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NEW MEASUREMENTS FOR SATURATION HEADWAYS AND CRITICAL GAPS AT STOP-CONTROLLED INTERSECTIONS

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ABSTRACT

In January 1993, a comprehensive study of traffic flow characteristics at two-way and all-way stop controlled intersections was initiated in the United States through the National Cooperative Highway Research Program. The primary motivation for this study was the need for a new set of capacity analysis procedures for stop-controlled intersections that were based on a comprehensive U.S. data base. Phase I of this study has now been completed. During Phase I, traffic operations at seventeen stop-controlled intersections (twelve two-way stop-controlled intersections and five all-way stop-controlled intersections) were videotaped.

This paper focuses on the saturation headway and critical gap data that have been collected as part of this study. For TWSC intersections, both critical gap and follow-up time estimates were made using both the Siegloch and maximum likelihood methods. For AWSC intersections, saturation headway estimates were made and compared with earlier results from Transportation Research Circular 373.

1. BACKGROUND AND INTRODUCTION

The basic parameter used to estimate capacity at a signalized intersection is saturation headway. Ideal saturation headway is the difference in the passage time at the intersection stop line between two consecutive vehicles once the queue is moving in a stable manner. The 1985 Highway Capacity Manual (HCM) notes that the saturation headway is "estimated as the constant average headway between vehicles which occurs after the 6th vehicle in the queue and continues until the last vehicle in the queue clears that intersection." Field measurements must consider the start up lost time, or that time at the beginning of the green phase that is required for the queue to begin to move. The capacity procedures given in Chapter Nine of the HCM provide a standard value for the ideal saturation headway of 2.0 seconds per vehicle, which yields an ideal saturation flow rate of 1800 vehicles per hour of green. The procedure provides adjustments to this ideal value to consider the effects of intersection geometry, opposing traffic flow, signal timing parameters, and pedestrian flows.

The capacity analysis procedure for unsignalized intersections is given in Chapter Ten of the HCM. A new version of Chapter Ten is planned for release in 1994, with an improved procedure for two-way stop-controlled (TWSC) intersections based on a capacity methodology developed by Siegloch. The chapter also includes a procedure for estimating the capacity of an all-way stop-controlled (AWSC) intersection based on Transportation Research Circular 373.

Both of the capacity procedures for stop-controlled intersections use the concept of saturation headway. The TWSC intersection procedure is defined in terms of the critical gap and the follow-up time. The critical gap is the minimum time gap in the major traffic stream needed by a minor stream vehicle to merge into or travel through the major stream. The follow-up time is the minimum headway between the first vehicle and the second vehicle, and subsequent vehicle pairs, as they enter the same major stream gap, when a continuous queue exists on the minor street approach. In effect, the follow-up time is the saturation headway for the minor traffic stream when the conflicting major stream flow is zero. Table 10-2 in the new version of Chapter Ten gives critical gaps ranging from 5.0 seconds for major stream left turning traffic to 6.5 seconds for minor stream left turning traffic. Follow-up times range from 2.1 seconds for left turning traffic from the major street to 3.4 seconds for left turning minor stream traffic. The capacity on the minor stream approach, based on Siegloch's work, is a function of the major stream flow rate (v_c), the critical gap (t_g), and the follow up time (t_f). The capacity equation is given in *Equation (1)*.

$$c_p = \frac{3600}{t_f} e^{-v_c t_0 / 3600} \tag{1}$$

One of the problems with this procedure, however, is that it has not been validated with data collected from sites within the United States. Data contained in the HCM Table 10-2 were measured first in Germany, and then slightly modified based on studies of critical gap for a very limited number of sites in the United States. None of these U.S. studies attempted to measure the follow-up time. Further, it was assumed that a fixed relationship existed between the critical gap and the follow-up time given in *Equation (2)*.

$$t_g = 0.6\ t_f \tag{2}$$

A further complication is the inherent difficulty in the measurement of the critical gap. The HCM defines the critical gap as the median time headway between two successive vehicles in the major street traffic stream that is accepted by drivers in a subject movement that must cross and/or merge with the major street flow. Several researchers (for example, Kittelson and Vandehey) have pointed out the difficulty in using this definition. In fact, the formulation of the Siegloch equation is based on a very specific description of the gap acceptance process that may yield estimates of the critical gap that are different than those produced by the definition given in the HCM. According to the Siegloch formulation, one vehicle will accept a major stream gap that is greater than the critical gap, but less than the sum of the critical gap and the follow-up time. Two vehicles will use a gap that is greater than the sum of the critical gap and the follow-up time, but less than the sum of the critical gap and twice the follow-up time. In order to measure the critical gap in this way, a continuous minor stream queue is required. Brilon, Troutbeck, and Tracz recommend the use of either the maximum likelihood technique or Ashworth's method if a continuous queue is not present on the minor street approach.

The AWSC intersection capacity procedure is based on a set of four saturation headways, each defined according to the conditions faced by the subject approach driver. Table 10-5 in the new version of Chapter Ten gives values of 3.5 seconds per vehicle when the subject vehicle is faced with neither opposing nor conflicting stream vehicles and 9.0 seconds per vehicle when the subject vehicle is faced with both opposing and conflicting approach vehicles. *Table 1* lists the saturation headway from Table 10-5 of the new version of Chapter Ten.

The capacity of an approach is based on the mix of traffic conditions faced by the subject approach driver and is defined in terms of the volume proportions of each of the intersection approaches. The capacity of an approach varies from 1100 vehicles per hour when the subject driver faces no opposing or conflicting vehicles to 525 vehicles per hour when the subject driver faces a continuous queue of vehicles on both the opposing and conflicting approaches.

The four headway cases listed in *Table 1* do not directly consider the effects of turning traffic. The Case 2 headway, which is a subject vehicle faced by an opposing vehicle and no conflicting vehicles, does not consider the effects of the interaction of one or both of the vehicles turning and not traveling straight through the intersection. The value of 5.5 seconds given in *Table 1* is assumed to cover the range of combinations that actually make up Case 2: for example, pairs of through vehicles with no turning conflicts, one through vehicle opposed by a left turning vehicle, one through vehicle opposing by a right turning vehicle, etc. While the capacity equation given in the new version of Chapter Ten does provide an adjustment for turning movements, it is based only on the overall proportions of turning movements and not on the microscopic or vehicle by vehicle interactions that actually reflect the impedance resulting from turning vehicle conflicts.

TABLE 1

Saturation headway data for AWSC intersections

Condition	Mean Saturation Headway, sec/veh			
	Case 1	Case 2	Case 3	Case 4
All data	3.5	5.5	6.5	9.0
Single lane approach sample sites	3.9	5.6	6.5	9.0
Multi lane approach sample sites	1.5	4.3	6.3	9.3

2. STUDY METHODOLOGY

In January 1993, a study was initiated as part of the U.S. National Cooperative Highway Research Program to develop a new set of capacity and level of service procedures for stop-controlled intersections. The objective of the first phase of the study, completed in December 1993, was to conduct a pilot study designed to develop and test the methodologies that would be used in the larger second phase of the project. The pilot study included seventeen intersections, twelve of which are TWSC intersections and five of which are AWSC intersections. At each intersection, videotapes were made with two perspectives: one showing the intersection box and the other showing the queue activity on at least one stop controlled approach.

The data were extracted from the videotapes through the identification of all key events from the videotape and the recording of the time that each of these events occurred. Several events were defined that were noted during the data extraction phase. *Passage time* is the time that each vehicle passes through the conflict point. The conflict point is defined as the center point in the intersection that all through vehicles and left turn vehicles will most likely pass through. An equivalent point is also identified for right turning vehicles. In addition to the passage time, the directional movement, the vehicle type, and the lane usage are recorded. Vehicle type includes passenger car, light truck, heavy truck, recreational vehicle, bus, and motorcycle. Lane usage is recorded from the inner or left most lane for an approach, beginning with 1, next lane as 2, etc. Directional movement is noted by absolute direction (NB, SB, EB, and WB) and turning movement (LT, TH, RT). *Queue events* were recorded for each subject vehicle, including the time that a vehicle becomes part of the queue, the time that the vehicle arrives at the stop line, and the time that the vehicle passed into the intersection. *Blockage times* were noted as the beginning and ending times of each blockage event, due to bicycle or pedestrian crossings. Other events, such as the blockage of vehicles due to the presence of emergency vehicles, were also noted.

Events were recorded using the Traffic Data Input Program, PC-based software that provides an efficient method for noting important vehicle movement events. While observing the videotape, the user presses the appropriate key on the personal computer keyboard corresponding to each event of interest. The keypress records the time stamp of the event in a data file.

3. TWSC INTERSECTIONS

3.1 Site Characteristics

The twelve TWSC sites include a range of intersection geometry conditions. Five of the sites are three leg intersections, while seven of the sites are four leg intersections. Nine of the sites have single lane approaches, while three are multi-lane intersections on either the major street or the minor street. The data that were compiled from the raw event files include 324 5-minute data points. This includes:

- 187 data points for single lane sites with 4 legs,
- 28 data points for single lane sites with 3 legs,
- 48 points for large radius subject approach with 3 legs,
- 30 data points for multi-lane sites with 3 legs, and
- 31 data points for multi-lane sites with 4 legs.

Continuous queueing existed during 16 of the 324 five-minute periods. There were a total of 4,987 subject approach vehicles.

There was a wide range of flow rate conditions. Based on the five-minute data:

- The intersection flow rates ranged from a low of 732 veh/hr to a high of 2160 veh/hr.
- The major street flow rates ranged from 372 veh/hr to 1968 veh/hr.
- The subject approach flow rate ranged from 12 veh/hr to 600 veh/hr.
- The average delay per vehicle varied from 3.0 sec/veh to 99.7 sec/veh.

3.2 Estimation of Critical Gap

The critical gap and the follow-up time are two key parameters that are used in gap acceptance based models to estimate the capacity of a TWSC intersection. The critical gap is defined as the minimum interval between two successive vehicles in the major traffic stream that allows intersection entry to one minor stream vehicle. When more than one minor stream vehicle uses one major stream gap, the headways in the minor stream called follow-up times. In practice, the measurement of each of these variables is difficult. Two methods, one proposed by Siegloch and one based on maximum likelihood methods, are used here to estimate values for the critical gap.

The method developed by Siegloch provides a direct link between gap acceptance theory and the definitions of these parameters. In this method, both the size of the major stream gap and the number of minor stream vehicles using each major stream gap during periods of continuous queueing are recorded. The mean gap size used by n vehicles is computed and is plotted against n. The resulting regression line that best fits these points is used to calculate the critical gap and the follow up time. The data points in *Figure 1* show the size of the gap used by given numbers of vehicles. The mean gap size for each vehicle group is shown in *Figure 1* and in *Table 2*. The regression line is the least squares line through these mean gap points. The x-intercept is the zero gap, or t_0. The slope of the regression line is the reciprocal of the follow-up time. The critical gap is the zero gap plus one-half of the follow-up time.

The maximum likelihood method of estimating the critical gap is based on the fact that a driver's critical gap is both greater than the largest rejected gap and smaller than the accepted gap. A probability distribution is assumed for the critical gap; here both a log-normal and a Pearson distribution were used. The method uses the accepted gap and maximum rejected gap for each subject vehicle, with each measured gap setting an upper and lower bound on the estimated critical gap, respectively. The method converges to a likely estimate of the critical gap.

Table 3 presents the results of the critical gaps estimated using both Siegloch's method and the maximum likelihood estimate method. The results in the table are presented according to the geometric configuration and major street speed of the intersection so that reasonable comparisons are possible. Two estimates using Siegloch's method are given. The first estimate is based on counting only the subject approach vehicles that use each major stream gap; the second estimate includes all minor (i.e. non-priority) vehicles that use each gap. The second estimate yields more accurate results since it does reflect the total number of vehicles that use each gap. Two estimates using the maximum likelihood method are also given. The first is based on an assumption that the critical gap probability density function is log-normal; the second assumes a Pearson distribution. Both results are similar. The table also shows the range of values for the critical gap from the HCM.

Table 4 presents the results of the critical gaps estimates using the maximum likelihood method (assuming a log normal distribution) for each turning movement group of the subject approach vehicle. Again the data are segregated according to the geometric configuration and major street speed of the intersection. *Figure 2* compares the estimates generated by the maximum likelihood method and the Siegloch method for the four-leg approach sites, with major street speeds less than 40 mi/hr. The maximum likelihood method provides much more stable estimates (with a range from 5 to 6 seconds), while there is considerable variation in the Siegloch estimates (variation from 4 to 10 seconds).

After a review of this information, the following conclusions can be drawn:

- The two maximum likelihood methods yield nearly equivalent critical gap estimates.
- The maximum likelihood methods yield much more stable results than the Siegloch methods.
- In general, the turning movement of the subject approach vehicle affects the size of the critical gap. The critical gap for left turning vehicles is usually higher than for through vehicles, which in turn is usually higher than for right turning vehicles.
- The HCM critical gap values for lower speed sites are in the range of the values estimated by the Siegloch method; the maximum likelihood method produces values that are equal to or lower than the HCM values.

■The HCM critical gap values for the higher speed sites are significantly higher (by about 2 seconds) than those estimated by either the Siegloch method or the maximum likelihood method.
■There does not seem to be a difference in the critical gap estimates made by any of the methods considered here as a function of the speed of the major street.

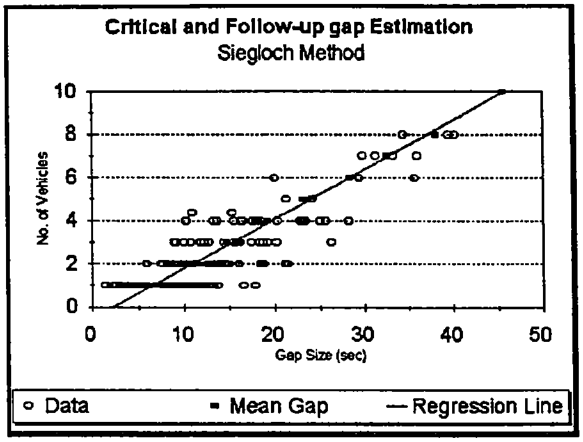

Fig. 1 - Siegloch's method for gap estimation

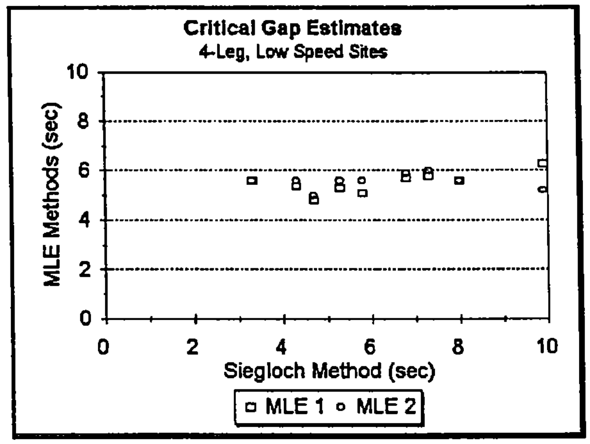

Fig. 2 - Critical gap estimates

TABLE 2

Critical gap and follow-up time estimation using Siegloch method

Number of Vehicles Using Gap	Mean Gap Size (sec)	Sample Size
1	6.8	105
2	11.8	52
3	14.7	27
4	19.2	23
5	23.1	4
6	28.4	3
7	32.6	4
8	38.0	3
9	-	-
10	45.7	1

t_0 (zero gap) = 2.15 sec; t_c (critical gap) = 4.34 sec; t_f (follow-up time) = 4.37 sec

TABLE 3

Critical gap results

Site	Siegloch Method		Maximum Likelihood Method		Obs	HCM Values
	Sub Veh Only	All Minor Veh	Method 1	Method 2		
3Leg/LowSpeed						
T003	-	-	5.2	5.6	138	
T008	8.0	6.6	6.9	6.8	189	5.5-7.0
T010	5.5	6.7	5.9	5.6	197	
T012	-	4.5	3.9	4.2	339	
3Leg/High Speed						
T005	4.2	4.9	4.8	4.9	291	6.5-8.5
T007	3.9	2.6	5.1	5.0	618	
T011	3.3	3.0	3.6	4.6	449	
4Leg/Low Speed						
T001-1	-	-	4.0	4.6	165	
T001-2	-	-	3.9	4.0	87	
T002-1WB	-	6.8	5.7	5.9	206	
T002-1EB	-	7.3	5.8	6.0	147	
T002-2WB	-	5.3	5.3	5.6	234	
T002-2EB	-	8.0	5.6	5.6	148	5.5-7.0
T006	4.3	4.3	5.4	5.6	646	
T009-1NB	-	4.7	4.8	5.0	338	
T009-1SB	-	5.8	5.1	5.6	184	
T009-2NB	-	9.9	6.3	5.2	223	
T009-2SB	-	3.3	5.6	5.6	124	
4Leg/High Speed						
T004-1EB		5.5	5.1	5.1	143	
T004-1WB	-	-	5.3	5.6	215	6.5-8.5
T004-2EB	-	8.1	5.3	5.3	88	
T004-2WB		-	5.7	5.7	130	

Notes:

1. Siegloch values were estimated using either only subject approach vehicles (noted by *Sub Veh Only*) or all non-priority vehicles (noted by *All Minor Veh*).
2. *Obs* is the number of observation.
3. The maximum likelihood methods 1 and 2 are based on the log-normal and Pearson distributions, respectively.

TABLE 4

Critical gap estimates using maximum likelihood method

Site	LT	TH	RT	Obs
3Leg/LowSpeed				
T003	6.0	-	4.3	138
T008	6.9	-	-	189
T010	6.5	-	4.5	197
T012	4.6	-	3.7	339
HCM	6.5-7.0	-	5.5	
3Leg/High Speed				
T005	6.3	-	3.4	291
T007	6.0	-	3.4	618
T011	3.7	-	3.7	449
HCM	8.0-8.5	-	6.5	
4Leg/Low Speed				
T001-1	4.9	-	3.7	165
T001-2	4.6	-	3.7	87
T002-1WB	7.1	6.9	4.3	206
T002-1EB	7.4	6.7	4.0	147
T002-2WB	6.8	5.7	4.3	234
T002-2EB	6.2	6.9	4.2	148
T006	5.6	5.0	3.8	646
T009-1NB	5.5	5.7	3.7	338
T009-1SB	5.2	5.6	3.3	184
T009-2NB	6.0	6.6	-	223
T009-2SB	5.4	6.0	3.4	124
HCM	6.5-7.0	6.0-6.5	5.5	
4Leg/High Speed				
T004-1EB	6.0	6.1	4.3	143
T004-1WB				215
T004-2EB	6.5	6.2	3.7	88
T004-2WB				130
HCM	8.0-8.5	7.5-8.0	6.5	

Notes:
The directional movements of the subject approach vehicle are denoted by *LT*, *TH*, and *RT*. *Obs* is the number of observations.

3.3. Estimation of Follow-up Time

The follow-up time was computed for each site using three methods. The first two methods were based on Siegloch's method described earlier in the computation of the critical gap. The third method directly computed the follow-up time from the headways that were measured in the field during periods of continuous queueing. The results are shown in *Table 5.*

The effect of the turning movement of the subject vehicle on the follow-up time was computed using the direct measurement method. These results are given in *Table 6.*

The follow-up times using both Siegloch's method and the direct computation method are also shown graphically for the four-leg approach sites with major street speeds less than 40 mi/hr.

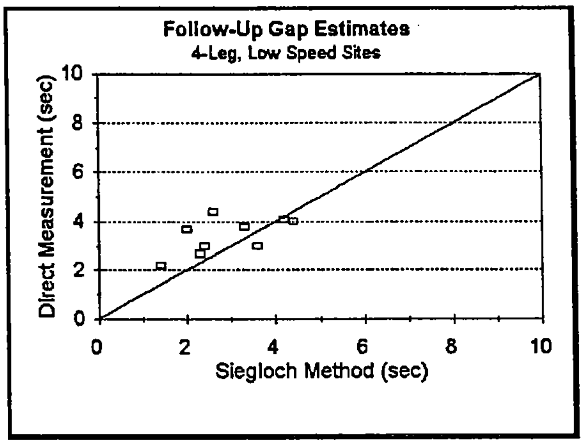

Fig. 3 - Follow-up time estimates

A number of important observations can be made about these data:

- There is some consistency between the Siegloch method and the direction measurement method.
- There is very little evidence that the follow-up time is a function of the turning movement of the subject vehicle.
- There is good consistency between the HCM values and those estimated by the Siegloch method. See *Figure 3.*
- There is good consistency between the HCM values and those directly measured.
- The measured follow-up times are smaller at those sites that allow for more than one vehicle at the stop line.

TABLE 5

Follow-up time results

Site	Siegloch Method		Actual Measurements	Obs	HCM Values
	Sub Veh Only	All Minor Veh			
3Leg/LowSpeed					
T003	-	-	1.9	16	
T008	3.7	4.3	4.1	116	2.1-3.6
T010	4.6	3.2	3.9	75	
T012	-	3.9	2.9	74	
3Leg/High Speed					
T005	3.4	3.0	3.2	66	
T007	3.7	4.0	2.9	285	2.1-3.6
T011	3.4	3.5	3.1	115	
4Leg/Low Speed					
T001-1	-	-	1.9	11	
T001-2	-	-	1.0	6	
T002-1WB	-	2.6	4.4	16	
T002-1EB	-	2.4	3.0	36	
T002-2WB	-	3.3	3.8	16	
T002-2EB	-	2.0	3.7	38	2.1-3.6
T006	4.6	4.4	4.0	300	
T009-1NB	-	2.3	2.7	66	
T009-1SB	-	1.4	2.2	30	
T009-2NB	-	3.6	3.0	94	
T009-2SB	-	4.2	4.1	7	
4Leg/High Speed					
T004-1EB		3.0	2.6	25	
T004-1WB	-	-	-	13	2.1-3.6
T004-2EB	-	2.9	3.4	15	
T004-2WB		-	-	3	

Notes:
1. Siegloch values were estimated using either only subject approach vehicles (noted by *Sub Veh Only*) or all non-priority vehicles (noted by *All Minor Veh*).
2. *Obs* is the number of observation.

TABLE 6

Measured follow-up times

Site	LT	TH	RT	Sample
3Leg/LowSpeed				
T003	2.0	-	1.9	16
T008	4.3	-	2.2	116
T010	5.1	-	3.5	75
T012	3.0	-	2.9	74
HCM	3.4	3.3	2.6	
3Leg/High Speed				
T005	3.7	-	2.4	66
T007	3.1	-	3.1	285
T011	3.3	-	1.0	115
HCM	3.4	3.3	2.6	
4Leg/Low Speed				
T001-1	1.3	-	2.4	11
T001-2	1.1	-	-	6
T002-1WB	-	3.7	2.5	16
T002-1EB	3.7	4.3	4.8	36
T002-2WB	3.3	4.6	3.6	16
T002-2EB	3.4	4.4	3.6	38
T006	4.1	3.8	3.8	300
T009-1NB	3.2	3.6	1.3	66
T009-1SB	3.4	3.8	1.6	30
T009-2NB	2.9	3.6	2.4	94
T009-2SB	3.5	4.6	-	7
HCM	3.4	3.3	2.6	
4Leg/High Speed				
T004-1EB	2.9	2.6	2.0	25
T004-1WB	1.9	4.1	3.0	13
T004-2EB	3.8	1.8	2.9	15
T004-2WB	-	-	2.7	3
HCM	3.3	3.3	2.6	

Notes:
The directional movements of the subject approach vehicle are denoted by *LT, TH,* and *RT*. *Obs* is the number of observations.

4. AWSC INTERSECTIONS

4.1 Site Characteristics

Five AWSC intersection sites were included in the pilot study. There was some difference in the geometric characteristics of the sites. Three of the sites have 4-legs with single lane approaches; one of the sites is a T-intersection with single lanes on the approaches; the final site includes more than one lane on several of its approaches.

For the single lane, 4-leg sites, the following ranges are noted for the five-minute data:

- The mean intersection flow rate ranged from 1150 veh/hr to 1566 veh/hr.
- The minimum intersection flow rate was 816 veh/hr; the maximum flow rate was 1860 veh/hr.
- The mean subject approach flow rate ranged from 228 veh/hr to 536 veh/hr.
- The minimum subject approach flow rate was 84 veh/hr; the maximum flow rate was 768 veh/hr.
- The minimum delay was 4.9 sec/veh; the maximum delay was 101.7 sec/veh.

The macroscopic data base that was compiled from the raw event data for the single-lane, four-leg sites includes:

- 181 5-minute data points,
- 6,152 subject approach vehicles, and
- 4,863 subject approach vehicles that were a part of a continuous queue.

4.1 Estimation of Saturation Headways

The approach capacity of an AWSC intersection depends on the degree of conflict between the vehicles on the subject approach and vehicles on the opposing and conflicting approaches. In TRC 373, four cases were defined that completely described the range of conditions faced by a subject approach driver. For the pilot study, saturation headways were measured for a total of 4,863 vehicles for these four cases. These results are shown in *Table 7*. While the specific headway measurements are somewhat different, the relative magnitudes for the two studies are similar.

TABLE 7

Saturation headway data

Case: Vehicle on Subject Approach Faced By ...	Pilot Study Data	TRC 373 Data
1. No other vehicle on any approach	3.2	3.9
2. One or more vehicles on the opposing approach only	4.9	5.6
3. One or more vehicles on the conflicting approaches only	6.3	6.5
4. One or more vehicles on both the opposing and conflicting approaches	8.0	9.0

One of the shortcomings of the TRC 373 classification system was the limited number of cases considered, particularly the lack of consideration given to (1) the number of opposing or conflicting vehicles faced by the subject approach vehicle, (2) the directional movement of the subject approach vehicle, and (3) the effect of vehicle type.

To more accurately assess the effects of opposing and conflicting vehicles, eight cases were defined. The saturation headways measured during the pilot study were classified into these eight cases and are listed in *Table 8*. The saturation headways measured for each subject approach are given in *Table 9*.

Several conclusions can be drawn from these results:

- The approach direction of a conflicting vehicle (either from the left or from the right) does not affect the saturation headway of the subject approach vehicle. The saturation headway values for Cases 3 and 4 are nearly equal; the values for cases 6 and 7 are also nearly equal.
- The number of vehicles on the conflicting approach (either one or two) does make a significant difference in the saturation headway of the subject approach vehicle. Note that the values for Cases 3 and 4 are lower than for Case 5.
- While the direction does make a difference for one vehicle (the headway for Case 2 is less than the headway for either Cases 3 or 4), it does not make a difference if the subject vehicle faces two opposing or conflicting vehicles (the values for Cases 6, 7, and 8 are nearly the same).

The saturation headways for each of the three directional movements of the subject approach vehicle were computed and are given in *Table 10*. The following conclusions can be drawn from the information presented in this table.

- The turning movement direction has a significant effect on the saturation headway of the subject approach vehicle. In seven of the eight cases listed, the saturation headway for the left turning vehicle is higher than for the through movement, which in turn is higher than for the right turn movement.

■If the through movement is taken as the base case, adjustment factors can be computed for the effect of turning vehicles on the saturation headway. For left turn vehicles, this adjustment factor is 0.90. This means that the capacity reduction due to left turning vehicles is 10 percent. For right turn vehicles, the adjustment factor is 1.36.
■The assessment of saturation headways without respect to subject vehicle turning movement did not show a difference between cases 3 and 4, and cases 6 and 7. Such a difference, however, might be expected when the directional movement of the subject vehicle is considered. The data presented in the table do not support this expectation.

TABLE 8

Saturation headway data for AWSC intersections, pilot study

Case	The Subject Vehicle Faces on ...		Saturation Headway (sec/veh)	
	The Opposing Approach	The Conflicting Approaches	All Sites	4-Leg, Single Lane Approach Sites
1	No other vehicle	No other vehicles	3.0	3.0
2	One vehicle	No other vehicles	4.7	4.8
3	No other vehicle	One vehicle from the left approach	5.7	5.9
4	No other vehicle	One vehicle from the right approach	5.7	5.6
5	No other vehicle	One vehicle from both the left and right approaches	7.3	7.1
6	One vehicle	One vehicle from the left approach	7.3	7.2
7	One vehicle	One vehicle from the right approach	7.4	7.3
8	One vehicle	One vehicle from both the left and right approaches	9.1	9.2

Note: The values are based on the mean of the mean values measured for each of the 13 subject approaches of this pilot study.

TABLE 9

Saturation headway data for each subject approach

Site	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8
A001								
NB	3.7	5.2	5.5	6.0	7.5	7.4	7.7	9.3
SB	3.6	5.2	6.1	6.0	7.8	7.6	7.5	8.9
EB	1.4	4.9	6.5	5.8	7.3	7.7	6.9	
A002								
NB	4.8	5.6	5.0	7.4	9.9	-	9.8	-
A003								
NB	3.9	5.1	6.4	5.8	7.2	7.5	7.7	9.7
SB	4.1	5.2	6.8	6.2	7.3	7.5	7.1	9.2
A004								
NB	3.4	4.8	5.4	5.1	6.3	7.1	7.4	9.5
SB	2.7	4.7	5.8	5.2	6.5	7.5	7.4	9.4
EB	1.6	3.3	4.7	4.7	7.0	5.5	6.5	8.7
A005								
NB	3.5	4.3	5.8	5.6	7.3	7.6	6.8	9.2
SB	2.7	4.5	5.1	6.1	6.9	6.8	7.2	9.2
EB	1.6	3.7	5.4	5.0	6.9	7.4	6.8	8.6
WB	2.5	4.1	5.8	4.8	7.0	8.4	7.4	9.0
All Sites								
Mean	3.0	4.7	5.7	5.7	7.3	7.3	7.4	9.1
St Dev	1.1	0.7	0.6	0.7	0.9	0.7	0.8	0.3
Minimum	1.4	3.3	4.7	4.7	6.3	5.5	6.5	8.6
Maximum	4.8	5.6	6.8	7.4	9.9	8.4	9.8	9.7
1Lane 4Leg Sites								
Mean	3.0	4.8	5.9	5.6	7.1	7.2	7.3	9.2
St Dev	1.1	0.6	0.7	0.5	0.5	0.7	0.4	0.4
Minimum	1.4	3.3	4.7	4.7	6.3	5.5	6.5	8.7
Maximum	4.1	5.2	6.8	6.2	7.8	7.7	7.7	9.7

TABLE 10

Saturation headways by subject approach movement direction

Case	Subject Approach Saturation Headways (sec/veh)								
	LT Movement			TH Movement			RT Movement		
	Hdwy	Obs	Std Dev	Hdwy	Obs	Std Dev	Hdwy	Obs	Std Dev
1	4.0	85	1.7	3.7	200	1.3	2.8	71	1.3
2	6.2	71	1.8	5.0	288	1.5	3.8	60	1.3
3	5.9	60	1.6	5.6	149	1.5	5.2	42	1.8
4	6.2	20	1.4	6.0	69	1.3	4.5	19	1.1
5	7.1	39	1.5	7.3	107	1.6	6.4	18	1.1
6	8.0	134	2.0	7.0	308	1.8	6.3	37	1.8
7	8.9	37	2.1	7.5	151	1.8	5.4	11	1.1
8	10.7	70	4.6	9.2	203	2.7	8.0	22	2.1
Mean	7.1	516		6.4	1475		4.7	280	

Notes:
Hdwy is the saturation headway; *Obs* is the number of observations; *St Dev* is the standard deviation.

TABLE 11

Saturation headways for passenger cars and trucks

Case	For Vehicles		For Heavy Trucks	
	Headway	Obs	Headway	Obs
1	3.2	798	4.6	15
2	4.8	758	5.9	19
3	5.7	355	7.6	5
4	5.7	341	8.7	3
5	7.4	521	8.3	12
6	7.2	624	7.9	15
7	7.6	668	7.9	17
8	9.2	694	11.6	18
Total	6.4	4759	7.8	104

The saturation headway measurements for passenger cars and heavy trucks show a significant difference. *Table 11* shows the computed saturation headways for these two vehicle types. In every case, the saturation headway for passenger cars is lower than that for heavy trucks. This difference averages 1.5 seconds, or 22 percent. These headways can be translated into capacity flow rates, 563 veh/hr for passenger cars and 462 veh/hr for trucks. This results in a passenger car equivalent of 1.2 for trucks at AWSC intersections.