Capacity at All-Way Stop-Controlled and First-In-First-Out Intersections

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ABSTRACT

A new theoretical approach for the determination of capacities at All-Way Stop-Controlled and First-In-First-Out intersections is presented. This approach is based on the Addition-Conflict-Flow method developed from the graph theory. The new procedure considers All-Way Stop-Controlled intersections in such a way that the First-In-First-Out discipline applies. The new procedure can handle most common lane configurations in the real world including multilane approaches.

A simple and more practical procedure is recommended. In practice, this simplified procedure can be used for single-lane approaches as well as single-lane approaches with separate left-turn traffic lanes. This procedure is verified and calibrated with measured data from All-Way Stop-Controlled intersections in the USA. For the calculation of capacity at All-Way Stop-Controlled intersections, general parameters, which are determined by calibration, are proposed. The procedure produces coincides with measurements more precise than the existing methods.

1. INTRODUCTION

All-Way Stop-Controlled (AWSC) intersections are the most used road intersections in the North America. First-In-First-Out (FIFO) intersections are broadly used in the developing countries (e.g., China and India). Because no traffic streams at AWSC intersections possesses the absolute priority of driving, the AWSC intersections can also be considered in such a way that the First-In-First-Out discipline applies.

Hebert (6) investigated the AWSC intersections in 1963. Based on the departure headways measured by Hebert, Richardson (11) developed a model for the calculation of the capacity and delay at AWSC intersections. He calculated the capacity based on the service time. In the 1994 HCM (12) an empirical approach is applied. The capacity and delay are determined by regression of field data (7,9). In the 1997 HCM (10) the model of Richardson is used with some extensions. The disadvantage of this procedure is that the result can only be determined by iteration. Therefore, a calculation without computational aids is impossible.

This paper presents a new theoretical approach based on the idea of the Addition-Conflict-Flow (ACF) procedure (4). The mathematical background of this approach is the graph theory (2). This approach considers all possible traffic streams and conflict points at AWSC/FIFO intersections simultaneously. Combining with the procedure of shared and short lanes (13), all traffic constellations at AWSC/FIFO intersections can be taken into account. The application of this approach is relatively simple. This approach is explicit with respect to the parameters to be determined (capacity, delay, etc.) and therefore it requires no iterative steps of computation.

This procedure can take into account (a) the number of lanes at the approaches, (b) the distribution of traffic flow rates, (c) the number of pedestrians at the approaches, (d) the flared area at the approaches, and (e) the interaction between the different streams.

2. DEPARTURE MECHANISMS AT AWSC/FIFO INTERSECTIONS

2.1 Capacity of Streams in a Departure Sequence

Since all streams at AWSC/FIFO intersections are considered to be equal in the hierarchy of the priority of departure, the vehicles of different streams must enter the intersection alternatively (one stream's vehicle after another stream's vehicle). The vehicles in different streams have to pass the same conflict area alternatively, one after another. Every vehicle of the streams occupies the conflict area by a time of t_B seconds. In the case of an intersection of two one-way streets (only two streams), this corresponds to the rule of zipping. The vehicles of the two streams can enter the intersection alternately. The two streams should have the same capacity by permanent queuing. That means, in a departure sequence, all streams must have the same capacity if all traffic flow rates, Q_i, exceed their capacities, C_i, (total overload). That is, under the overload condition, the capacities of all streams in one departure sequence have the same value of

$$C_i = C = \frac{3600}{\sum t_{B,i}} \text{ for } Q_i \geq C \qquad \text{(veh/h)} \qquad (1)$$

This equally distributed capacity, C, is equal to the number of the seconds within an hour divided by the sum of the occupation times of all involved streams, $t_{B,i}$.

Considering the fictive configuration of streams in Figure 1 and searching for the capacity of stream 3, $C_{3,}$ the following equation can be obtained:

$$C_3 = C_2 = C_1 = C = \frac{3600}{t_{B,1} + t_{B,2} + t_{B,3}} \text{ for } Q_1 \geq C,\ Q_2 \geq C, \text{ and } Q_3 \geq C \qquad \text{(veh/h)} \qquad (2)$$

From the point of view of the subject stream for which capacity should be determined, the capacity C will be achieved also in the case that the traffic flow of this stream is lower

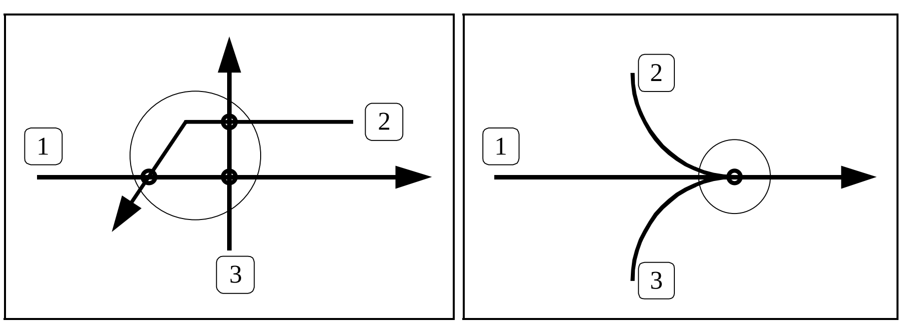

FIGURE 1 Three streams in a departure sequence.

than the capacity C. The argument of this consideration is that for estimating the capacity a traffic flow rate, which is equal to or larger than the capacity, has to be applied fictively. Stream 3, e.g., has the potential to serve as many vehicles as C, even if Q_3 is lower than this volume C. That is, stream 3 has at least the capacity of

$$C_3 = C = \frac{3600}{t_{B,1} + t_{B,2} + t_{B,3}} \text{ for } Q_1 \geq C \text{ and } Q_2 \geq C \qquad \text{(veh/h)} \qquad (3)$$

as long as streams 1 and 2 are oversaturated.

If, however, other streams than the subject stream (here stream 3) cannot consume the admitted capacity C, i.e., the traffic flow rates there are lower than the admitted capacities, these capacities can then be used by the other streams. In the case of $Q_1 < C$ and $Q_2 > C$ (partial overload for stream 2), the capacity for stream 3 is

$$C_3' = \frac{3600 - Q_1 \cdot t_{B,1}}{t_{B,2} + t_{B,3}} \text{ for } Q_1 < C \text{ and } Q_2 > C \qquad \text{(veh/h)} \qquad (4)$$

C' is always larger than C. If now Q_2 is in turn lower than C', the remaining capacity must be distributed again. Then a capacity of

$$C_3'' = C'' = \frac{3600 - \left(Q_1 \cdot t_{B,1} + Q_2 \cdot t_{B,2}\right)}{t_{B,3}} \text{ for } Q_1 < C \text{ and } Q_2 < C' \qquad \text{(veh/h)} \qquad (5)$$

can be obtained for stream 3. C'' is always larger than C'. Analogously the capacities

$$C_3' = C' = \frac{3600 - Q_2 \cdot t_{B,2}}{t_{B,1} + t_{B,3}} \text{ for } Q_1 > C \text{ and } Q_2 < C \qquad \text{(veh/h)} \qquad (6)$$

and

$$C_3'' = C'' = \frac{3600 - \left(Q_1 \cdot t_{B,1} + Q_2 \cdot t_{B,2}\right)}{t_{B,3}} \text{ for } Q_1 < C' \text{ and } Q_2 < C \qquad \text{(veh/h)} \qquad (7)$$

can be obtained in the case of partial overload for stream 1.

Summarizing the results in the case of $Q_1 < C$ and $Q_2 > C$ and in the case of $Q_2 < C$ and $Q_1 > C$, the capacity for stream 3—in the case that no total overload in the conflict streams (stream 1 and 2) occurs—is

$$C_3'' = C'' = \frac{3600 - \left(Q_1 \cdot t_{B,1} + Q_2 \cdot t_{B,2}\right)}{t_{B,3}} \qquad \text{(veh/h)} \qquad (8)$$

for $$Q_1 < \frac{3000}{t_{B,1} + t_{B,2} + t_{B,3}} \text{ and } Q_2 < \frac{3000 - Q_1 \cdot t_{B,1}}{t_{B,2} + t_{B,3}}$$

or $$Q_1 < \frac{3000 - Q_2 \cdot t_{B,2}}{t_{B,1} + t_{B,3}} \text{ and } Q_2 < \frac{3000}{t_{B,1a} + t_{B,2} + t_{B,3}}$$

This regularity of operation can be extended to departure sequences with arbitrarily many streams. In all cases the following is valid for the average occupation time of the stream i:

$$t_{B,i} = \frac{3600}{C_i} \quad \text{(sec)} \qquad (9)$$

It is clear that the capacities of the streams are not distributed proportionally to their traffic flow rates Q_i. In an overloaded departure sequence, the capacities of all streams are equal. In a not overloaded departure sequence, the capacity of a stream is the traffic flow rate that can departure within the time which cannot be consumed by the other streams.

It can be recognized that there is a partial-overloaded state between the states of overload and non-overload. In this state, the capacities of the overloaded streams have the same value. The capacities of other non-overloaded streams can be determined in such a way that the traffic flow rates are increased until these streams are overloaded too. For the mentioned example above, the capacity of stream 3 in the partial-overloaded state is given by Equations (4) and (6).

The determination of capacities in the partial-overloaded state is in general a problem of optimization in sense of Operations Research since the capacities in overloaded state have to be distributed between the streams repeatedly. With the method of Linear Planing, the capacities of all individual streams can be determined. Here, the accurate formulation of this problem is renounced. The formulae derived for partial-overloaded state for a special configuration with three streams have shown the way of working for a simplified problem [cf. Equations (4) and (6)].

The partial-overloaded state is a transition state between the overloaded and the non-overloaded state. This partial-overloaded state occurs very rarely and very briefly, and it is therefore neglected for further derivations and replaced by the non-overloaded state. The resulted deviations caused by this simplification can be considered as very small. An insignificant underestimate of capacity may be caused by this simplification in the partial-overloaded area. From the point of view of traffic performance, this underestimate lies in general on the safer side.

2.2 Capacity of Streams Involved in More Than One Departure Sequence

The capacity of a stream involved in more than one departure sequence is the smallest capacity which a stream can achieve in each of these departure sequences. That is (cf. Figure 2):

$$C = \min\left(C(\text{Sequence A}),\ C(\text{Sequence B}),\ \ldots\right) \qquad (10)$$

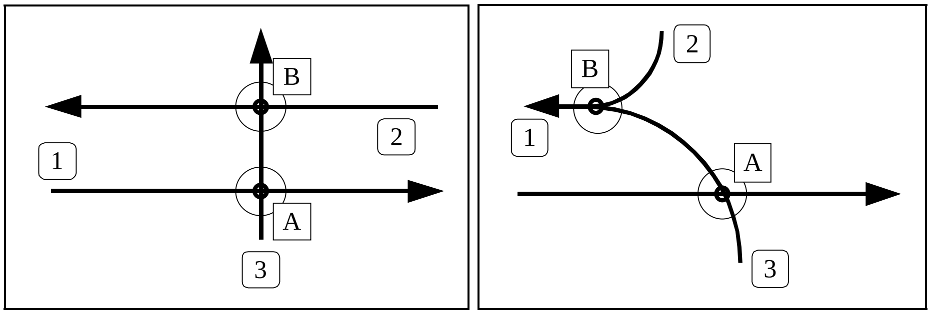

FIGURE 2 A stream involved in several departure sequences.

This postulate is based on the fact that the vehicles from more than one departure sequences can departure together and simultaneously.

3. INTERSECTIONS OF TWO TWO-LANE STREETS

Now an intersection of two two-lane streets is considered. At such an intersection, there is only one traffic lane in each of the approaches. For the derivation of the capacity formula, it is first assumed that each turning movement has its own traffic lane at the intersection (Figure 3a). For this special configuration, the capacities of all streams can be derived in the following.

For the considered intersection, the critical conflict areas can be defined according to the graph theory (Figure 3b) (4). The conflict areas can be distinguished according to the types of the conflict into (a) exit-conflicts (departure sequences No. 1, 2, 3, 4); (b) center-conflicts (departure sequences No. 5, 6, 7, 8); and (c) entrance-conflicts (departure sequences No. 9, 10, 11, 12).

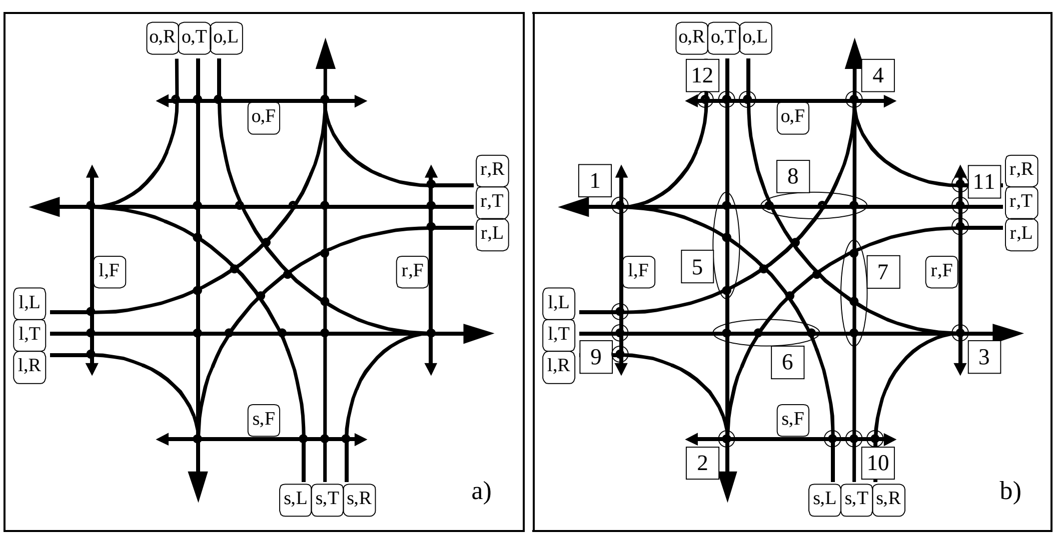

FIGURE 3 Intersection with 12 vehicle and 4 pedestrian streams and the critical conflict areas between the streams.

For further derivations, the following indices are used with the southern approach (at the bottom) as subject approach:

s, o, r, l	=	subject approach, opposite approach, approach to the right, and approach to the left
R, T, L	=	right-turn stream, through-ahead stream, and left-turn stream
A, Z, E	=	conflict at exit, conflict in the center of the intersection, and conflict at entrance.

3.1 Capacity of the Streams

The streams involved in the same conflict area form a departure sequence. In a departure sequence, the streams are incompatible with each other and they can only enter the intersection alternatively. A stream at AWSC/FIFO intersections is always involved in several departure sequences. The smallest capacity, which a stream can achieve from these departure sequences, is the decisive capacity. It is hereby assumed that vehicles of two streams, which are compatible with each other, can enter the intersection simultaneously. The capacity of the individual streams is derived in the following. Here, only the cases of overload and non-overload are considered. The case of partial-overload is neglected for simplification.

The streams from the subject approach are involved in different departure sequences. These departure sequences are described in Table 1.

TABLE 1 Departure Sequences at Intersections of Two Two-Lane Streets (Figure 3b)

Subject stream	Involved departure sequences	Conflict streams
Left turn (s,L)	No. 1 (A) No. 5 (Z1) No. 6 (Z2) No. 10_L (E)	**s,L**-o,R-r,T-l,F **s,L**-o,T-r,T-l,L **s,L**-o,T-r,L-l,T **s,L**-s,F
Through ahead (s,T)	No. 4 (A) No. 7 (Z1) No. 8 (Z2) No. 10_T (E)	**s,T**-r,R-l,L-o,F **s,T**-o,L-r,L-l,T **s,T**-o,L-r,T-l,L **s,T**-s,F
Right turn (s,R)	No. 3 (A) No. 10_R (E)	**s,R**-o,L-l,T-r,F **s,R**-s,F

In general, the capacity of the stream i in a departure sequence with n streams is [cf. Equations (3) and (8)]

$$C_i = \max\begin{cases} \dfrac{3600 - \sum\limits_{j=1, j\neq i}^{n} (Q \cdot t_B)_j}{(t_B)_i} & \text{in case of non-overload} \\ \dfrac{3600}{\sum\limits_{j=1}^{n} (t_B)_j} & \text{in case of overload} \end{cases} \qquad \text{(veh/h)} \quad (11)$$

Therefore, it is the larger one of the capacities of the two cases of overload and non-overload. (The case of partial-overload is neglected here.)

If one stream has priority over the other streams, the time that the intersection is occupied by this priority stream must be subtracted from the total time for the case of overload. The general formula for the capacity of the stream i in a departure sequence with n streams, within which m streams have the absolute priority, then is

$$C_i = \max\begin{cases} \dfrac{3600 - \sum\limits_{j=1, j\neq i}^{n} (Q \cdot t_B)_j}{(t_B)_i} & \text{in case of non-overload} \\ \dfrac{3600 - (Q \cdot t_B)_m}{\sum\limits_{j=1, j\neq m}^{n} (t_B)_j} & \text{in case of overload} \end{cases} \qquad \text{(veh/h)} \quad (11)^*$$

In case of pedestrian streams, the number of the pedestrian groups should be applied for Q instead of the absolute number of the pedestrians. The determination of the number of the pedestrian groups as a function of the absolute number of the pedestrians is not handled here, it is given elsewhere by other authors. In case of very weak pedestrian flow rates, also the absolute number of the pedestrians can be used for Q.

For instance, the capacity of the left-turn stream is determined by the departure sequences No. 1, No. 5, No. 6, and No. 10_L (cf. Table 1 and Figure 3b). For the Exit-conflict (departure sequence No. 1), the departure sequence is: **s,L**-o,R-r,T-l,F. It is assumed that the pedestrians have always priority over car traffic. The capacity of the left-turn stream in this sequence then is

$$C_{s,L,A} = \max\begin{cases} \dfrac{3600 - \left[(Q \cdot t_B)_{o,R} + (Q \cdot t_B)_{r,T} + (Q \cdot t_B)_{l,F}\right]}{(t_B)_{s,L}} \\ \dfrac{3600 - (Q \cdot t_B)_{l,F}}{(t_B)_{s,L} + (t_B)_{o,R} + (t_B)_{r,T}} \end{cases} \qquad \text{(veh/h)} \quad (12)$$

Similarly, the capacity of the left-turn stream can be calculated according to the departure sequences No. 5 ($C_{s,L,Z1}$), No. 6 ($C_{s,L,Z2}$), and No. 10_L ($C_{s,L,E}$), respectively. The decisive capacity of the left-turn stream is then

$$C_{s,L} = \min\left(C_{s,L,A}, C_{s,L,Z1}, C_{s,L,Z2}, C_{s,L,E}\right) \quad \text{(veh/h)} \quad (13)$$

Using the same procedure, the decisive capacity of the through-ahead stream and the right-turn stream can be computed by the equations

$$C_{s,T} = \min\left(C_{s,T,A}, C_{s,T,Z1}, C_{s,T,Z2}, C_{s,T,E}\right) \quad \text{(veh/h)} \quad (14)$$

and

$$C_{s,E} = \min\left(C_{s,R,A}, C_{s,R,E}\right) \quad \text{(veh/h)} \quad (15)$$

3.2 Capacity of the Approach

The approaches of an intersection of two two-lane streets, which have only one traffic lane (SL = single lane) in each direction, are now considered. Two cases can be distinguished: (a) there is no separate lane for the turning streams (Figure 4a) and (b) there is a separate lane for the left-turn streams (Figure 4b).

The capacity of the approach can be calculated by the formula for shared traffic lanes (13). In case of no flared area at the approach for the right-turn stream the capacity of the shared traffic lane is

$$C_{s,m} = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{x_{s,L} + x_{s,T} + x_{s,R}} \quad \text{(veh/h)} \quad (16)$$

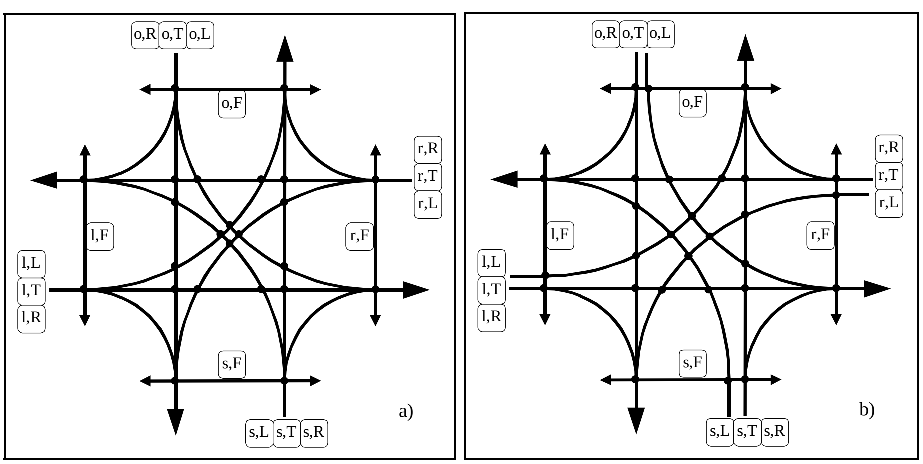

FIGURE 4 Lane distribution at approaches of an intersection of two two-lane streets.

In addition, the capacity of the traffic lane has to be checked in accordance with the restriction

$$(Q \cdot t_B)_{s,L} \cdot + (Q \cdot t_B)_{s,T} + (Q \cdot t_B)_{s,R} + (Q \cdot t_B)_{s,F} \leq 3600 \qquad \text{(sec)} \quad (17)$$

4. INTERSECTION OF TWO FOUR-LANE STREETS

An intersection of two streets, which have two lanes for each direction (DL = double lane), is now considered. It is assumed that no separate turning lanes are available (Figure 5a). The conflict areas of this configuration are represented in Figure 5b. The departure sequences of the streams of this intersection are represented in Table 2.

As for intersection with SL-approaches, the capacities for all streams in the DL-approaches are at first determined under the assumption that they all have their own separate traffic lanes. Here the following capacities should be calculated according to Equation (11). The decisive capacities of the left-turn, through-ahead, and right-turn streams can be computed by the equations

$$C_{s,L} = \min\left(C_{s,L,A}, C_{s,L,Z1}, C_{s,L,Z1}, C_{s,L,E}\right) \qquad \text{(veh/h)} \quad (18)$$

$$C_{s,T,left} = \min\left(C_{s,T,A}, C_{s,T,Z1,left}, C_{s,T,Z1^*,left}, C_{s,T,Z2,left}, C_{s,T,E,left}\right) \qquad \text{(veh/h)} \quad (19)$$

$$C_{s,T,right} = \min\left(C_{s,T,A}, C_{s,T,Z1,right}, C_{s,T,Z1^*,right}, C_{s,T,Z2,right}, C_{s,T,E,right}\right) \qquad \text{(veh/h)} \quad (20)$$

and

$$C_{s,E} = \min\left(C_{s,R,A}, C_{s,R,E}\right) \qquad \text{(veh/h)} \quad (21)$$

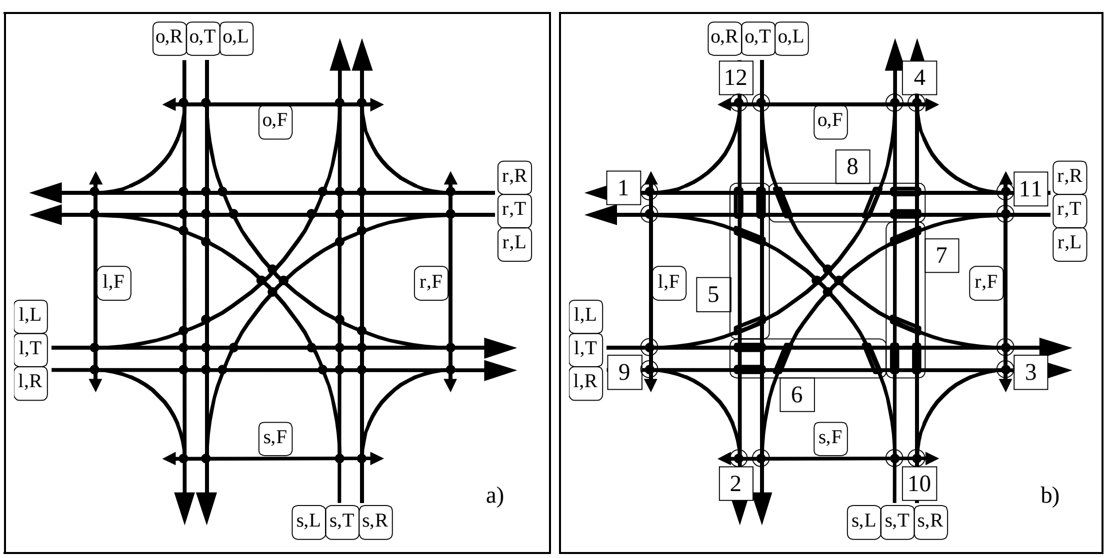

FIGURE 5 Intersection of two four-lane streets and the conflict-areas.

TABLE 2 Departure Sequences at AWSC Intersections of Two Four-Lane Streets

Subject stream	Involved departure sequences	Conflict streams
Left turn (s,L)	No. 1 No. 5 No. 6 No. 10_L	**s,L**-o,R-r,T_{left}-l,F **s,L**-o,$T_{max(left,right)}$-r,T_{left}-l,L **s,L**-o,$T_{max(left,right)}$-r,L-l,$T_{max(left,right)}$ **s,L**-s,F
Through ahead (s,T) on the left lane	No. 4 No. 7 No. 7* No. 8 No. 10_T	**s,T_{left}**-l,L-o,F **s,T_{left}**-o,L-r,L-l,T_{left} **s,T_{left}**-r,L-l,T_{right} **s,T_{left}**-o,L-r,$T_{max(left,right)}$-l,L **s,T_{left}**-s,F
Through ahead (s,T) on the right lane	No. 4 No. 7 No. 7* No. 8 No. 10_T	**s,T_{right}**-r,R-o,F **s,T_{right}**-o,L-r,L-l,T_{left} **s,T_{right}**-r,L-l,T_{right} **s,T_{right}**-o,L-r,$T_{max(left,right)}$ **s,T_{right}**-s,F
Right turn (s,R)	No. 3 No. 10_R	**s,R**-l,T_{right}-r,F **s,R**-s,F

The capacities of the traffic lanes at the approach are

$$C_{s,\,m,\,left} = \frac{Q_{s,L} + Q_{s,T,\,left}}{x_{s,L} + x_{s,T,\,left}} \text{ and } C_{s,\,m,\,right} = \frac{Q_{s,T,\,right} + Q_{s,R}}{x_{s,T,\,right} + x_{s,R}} \qquad \text{(veh/h)} \quad (22)$$

for the left and the right traffic lane (shared), respectively. These capacities are valid for the case of no flared areas at the approach.

The capacities of the traffic lanes have to be checked in accordance with the restrictions

$$(Q \cdot t_B)_{s,L} + (Q \cdot t_B)_{s,T,\,left} + (Q \cdot t_B)_{s,F} \le 3600 \qquad \text{(sec)} \quad (23)$$

$$(Q \cdot t_B)_{s,T,\,right} + (Q \cdot t_B)_{s,R} + (Q \cdot t_B)_{s,F} \le 3600 \qquad \text{(sec)} \quad (24)$$

The capacity of the approach is then

$$C_s = C_{s,\,m,\,left} + C_{s,m,\,right} \qquad \text{(veh/h)} \quad (25)$$

Certainly, the capacities at intersections of one two-lane street and one four-lane street or at T-junctions can be calculated similarly according to this procedure. The details are not included here for simplification.

5. QUEUE LENGTHS AND DELAYS

The queuing system at AWSC/FIFO intersections can be simplified as an M/G/1 queuing system. Nevertheless, it is recommended to use the M/M/1 queuing system especially if the queue lengths and the delays for shared traffic lanes are calculated.

The average delay of vehicles consists of two parts: (1) delay at the first position and (2) delay in the queue. The delay at the first position is equal to the service time (including move-up time from the second position to the first position), and it is equal to the reciprocal of the capacity. The delay in the queue is a function of the saturation degree, x. If at AWSC/FIFO intersections the queue of a shared lane is considered, the delay in the queue is a function of the saturation degree of the shared lane, x_m. According to this consideration, the delay of a stream, which uses a shared lane with other streams together, is

$$d_i = d_1 + d_2 = \frac{3600}{C_i} + d_2 \qquad \text{(sec)} \quad (26)$$

with

$$d_2 = \frac{3600 \cdot x_m \cdot k}{Q_m \cdot (1 - x_m)} \qquad \text{(sec)} \quad (27)$$

Q_m and x_m are the traffic flow rate and the degree of saturation of the shared traffic lane. Here, k is a parameter taking into account the stochastic property of the queuing system. For a M/M/1 queuing system is k = 1, for a M/D/1 queuing system is k = 0.5. For the queuing system at AWSC/FIFO intersections, k should be between 0.5 and 1. For non-stationary traffic conditions, the formula from the HCM can be applied for calculating delays in the queue. This formula is given as

$$d_2 = 900 \cdot T \cdot \left[x_m - 1 + \sqrt{(x_m - 1)^2 + \frac{\frac{3600}{C_m} \cdot x_m \cdot k}{450 \cdot T}} \right] \qquad \text{(sec)} \quad (28)$$

T is the length of the non-stationary peak period in (h). The queue length can be obtained in accordance with the rule of Littel in case of stationary conditions. In case of non-stationary (oversaturated) conditions the relationship $d_2 = N / x_m$ [Delay = Queue length/Degree of saturation, cf. Akcelik (1)] is applicable. The percentiles of the queue length can be estimated according to Wu (14).

6. SIMPLIFIED PROCEDURES

For SL approaches and SL approaches with separate left-turn traffic lanes the following simplified procedures can be recommended. The accommodations are valid for AWSC/FIFO intersections without flared approaches. The traffic flows of pedestrians are not taken into account.

6.1 Recommendations for Single-Lane Approaches

For simplification it is assumed that with sufficient precision the occupations times t_B are identical for all streams (i.e., $t_{B,L} = t_{B,T} = t_{B,R} = t_B$). Then the procedure for the calculation of capacity at AWSC/FIFO intersections can be simplified into the following form.

Capacity of the left-turn stream:

$$C_{s,L} = \max\begin{cases} \dfrac{3600}{t_B} - \max\left[\left(Q_{o,R} + Q_{r,T}\right), \left(Q_{o,T} + Q_{r,T} + Q_{l,L}\right), \left(Q_{o,T} + Q_{r,L} + Q_{l,T}\right)\right] \\ \dfrac{3600}{4 \cdot t_B} \end{cases}$$

(pcu/h) (29)

Capacity of the through-ahead stream:

$$C_{s,T} = \max\begin{cases} \dfrac{3600}{t_B} - \max\left[\left(Q_{r,R} + Q_{l,L}\right), \left(Q_{o,L} + Q_{r,L} + Q_{l,T}\right), \left(Q_{o,L} + Q_{r,T} + Q_{l,L}\right)\right] \\ \dfrac{3600}{4 \cdot t_B} \end{cases}$$

(pcu/h) (30)

Capacity of the right-turn stream:

$$C_{s,R} = \max\begin{cases} \dfrac{3600}{t_B} - \left(Q_{o,L} + Q_{l,T}\right) \\ \dfrac{3600}{3 \cdot t_B} \end{cases} \qquad \text{(pcu/h)} \quad (31)$$

Capacity of the approach:

$$C_s = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{x_{s,L} + x_{s,T} + x_{s,R}} \qquad \text{(pcu/h)} \quad (32)$$

The parameter t_B can be chosen between 3.5 s/pcu and 4 s/pcu. The calibration with field measurements (cf. the following section) delivers a value of t_B = 3.5 s/pcu for AWSC intersections. The traffic flow rates Q_L, Q_T, and Q_R should be converted into the unit of pcu/h in advance.

6.2 Recommendations for Approaches with Separate Left-Turn Traffic Lanes

Assuming a identical occupation time $t_{B,T+R}$ (i.e., $t_{B,T} = t_{B,R} = t_{B,T+R}$) for the through-ahead and right-turn streams and a different occupation time $t_{B,L}$ for left-turn streams, the procedure for calculating the capacity at AWSC/FIFO intersections with separate left-turn traffic lanes can be simplified into the following form.

Capacity of the left-turn stream:

$$C_{s,L} = \max\begin{cases} \frac{3600}{t_{B,L}} - f \cdot \max\left[\left(Q_{o,R} + Q_{r,T}\right), \left(Q_{o,T} + Q_{r,T} + \frac{Q_{l,L}}{f}\right), \left(Q_{o,T} + \frac{Q_{r,L}}{f} + Q_{l,T}\right) \right] \\ \frac{3600}{2 \cdot t_{B,L} \cdot (1+f)} \end{cases} \quad \text{(pcu/h)} \quad (33)$$

Capacity of the through-ahead stream:

$$C_{s,T} = \max\begin{cases} \frac{3600}{t_{B;T+R}} - \frac{1}{f}\max\left[\left(Q_{r,R} \cdot f + Q_{l,L}\right), \left(Q_{o,L} + Q_{r,L} + Q_{l,T} \cdot f\right), \left(Q_{o,L} + Q_{r,T} \cdot f + Q_{l,L}\right)\right] \\ \frac{3600}{2 \cdot t_{B,T+R} \cdot \left(1 + \frac{1}{f}\right)} \end{cases} \quad \text{(pcu/h)} \quad (34)$$

Capacity of the right-turn stream:

$$C_{s,R} = \max\begin{cases} \frac{3600}{t_{B,T+R}} - \left(\frac{Q_{o,L}}{f} + Q_{l,T}\right) \\ \frac{3600}{t_{B,T+R} \cdot \left(2 + \frac{1}{f}\right)} \end{cases} \quad \text{(pcu/h)} \quad (35)$$

Capacity of the approach:

$$C_{s,left} = Q_{s,L} \text{ and } C_{s,through+right} = \frac{Q_{s,T} + Q_{s,R}}{x_{s,T} + x_{s,R}} \quad \text{(pcu/h)} \quad (36)$$

where $f = t_{B,T+R}/t_{B,L}$.

$t_{B,L}$ should be chosen between 3.5 s/pcu and 4 s/pcu. The calibration with field measurements (cf. the following section) delivers the value $t_{B,L}$= 3.6 s/pcu. $t_{B,T+R}$ should be chosen between 4 s/pcu and 4.5 s/pcu. The calibration with field measurements delivers the value $t_{B,T+R}$ = 4.4 s/pcu for AWSC intersections. The traffic flow rates Q_L, Q_T, and Q_R should be converted into the unit of (pcu/h) in advance.

7. COMPARISON WITH FIELD MEASUREMENTS

In order to calibrate the new procedure, field data collected at real world AWSC intersections in the United States within NCHRP project 3-46 were employed. At 10 intersections 32 SL approaches and at 2 intersections 7 SL approaches with separate left-turn traffic lanes were measured. There are totally 203 data records related to approaches in case of SL approaches and 84 data records related to traffic lanes in case of SL approaches with separate left-turn lanes. The data are aggregated into 15-min intervals. At the measured AWSC intersections, delays and queue lengths instead of capacities were measured.

With these data, the new procedure was calibrated. The recommendations in Section 6 were used. A homogeneous parameter t_B for all streams was used (7) in case that all streams use the same traffic lane at the approaches. For left-turn streams with separate traffic lanes, a separate t_B value was applied. In addition, the predefined pcu equivalent factor of (a) 1 heavy truck = 2 pcu, (b) 1 light truck = 1.5 pcu, and (c) 1 motorcycle = 0.5 pcu and the delays formula according to HCM with T = 0.25 h are applied.

The mean occupation time $t_{B,L}$, $t_{B,T}$, $t_{B,R}$, and the factor for stochastic feature k were calibrated. From the calibration, no significant difference between the value $t_{B,L}$, $t_{B,T}$, and $t_{B,R}$ at SL approaches could be found. The calibration gave an optimal solution with k = 1 and $t_B = t_{B,L} = t_{B,T} = t_{B,R} = 3.5$ s for all streams at SL approaches. For SL approaches with separate left-turn traffic lane, the calibration gave an optimal solution with k = 1 and $t_{B,L} = 3.6$ s for the left-turn stream and $t_{B,T+R} = t_{B,T} = t_{B,R} = 4.4$ s for the through-ahead and right-turn stream. The parameter k = 1 indicates that the queuing system at AWSC intersections can be considered as an M/M/1 queuing system.

Figure 6 shows the comparison between the measured and the calculated delays at SL approaches. Figure 7 shows the comparison of queue lengths at SL approaches. Figure 8 shows the comparison between the measured and the calculated delays at SL approaches with separate left-turn lanes. Here the data for the left-turn lane and for the combined through-ahead and right-turn lane are depicted by different symbols.

The key parameters of the regression analysis of delays for the new procedure and two further procedures (7) are represented in Table 3. Here the procedure of TRC 373 is the procedure, which was incorporated into HCM 1994 (12). AWSIM is a simulation program

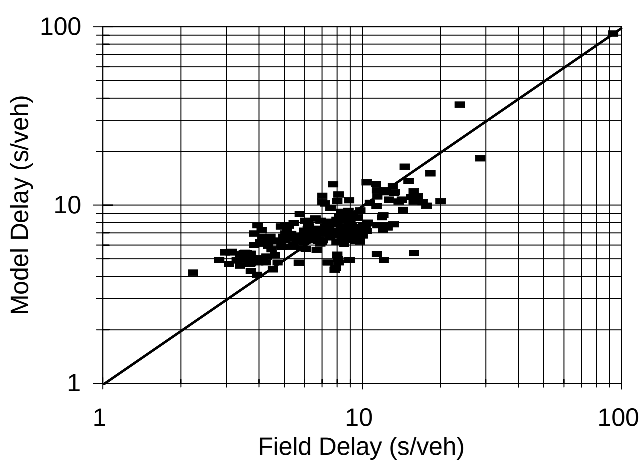

FIGURE 6 Average delays at SL approaches.

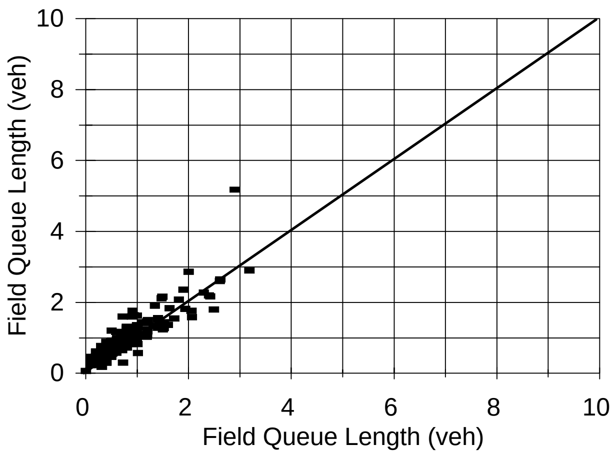

FIGURE 7 Average queue lengths at SL approaches.

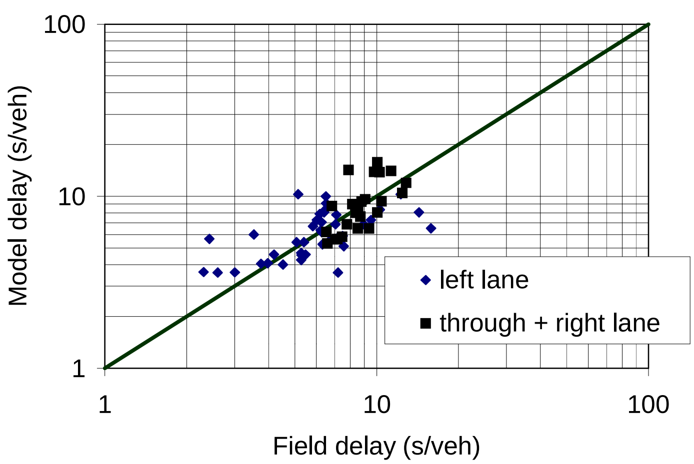

FIGURE 8 Average delays at SL approaches with left-turn traffic lanes.

developed by M. Kyte for AWSC intersections. The regression results of AWSIM and TRC 373 were taken from (7). It can be recognized that the new procedure describes the measured field data better than the other two procedures. It is to be mentioned that the same database (field measurements from NCHRP project 3-46) is used for the calibration of the new procedure as well as for the verification of the procedures of AWSIM and TRC 373.

Also for SL approaches with separate left-turn lanes the new procedure provides quite good results. Here, however, the very small value of B (=R^2) and the very large value of MAPE indicate that the range of the measured delays is very small (<13 s). Therefore, no appropriate regression can be performed. The relatively small standard deviation and MAE-value, however, show a very good result of the new procedure in the range of available field data.

TABLE 3 Results of Regressions

	Regression analysis			
Type of approaches	SL approaches			SL approaches with separate left-turn lanes
Type of models	AWSIM	TRC 373	New model t_B=3.5 s	New model $t_{B,L}$=3.6 s $t_{B,T+R}$=4.4 s
Constant factors	2.51	2.78	0.72	1.38
Standard errs Y	3.63	3.53	2.63	2.50
Certainties B	0.71	0.61	0.85	0.50
Number of measurements	203	203	203	84
Degree of freedom	201	201	201	82
X-coefficients	0.68	0.53	0.87	0.67
Std. errs of coeff.	0.03	0.02	0.03	0.07
MAE	2.4	3.1	2.04	1.94
MAPE	25.5%	31.5%	26.5%	48.9%

MAE = mean absolute error, MAPE = mean absolute percentage error.

Data for the calibration of mean occupation times at AWSC intersections within multi-lane streets are not available. The values of mean occupation times should be between 3.6 s and 4.4 s. As a first estimation, the value for the through-ahead/right-turn stream at SL approaches with separate left-turn traffic lane could be used. Furthermore, the average value of the right-turn and the through-ahead/right-turn stream could be applied. Thus, t_B = 4.4 s or t_B = (4.4 s + 3.6 s)/2 = 4 s for all streams is used for further calculations.

8. EXAMPLES FOR MAXIMUM CAPACITIES OF INTERSECTIONS

For different split of traffic volumes between the two streets and for different contributions of flows at the approaches (left stream, L; through stream, T; and right stream, R), capacities of the intersection are calculated according to the recommendations in Section 6 (15). For these examples, it is assumed (a) that no pedestrians are to be considered, (b) that the occupation times $t_{B,i}$ can be considered to be identical for all approaches at the intersection, and (c) there are no flared areas at the approaches for the right-turn streams.

The capacities can be calculated according to different points of view: (a) capacity of the subject approach and (b) total capacity of the intersection. Normally, the capacity of the subject approach is calculated such that the traffic flow rate at other (conflict and opposite) approaches are held constant (indicated in Table 4 as fixed). The corresponding total capacity of the intersection is in this case the sum of the capacity of the subject approach and the traffic flow rates at other approaches. The capacity of the subject approach can be calculated from the recommended procedures in Section 6 (15).

On the other side, the total capacity of the intersection usually is determined by increasing all the traffic flow rates at the intersection proportionally (indicated in Table 4 as float).

The capacity of the intersection can be obtained using the following steps: (1) definition of the distribution of the total flow at the intersection to the individual streams; (2) calculation of the capacities of the individual streams at the approaches; (3) calculation of the capacities of the shared lanes and then the capacities of the approaches and then the total capacity of the intersection; (4) postulate: total capacity of the intersection = total traffic flow rate of the intersection (i.e., degree of saturation x = 1); (5) calculation of the total traffic flow rate (i.e., the total capacity because x = 1) of the intersection as a function of the t_B values (15).

The calculation of the total capacities at the intersection are carried out for (a) split of flows by the two streets: 50/50, 70/30, and 100/0 (%) and (b) flow distributions at the approaches (L/T/R): 0.2/0.6/0.2 and 0.0/1.0/0.0. The share of trucks is assumed to be 5%.

In Table 4 the total capacities of the intersection from the new procedure and other existing sources are assembled together. It can be recognized that the total intersection capacities estimated by the new model do agree with the measured or/and simulated results from other sources. The new model is a very simple one compared to the other models, and it can handle much more complicate lane and traffic conditions.

TABLE 4 Comparison of Maximum Capacities of the Intersection

	Capacity of the approach or of the intersection C (veh/h) for trucks = 5%									
	SL approaches (two 2-lane streets)					SL approaches with separate left-turn lane (two 2-lane streets)			DL approaches (two 4-lane streets)	
	flow of other streams fixed (fixed)		flows of all streams at the intersection proportionally increased (float)						fixed	float
Source / street split	ex.2[3] HCM	ex.1[3]	50/50	70/30	100/0	50/50	70/30	100/0	ex.3[3]	50/50
available sources										
Herbert			1900	1500	-					
Richardson			1900	1560	1800					
Chan			1076	2419	-					
AWSIM[1]			2100	1800	1600					
AWSIM[2]			1700	1600	1400					
HCM (c.1)	1513									
new model for AWSC										
(t_B=4s)[2]						2040	1971	1886		
(t_B=3.5s)[1]			1960	1960	1960					
(t_B=3.5s)[2]	1546	1564	1881	1699	1470					
($t_{B,L}$=3.6s, $t_{B,T+R}$=4.4)[2]						1948	1896	1823		
(t_B=4.4s)[2]									1950	2026
(t_B=4s)[2]										2229

[1] Only through-ahead stream; [2] 20% left-turn, 60% straight ahead, and 20% right-turn; [3] See ref. 15.
(The underlined capacities are calculated according to the calibrated parameter, t_B.)

According to Table 4, the maximum capacities of AWSC intersections with SL approaches are between 1500 and 1900 veh/h. The maximum capacities of AWSC intersections with SL approaches and separate left turn lanes are between 1800 and 1950 veh/h. The capacities of AWSC intersections with DL approaches are about 2000 to 2200 veh/h for a 50/50 split. It is to recognize that the additional lanes (left turn) significantly affect capacity increases only for asymmetric street-flow-splits. According to the new procedure, the street-flow-split does not affect the total capacity of the intersection if only through-ahead streams at the approaches are considered.

9. SUMMARY

A procedure for the calculation of capacities at AWSC/FIFO intersections was presented. This procedure is based on the ACF method developed by GLEUE (4). The mathematical basis is the graph theory. Although the procedure seems relatively simple, it delivers more precise results compared to the measured field data including approaches with left-turn lanes. The sophistication of the model, which refers to all traffic streams at the intersection, allows a realistic analysis of the traffic process at AWSC/FIFO intersections. Therefore, the new procedure can handle most common lane configuration in the real world. The new procedure also considers the overloaded situation in which the capacities of the competing streams are distributed uniformly among each other.

A simple and practical procedure for the determination of capacity at AWSC/FIFO intersections is recommended (Section 6). This simplified procedure can be applied for SL approaches and SL approaches with separate left-turn traffic lanes. This procedure is verified and calibrated with measured field data at real AWSC intersections. For the calculation of capacities at AWSC intersections, general parameters based on field data calibration are proposed. For determining the capacity at FIFO intersection, the parameters of the new model have to be re-calibrated.

As a result, the total capacities of AWSC intersections with SL approaches are between 1500 and 1900 veh/h. The total capacities of AWSC intersections with SL approaches and separate left-turn lanes range between 1800 and 1950 veh/h. The total capacities of AWSC intersections with DL approaches are estimated as 2000 to 2200 veh/h. The additional lanes (left turn) significantly affect capacity increases only for asymmetric street-flow-splits.

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