehicle delay, queue length, and saturation headway. The effect of volume split on intersection capacity was also investigated. The following conclusions are based on the results presented in this paper:

- Delay forecasts by the simulation model were well correlated with the field data. Simulation predicted the same level of service for about 66 percent of the cases and was within one level difference for 100 percent of the cases. TRC 373 procedure also gives good forecasts for vehicle delay and level of service. However, the TRC 373 procedure could be invalid when traffic volume compositions exceed model ranges.

**TABLE 2 Comparison of Saturation Headways Between Field Data and Simulation**

| Case | Saturation Headway, seconds | | |
|---|---|---|---|
| | Field | Simulation | Absolute Error, seconds |
| 1 | 4.1 | 3.8 | 0.3 |
| 2 | 4.9 | 4.6 | 0.3 |
| 3 | 5.7 | 5.7 | 0 |
| 4 | 6.0 | 5.9 | 0.1 |
| 5 | 7.1 | 7.5 | 0.4 |
| 6 | 7.0 | 6.8 | 0.2 |
| 7 | 7.2 | 6.6 | 0.6 |
| 8 | 9.0 | 8.9 | 0.1 |

**TABLE 3 Capacity for AWSC Intersections with Different Volume Splits**

| | Capacity | | | | |
|---|---|---|---|---|---|
| Demand Split | Hebert | Richardson | Chan | AWSIM[1] | AWSIM[2] |
| 50/50 | 1900 | 1900 | 1076 | 2100 | 1700 |
| 70/30 | 1500 | 1560 | 2419 | 1800 | 1600 |
| 100/0 | * | 1800 | * | 1600 | 1400 |

Notes:
1. Only through movement on each approach.
2. 20% left turn, 60% through, and 20% right turn.

- Although the queue length forecast by simulation is consistently higher than the field data, the results showed that there is good correlation between simulation results and field data. The difference was caused by the methods used to collect queue data from the simulation model and in the field. The methods can be changed to improve consistency.
- Saturation headway forecasts by simulation showed good correlation with the field measurements, which means that the model correctly simulates traffic operational characteristics for AWSC intersections.
- The effect of volume split on intersection capacity was found to be consistent with both Hebert's and Richardson's data. However, with a split of 100/0, this result contradicts Richardson's result. No field data are available for verification.

The current version of AWSIM is limited to simulating intersections with single-lane approaches. Further enhancement is necessary to handle intersections with flared approaches or multilane approaches, but the model provides, even in its present form, a useful tool in the analysis of AWSC intersections.

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