DETERMINATION OF CAPACITY AT ALL-WAY STOP-CONTROLLED (AWSC) INTERSECTIONS

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Abstract

A new theoretical approach for the determination of capacities at All-Way Stop-Controlled intersections is presented. This approach is based on the Addition-Conflict-Flow method developed from the graph theory. The point of view related to the traffic streams - in contrast to existing procedures, which handle only the approaches - allows a systematic and realistic analysis of the traffic process at All-Way Stop-Controlled intersections. The new procedure can handle most common lane configurations in the real world.

A simple and more practical procedure is then recommended. In practice, this simplified procedure can be used for single-lane approaches as well as approaches with separate left-turn traffic lanes. This procedure is verified and calibrated with measured data. For the calculation of the capacity at All-Way Stop-Controlled intersections, general parameters, which are found by calibration, are proposed. The procedure produce results more precise than the existing ones compared to the measured data.

As a result, the maximum total capacities of All-Way Stop-Controlled intersections with single-lane approaches are found to be between 1500 and 1900 personal cars per hour. The total maximum capacities of All-Way Stop-Controlled intersections with single-lane approaches and separate left-turn lanes are between 1800 and 1950 personal cars per hour. Furthermore, it shows that the additional (left-turn) lanes significantly affect capacity increases only for asymmetric street-flow-splits.

The present procedure can easily be extended to All-Way Stop-Controlled intersections with multilane approaches.

Keywords: Capacity, unsignalized intersection, AWSC intersection, graph theory

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1 Introduction and concept

All-Way Stop-Controlled (AWSC) intersections are the most used road intersections in the United States and other countries in North America. Although many significant investigations for signalized and Two-Way Stop-Controlled (TWSC) intersections were already carried out, there is only a limited number of studies handling the traffic process at AWSC intersections. General analytical procedures for AWSC intersections exist /12//10/, however, it is not possible for these procedures to handle the much varied conditions in the real world.

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 intersections simultaneously. Combining with the procedure of shared and short lanes /13/, all traffic constellations at AWSC intersections can be handled. 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

- the number of lanes of the subject, the opposite, and the conflict approach,
- the distribution of traffic flow rate in the approaches,
- the number of pedestrians in the approaches,
- the flared approaches, and
- the interaction between the streams of the subject and the other approaches.

At intersections with single-lane (SL) approaches and single-lane approaches with separate left-turn traffic lane, this procedure was verified and calibrated by field data collected at real AWSC intersections in the United States within the NCHRP project 3-46.

2 Existing works

Hebert /6/ investigated the AWSC intersections in 1963. He measured some SL approaches at T-intersections in Chicago, and examined the average departure headways between vehicles for a) only a vehicle in the conflict approaches, and b) no vehicle in the conflict approaches is queued. The capacity is determined on the basis of these average departure headways. Hebert found that the distribution of the traffic flow on both streets (street-flow-split) affects the departure time headways and therefore also the capacity; the maximum capacity can be obtained by regular distribution of traffic flow on both streets (50/50 street-flow-split on both streets); the sharing of the left-turning vehicles has no influence on the capacity; and the capacity increases by 0.2 percent for each percent of right-turning vehicle.

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 in terms of the service time. The calculation of the service time was carried out in two different cases: a) the service time is 4.0 s if no vehicle is queued in the conflict approaches and b) the service time is 7.6 s if at least one vehicle is queued in the conflict approaches. The value 4.0 s and the value 7.6 s - 4.0 s = 3.6 s represent the occupation times (t_B) of the queuing system by vehicles at AWSC intersections for the cases a) and b). For determining the capacity the occupation time 3.6 s has to be used since in this case all vehicles must be queued in all approaches.

In the 1994 HCM /12/ an empirical approach is applied. The capacity and delay are determined by regression of field data /9//7/.

In the new 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 steps. Therefore, a calculation without computational aids is impossible.

All existing procedures concentrate on the traffic streams at the approaches. The interaction between the streams (left-turn, L; through-ahead, T; and right-turn, R) cannot or can only with very limited conditions be taken into account.

3 Departure mechanisms at AWSC intersections

3.1 Capacity of streams in a departure sequence

Since all streams at AWSC intersections are considered to be equal in the hierarchy of the priority of departure, the vehicles of different streams can (or have to) 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 stream i occupies the conflict area by a time of exactly $t_{B,i}$ seconds. In the case of intersection, which has only two streams, this corresponds to the rule of zipping. That means all streams must have the same capacity in a departure sequence if all traffic flows, Q_i, exceed their capacities, C_i, (total overload). That is, capacities of all streams in one departure sequence have under the overload condition the same value of

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

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

Considering the fictive stream configuration in Fig. 1 and searching for the capacity of the stream 3, C_3, the following equation can be obtained:

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

From the point of view of the subject stream whose capacity is to be determined, the distributed capacity, C, will be admitted also in the case that the traffic flow of this stream is lower than the distributed capacity, C. The argument of this consideration is that for estimating the capacity a traffic flow which is equal to or larger than the capacity has to be applied fictively. Stream 3, e.g., obtains also then the distributed capacity, C, if its traffic flow, Q_3, is lower than its capacity. That is, stream 3 always has the capacity of

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

If however some streams except the subject stream (here stream 3) cannot consume the admitted capacity, i.e., the traffic flows 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 Q_2), the capacity for stream 3 reads

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

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

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

can be obtained for the stream 3.

C" is always larger than C'. Analogously a capacity of

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

and

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

can be obtained in the case of Q_2<C and Q_1>C (partial overload for Q_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 the stream 3 in the case that no overload in the conflict streams (stream 1 and 2) occurs reads

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

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 service time $t_{B,i}$ (departure time headway) of stream i:

$$t_{B,i} = \frac{3600}{C_i} \qquad \text{[s]} \qquad (9)$$

It is clear that the capacities of the streams are not distributed proportionally to their traffic flow rates. In an overloaded departure sequence, the capacities of all streams are distributed uniformly. 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 it exits a partial-overloaded state between the states of overload and non-overload. In this state, the capacities of the overloaded streams are distributed uniformly among themselves. 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 the stream 3 in the partial-overloaded state are given by eqs.(4) and (6).

The determination of capacities in the of partial-overloaded state is in general a problem of optimization in sense of Operation Research since the capacities in overloaded state have to be distributed between the streams again and again. With the method of Linear Planing the capacities of every individual streams can be determined. Here the accurate formulation of this problem is renounced. The formulae derived for partial-overloaded state show the way of working for a simple problem with three streams (cf. eqs.(4) and (6)).

The partial-overloaded state is a transition 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. For the further derivations, this state is replaced by the non-overloaded state. The resulted deviations can be considered as very small. An insignificant under-estimate of capacity may be caused by this simplification in the partial-overloaded area. From the point of view of traffic performance this under-estimate lies in general on the safer side.

3.2 Capacity of streams in more than one departure sequences

The capacity of a stream in more than one departure sequences is the smallest capacity, which a stream can achieve in all of the departure sequences. That is (cf. Fig. 2):

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

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

3.3 Capacity of shared streams

The capacity of shared streams can be determined according to the procedure of Wu (1997). With this procedure, the length of shared/short lanes can also be taken into account. For the common case that all streams at an approach use the same shared lane the capacity of this shared lane, C_m, is given by

$$C_m = \frac{\sum Q_i}{\sum x_i} \qquad \text{[veh/h]} \quad (11)$$

where $x_i = Q_i/C_i$ is the degree of saturation of the stream i.

4 Intersections of two two-lane (single-lane approach) streets

Now an intersection of two streets at which there is only one traffic lane in all approaches is considered. First, for the derivation of the capacity formula, it is assumed that each turning movement has its own traffic lane at the intersection (cf. Fig. 3a). For this specific configuration the capacities of all streams can be derived.

For the further derivation, the following indices are used from the standpoint of the southern approach (at the bottom)

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 between exit and entrance, conflict at entrance

For thus an intersection, the critical conflict areas can be defined according to the graph theory (cf. Fig. 3b) /4/. The conflict areas can be distinguished according to the types of the conflict into

- Exit-conflicts (departure sequences No.1, 2, 3, 4)

- Between-conflicts (departure sequences No.5, 6, 7, 8), and
- Entrance-conflicts (departure sequences No.9, 10, 11, 12).

4.1 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. For the exit- and entrance-conflicts this explains itself since the streams involved have to pass the same point. For the between-conflicts it is not so clear according to Fig. 3. To clarify these 4 departure sequences the between-conflict areas are redrawn in Fig. 4 again with special respect to the departure sequences of the streams involved. All streams involved and conflict points are represented there.

A stream at AWSC intersections is mostly 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 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.

The streams from the subject approach (bottom) are involved in each case in different departure sequences. These departure sequences are described in Table 1. For each of these departure sequences, the capacity can be determined for the subject stream. The decisive capacity is the smallest one of all of the possible capacities within these departure sequences.

In general, the capacity of a stream i in a departure sequence with n streams reads

$$C_i = \max\begin{cases} \dfrac{3600 - \sum\limits_{j=1, j\neq i}^{n} (Q \cdot t_B)_j}{(t_B)_i} & \text{in case of Non-overload} \\ \dfrac{3600}{\sum\limits_{j=1}^{n} (t_B)_j} & \text{in case of Overload} \end{cases} \qquad \text{[veh/h]} \quad (12)$$

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

If a stream has priority to the other streams, the service time of this stream must be subtracted from the total time for the case of Overload. The general formula for the capacity of a stream i in a departure sequence with n streams within which m streams have the absolute priority reads then

$$C_i = \max\begin{cases} \dfrac{3600 - \sum\limits_{j=1, j\neq i}^{n} (Q \cdot t_B)_j}{(t_B)_i} & \text{in case of Non-Overload} \\ \dfrac{3600 - (Q \cdot t_B)_m}{\sum\limits_{j=1, j\neq m}^{n} (t_B)_j} & \text{in case of Overload} \end{cases} \qquad \text{[veh/h]} \quad (12)^*$$

In case of pedestrian streams, the number of pedestrian groups should be applied for Q instead of the absolute number of pedestrians. The determination of the number of the pedestrian groups as a function of the absolute number of pedestrians is not handled here, it is given elsewhere. In case of very weak pedestrian flow rates also the absolute number of 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 Fig. 3b). For the Exit-conflict (departure sequence No.1), the departure sequence reads: s,L-o,R-r,T-l,F. The pedestrians have always priority. The capacity for this sequence reads

$$C_{s,L,A} = \max \begin{cases} \dfrac{3600 - \left[(Q \cdot t_B)_{o,R} + (Q \cdot t_B)_{r,T} + (Q \cdot t_B)_{l,F} \right]}{(t_B)_{s,L}} \\ \dfrac{3600 - (Q \cdot t_B)_{l,F}}{(t_B)_{s,L} + (t_B)_{o,R} + (t_B)_{r,T}} \end{cases} \quad \text{[veh/h]} \quad (13)$$

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

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

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

$$C_{s,T} = \min\left(C_{s,T,A}, C_{s,T,Z1}, C_{s,T,Z2}, C_{s,T,E}\right) \quad \text{[veh/h]} \quad (15)$$

and

$$C_{s,E} = \min\left(C_{s,R,A}, C_{s,R,E}\right) \quad \text{[veh/h]} \quad (16)$$

4.2 Capacity of the approach

The approaches of an intersection consisting of two two-lane streets with only one traffic lane in each direction are now considered. Two cases can be distinguished: a) there is no separate traffic lane for the turning streams (cf. Fig. 5a) and b) there is a separate traffic lane for the left-turn streams (cf. Fig. 5b).

The capacity of the approach can be calculated by the formula for shared traffic lanes /13/.

In the case of no existing flared approach for the right-turn stream the capacity of the shared traffic lane reads

$$C_{s,m} = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{x_{s,L} + x_{s,T} + x_{s,R}} \quad \text{[veh/h]} \quad (17)$$

In the case of a existing flared approach for one right-turning vehicle the capacity of the shared traffic lane reads /13/

$$C_{s,m} = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{\sqrt{\left(x_{s,L} + x_{s,T}\right)^2 + \left(x_{s,R}\right)^2}} \quad \text{[veh/h]} \quad (18)$$

The capacity of the traffic lane has to be checked in accordance with

$$\left(Q \cdot t_B\right)_{s,L} \cdot + \left(Q \cdot t_B\right)_{s,T} + \left(Q \cdot t_B\right)_{s,R} + \left(Q \cdot t_B\right)_{s,F} \le 3600 \quad \text{[s]} \quad (19)$$

5 Queue lengths and delays

The queuing system at AWSC 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 for 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, 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 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.

According to this consideration, the delay of a stream, which uses a shared lane with other streams together, reads

$$\begin{aligned} d_i &= d_1 + d_2 \\ &= \frac{3600}{C_i} + d_2 \end{aligned} \qquad \text{[s]} \qquad (20)$$

with

$$d_2 = \frac{3600 \cdot x_m \cdot k}{Q_m \cdot (1 - x_m)} \qquad \text{[s]} \qquad (21)$$

where 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 the M/M/1 queuing system is k=1, for the M/D/1 queuing system is k=0.5. For the queuing system at AWSC intersections, k should be between 0.5 and 1.

For non-stationary traffic conditions, the formula from HCM can be applied for calculating delays in the queue. This formula reads

$$d_2 = 900 \cdot T \cdot \left[x_m - 1 + \sqrt{(x_m - 1)^2 + \frac{\frac{3600}{C_m} \cdot x_m \cdot k}{450 \cdot T}} \right] \qquad \text{[s]} \qquad (22)$$

T is the length of the non-stationary 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 (over-saturated) conditions the relationship $d_2 = N / x_m$ (Delay = Queue length / Degree of saturation, cf. Akcelik /1/) states. The percentiles of the queue length can be estimated according to the work of Wu /14/.

6 Recommendations

For SL approaches and SL approaches with separate left-turn traffic lanes at AWSC intersections the following recommendations can be presented. The accommodations are valid for intersections without flared approaches. The traffic flow of pedestrians is not taken into account.

6.1 Single-lane approaches

Assuming a unique occupation time t_B (i.e., $t_{B,L}=t_{B,T}=t_{B,R}=t_B$) for all streams the procedure for calculating the capacity at AWSC intersections can be simplified into the following forms.

Capacity of the left-turn stream:

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

[pcu/h] (23)

Capacity of the through-ahead stream:

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

[pcu/h] (24)

Capacity of the right-turn stream:

$$C_{s,R} = \max\begin{cases} \frac{3600}{t_B} - \left(Q_{o,L} + Q_{l,T}\right) \\ \frac{3600}{3 \cdot t_B} \end{cases}$$ [pcu/h] (25)

Capacity of the approach:

$$C_s = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{x_{s,L} + x_{s,T} + x_{s,R}}$$ [pcu/h] (26)

The parameter t_B can be chosen between 3.5 s/pcu and 4 s/pcu. t_B=3.6 s/pcu is a good choice. The calibration against field measurements (cf. the following section) delivers a value of t_B=3.5 s/pcu. The traffic flow rates Q_L, Q_T, and Q_R should be converted into the unit of [pcu/h] in advance.

6.2 Approaches with separate left-turn traffic lanes

Assuming a unique 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 separate occupation time $t_{B,L}$ for left-turn streams the procedure for calculating the capacity at AWSC intersections can be simplified into the following forms.

Capacity of the left-turn stream:

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

[pcu/h] (27)

Capacity of the through-ahead stream:

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

[pcu/h] (28)

Capacity of the right-turn stream:

$$C_{s,R} = \max\begin{cases} \dfrac{3600}{t_{B,T+R}} - \left(\dfrac{Q_{o,L}}{f} + Q_{l,T}\right) \\ \dfrac{3600}{t_{B,T+R} \cdot (2+\frac{1}{f})} \end{cases}$$ [pcu/h] (29)

Capacity of the approach:

$$C_{s,left} = Q_{s,L}$$ [pcu/h] (30)

$$C_{s,through+right} = \frac{Q_{s,T} + Q_{s,R}}{x_{s,T} + x_{s,R}}$$ [pcu/h] (31)

Where the parameter $f = t_{B,T+R}/t_{B,L}$. $t_{B,L}$ should be chosen between 3.5 s/pcu and 4 s/pcu. The calibration against 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 against field measurements delivers the value $t_{B,T+R}$= 4.4 s/pcu. 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 AWSC intersections in the United States within the 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 the 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.

Against these data the new procedure was calibrated. The recommendations in section 6 were used. A homogeneous parameter t_B for all streams was used (cf./7/) in case of 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 following parameters are set as default values:

- pcu equivalent factor: heavy truck = 2 pcu
  light truck = 1.5 pcu
  motorcycle = 0.5 pcu
- delays according to HCM with T=0.25 h
- parameters for calibration mean occupation time $t_{B,L}$, $t_{B,T}$, and $t_{B,R}$
  factor for stochastic feature k

From the calibration, no significant difference between the values $t_{B,L}$, $t_{B,T}$, 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 does can be considered as an M/M/1 queuing system.

Fig. 6 shows the comparison between the measured and the calculated delays at SL approaches. Fig. 7 shows the comparison of queue lengths at SL approaches. Fig. 8 shows the comparison between the measured and the calculated delays at SL approaches with separate left-turn traffic 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 statistical parameters of the regression analysis of delays for the new procedure and two further procedures /7/ are represented in Table 2. Here the procedure of TRC 373 is the procedure, which was incorporated into HCM 1994. AWSIM is a simulation program developed by M. Kyte for AWSC intersections. The regression results of AWSIM and TRC 373 were taken from the source /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 the NCHRP project 3-46) is used for the regression of the new procedure as for the procedures of AWSIM and TRC 373.

Also for SL approaches with separate left-turn traffic lanes the new procedure provides very good results. Here, however, the very small value of B (=R^2) and the very large value of MAPE indicate that the measured delays vary very weakly. Therefore, no appropriate regression can be carried out. 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.

8 Numerical examples for maximum capacities of intersections

For different street-flow-split of the two streets and for different flow-distributions for L, T and R at the approaches, maximum capacities of the intersection are calculated according to the recommendations in section 6 (cf. /15/). For the calculation 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 existing flared approaches for the right-turn streams.

The capacities can be calculated according to different points of view a) of capacity of the subject approach and b) of capacity of the intersection

Normally, the capacity of the subject approach is so calculated that the traffic flow rate at other (conflict ad opposite) approaches are held constant (fixed). The corresponding 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 (cf. /15/).

On the other side, the capacity of the intersection has to be calculated by increasing all the traffic flow rates at the intersection proportionally. The resulting capacity of the intersection indicates then the maximum throughput of the intersection. The capacity of the intersection can be obtained using the following work steps:

1. Definition of the distribution of the total traffic flow at the intersection to the individual streams. Normally, the street-flow-split (distribution of the total traffic flow rate to the both streets, e.g., 50/50 (%) or 70/30) and the flow distribution to the streams at an approach (e.g.,

.0.2/0.6/0.2 for L, T, and R) can be used. Therefore the traffic flow rate of any individual streams can be expressed as a function of the total traffic flow rate of the intersection.

2. Calculation of the capacities of the individual streams at the approaches. According to the new procedure, the capacities are - for predefined t_B values - only functions of the traffic flow rates at the approaches and, therefore, only functions of the total traffic flow rate of the intersection (cf. Step 1).
3. Calculation of the capacities of the shared lanes and then the capacities of the approaches. The sum of the capacities of all approaches is the total capacity of the intersection. The total capacity of the intersection is now again a function of the total traffic flow rate of the intersection
4. Postulation: total capacity of the intersection = total traffic flow rate of the intersection (i.e., degree of saturation x = 1). Then, an expression, which contains only the total traffic flow rate of the intersection as a parameter, can be achieved. In general, this expression is a polynom of first to third orders.
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.

The calculation of maximum capacities are carried out for

- street-flow-splits: 50/50, 70/30, and 100/0 (%)
- flow distributions: 0.2/0.6/0.2 and 0.0/1.0/0.0

In Table 3, the maximum 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 deal with much more complicate lane and traffic conditions.

According to the Table 3, the maximum capacities of AWSC intersections with SL approaches are between 1500 and 1900 pcu/h. The maximum capacities of AWSC intersections with SL approaches and separate left-turn lanes are between 1800 and 1950 pcu/h. It is to recognize that the additional lanes (left-turn) significantly affect capacities 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 Summaries

A procedure for the calculation of capacities at AWSC 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 amazingly precise results compared to the measured field data.

The point of view related to the traffic streams - in contrast to the existing procedures, which only handle the approaches - allows a realistic analysis of the traffic process at AWSC 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 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, which are found by calibration, are proposed. For special intersections with extremely unsymmetrical configurations the parameters must be adapted correspondingly.

As a result, the maximum capacities of AWSC intersections with SL approaches are between 1500 and 1900 pcu/h. The maximum capacities of AWSC intersections with SL approaches and separate left-turn lanes are between 1800 and 1950 pcu/h. The additional lanes (left-turn) significantly affect capacities existing only for asymmetric street- flow-splits.

The present procedure can easily be extended to intersections with multilane approaches /15/.

The new procedure can also be adopted to intersections with “First-In-First-Out” and "Off Side Priority" regulations which are broadly used elsewhere in the world.

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Legends to tables and figures

Table 1-Departure sequences of two two-lane streets at AWSC intersections
(cf. Fig. 3b)

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

Table 2-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

Table 3-Comparison of maximum capacities of the intersection from different models

	Capacity of the approach or of the intersection C [pcu/h]							
	SL approaches)					SL approaches with separate left-turn lanes		
	flow of other streams fixed		flows of all streams at the intersection proportionally increased					
Source / street-flow-split	HCM example 2[3]	example 1[3]	50/50	70/30	100/0	50/50	70/30	100/0
available sources								
Herbert			**1900**	**1500**	-			
Richardson			**1900**	**1560**	1800			
Chan			1076	2419	-			
AWSIM[1]			**2100**	**1800**	**1600**			
AWSIM[2]			**1700**	**1600**	**1400**			
HCM, concept 1	**1513**							
new model								
new model[1] (t_B=4s)			1714	1714	1714			
new model[2] (t_B=4s)	1288	1438	1646	1486	1286	**2040**	**1971**	**1886**
new model[1] (t_B=3.6s)			**1905**	**1905**	**1905**			
new model[2] (t_B=3.6s)	**1520**		**1829**	**1650**	**1429**	**2267**	**2190**	**2096**
new model[1] (t_B=3.5s)			**1960**	**1960**	**1960**			
new model[2] (t_B=3.5s)	**1546**	**1564**	**1881**	**1699**	**1470**	**2332**	**2254**	**2157**
new model[2] ($t_{B,L}$=3.5s, $t_{B,T+R}$=4.4)						**1948**	**1896**	**1823**

[1] Only through-ahead stream [2] 20% left-turn, 60% straight ahead, and 20% right-turn [3] See /15/

(The underlined capacities are calculated according to the calibrated parameter, t_B)

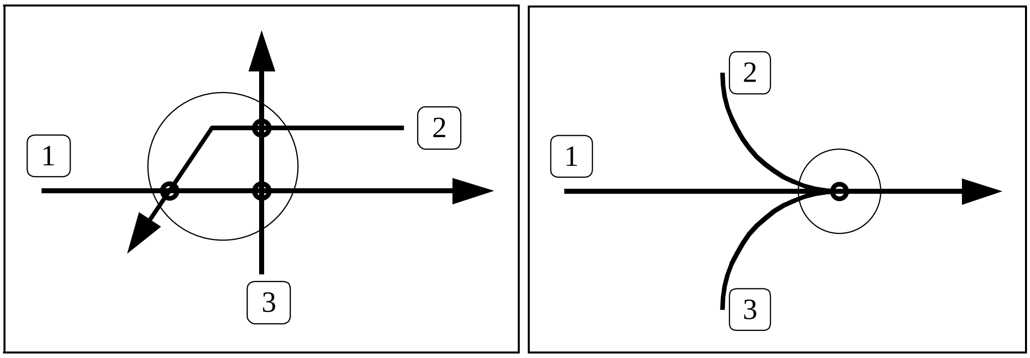

Fig. 1-Three streams in a departure sequence

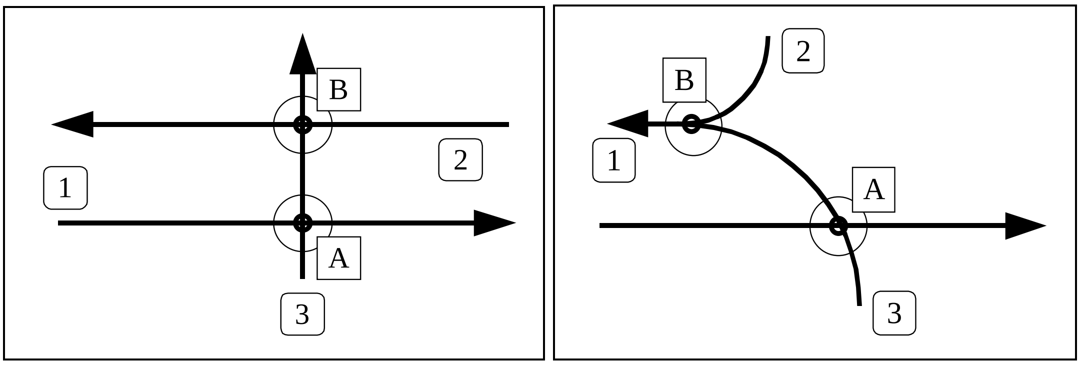

Fig. 2-A stream in several departure sequences

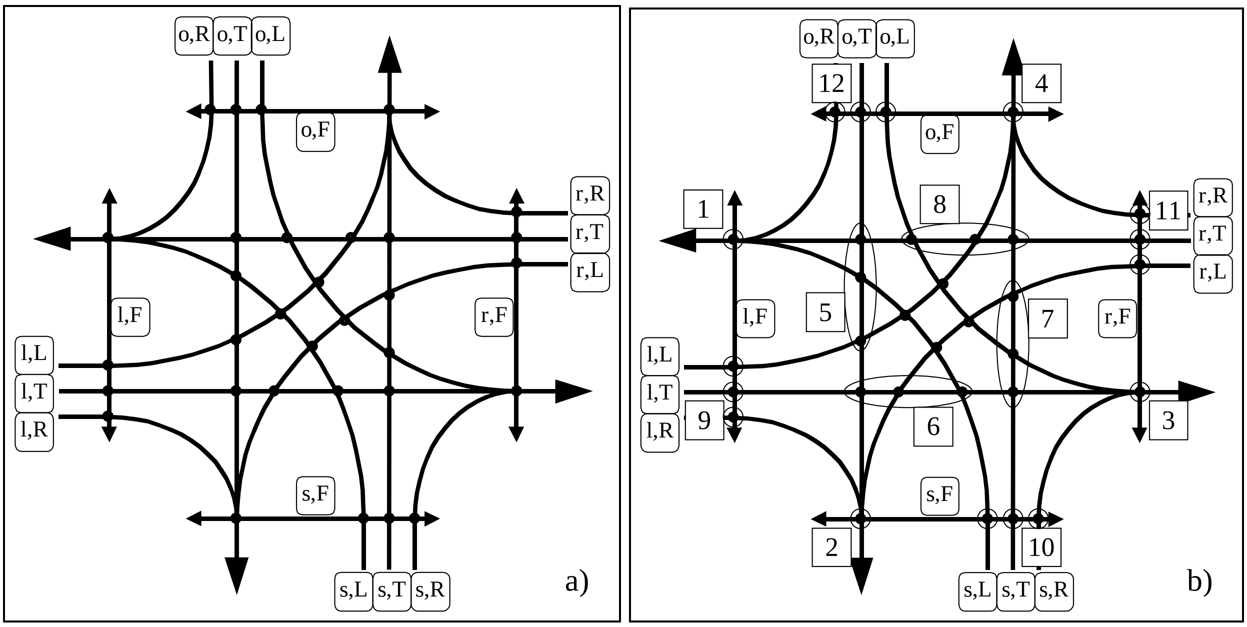

Fig. 3-Intersection with 12 vehicle- and 4 pedestrian streams and the critical conflict areas between the streams

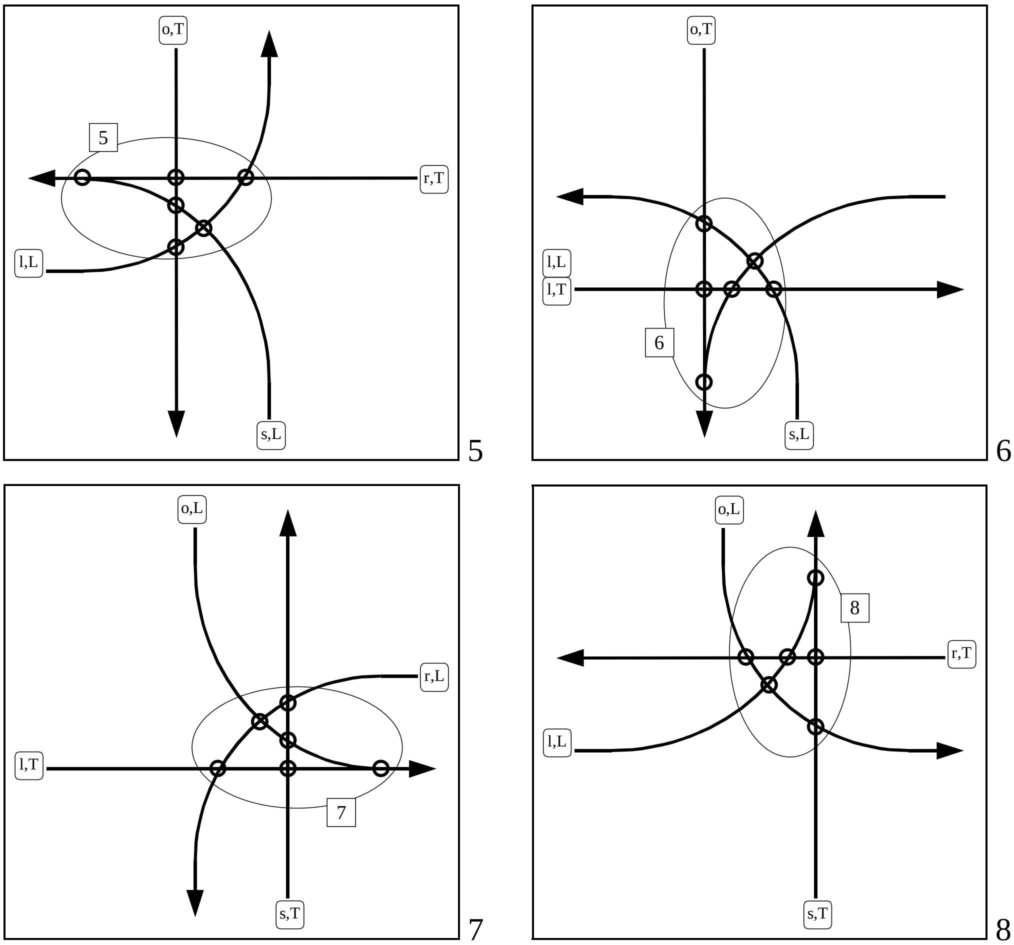

Fig. 4-Departure sequences of the conflict areas 5, 6, 7, and 8

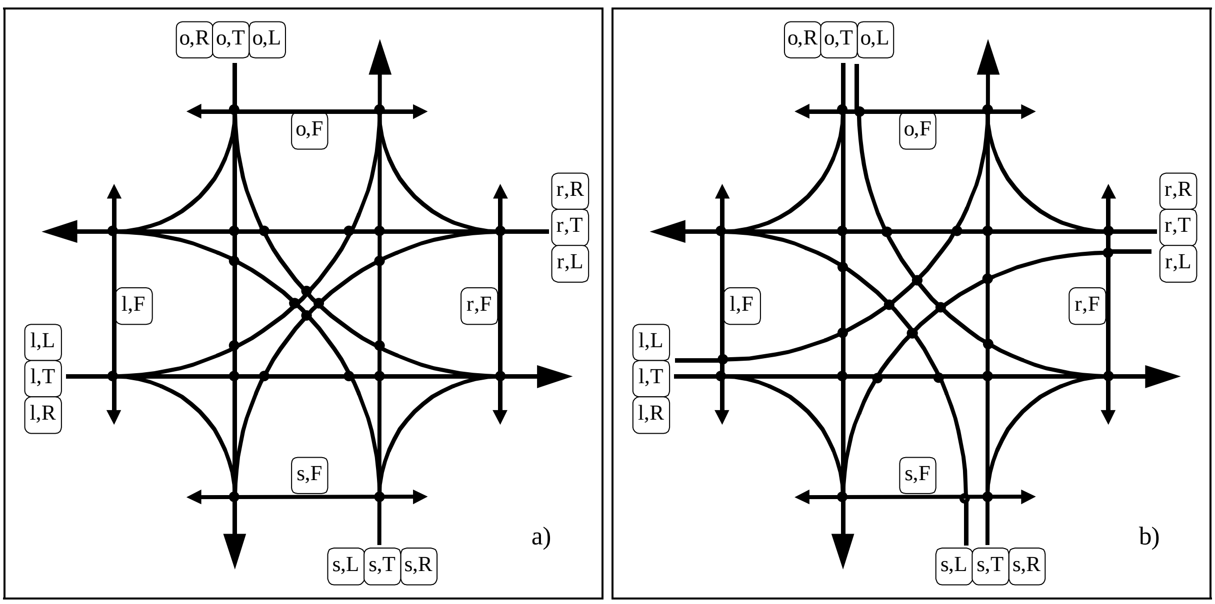

Fig. 5 - Lane distribution at approaches of a intersection of two two-lane streets

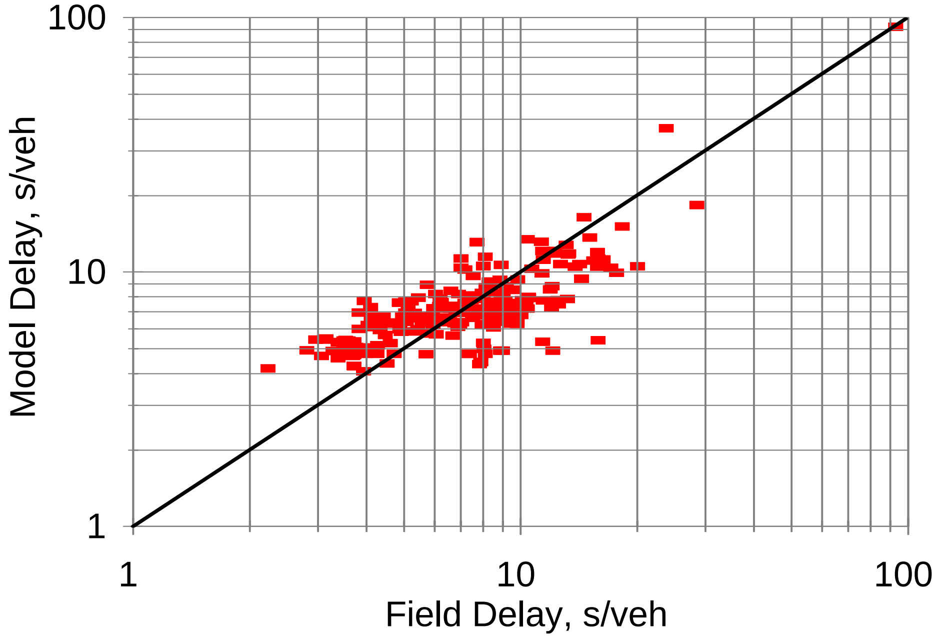

Fig. 6-Comparison of delays at SL approaches (veh=pcu)

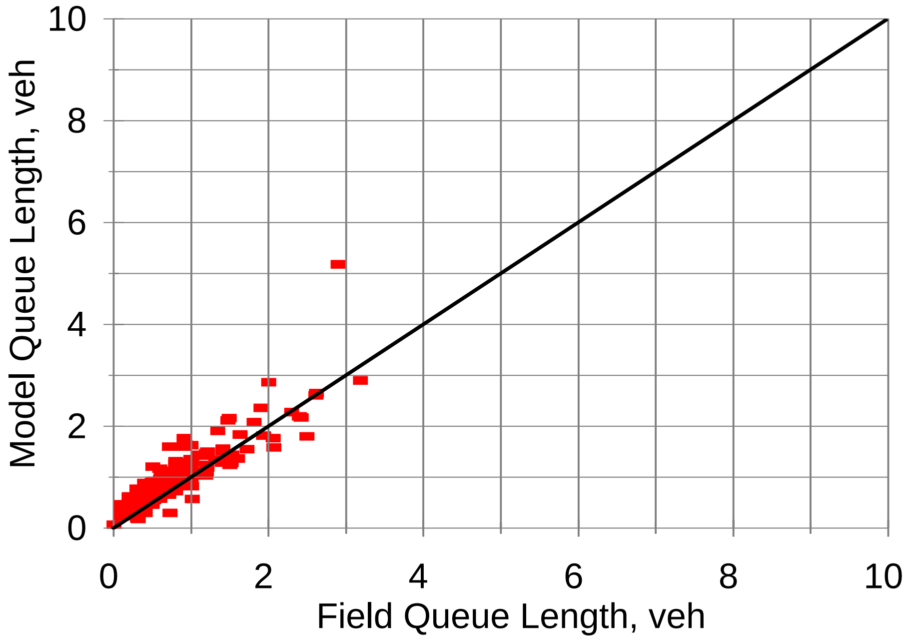

Fig. 7-Comparison of queue lengths at SL approaches (veh=pcu)

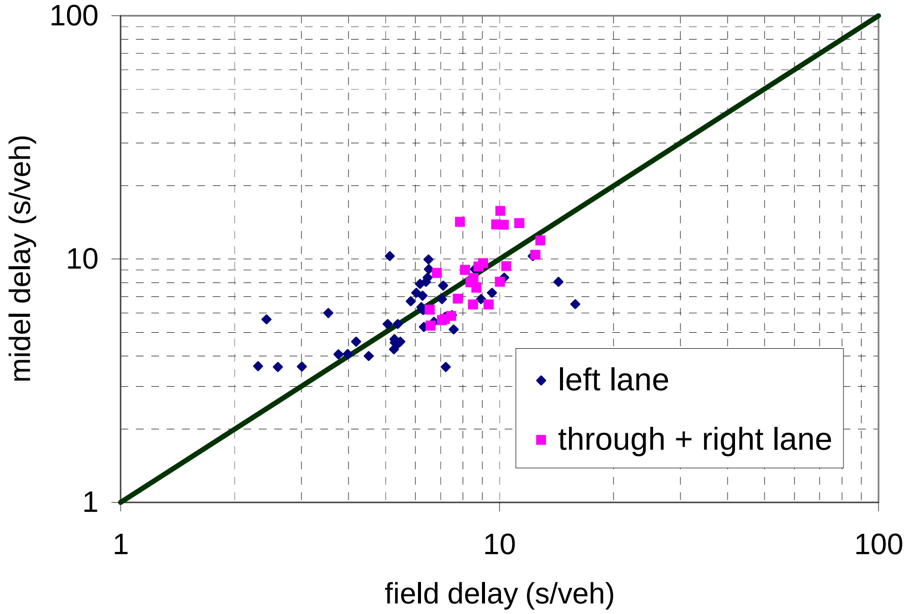

Fig. 8-Comparison of delays at SL approaches with left-turn traffic lanes (veh=pcu)