Estimating Capacity and Delay at a Single-Lane Approach, All-Way Stop-Controlled Intersection

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This paper presents the results of a study of single-lane approach, all-way stop-controlled intersections. Data have been collected for nearly 25 hr of operation for eight sites in Idaho, Oregon, and Washington. Estimates of the capacity and nature of the delay/flow rate relationship have been made for these sites. A methodology for analyzing operational performance is proposed. This study is part of a larger effort where data have been collected for 23 all-way stop-controlled intersections in Idaho, Oregon, Washington, Iowa, Colorado, and Texas. In the next phase of this study, the effects of nonstandard conditions (number of approach lanes, pedestrians, heavy vehicles, etc.) will be considered.

The 1985 *Highway Capacity Manual* (HCM) (*1*) establishes vehicle delay as the primary measure of effectiveness (MOE) for evaluating the performance of signalized intersections. Delay is the logical parameter because it satisfactorily describes intersection performance, can be easily measured by the transportation engineer, and can be clearly communicated to the layperson. Much literature exists on the operation and performance of signalized intersections. The relationship between vehicle delay and traffic flow and the procedures for capacity and level of service analysis are well documented.

The situation is different for unsignalized intersections. Reserve capacity is used to define the level of service for two-way stop-controlled (TWSC) intersections. This is a difficult notion for the layperson and not as satisfactory a concept as delay for the transportation engineer. Further, the method described in Chapter 10 of the HCM for evaluating TWSC intersections is based on procedures developed in Europe that have yet to be calibrated with data from the United States. The situation is even worse for all-way stop-controlled (AWSC) intersections. The HCM cites only one study as the basis for rather meager capacity guidelines for AWSC intersections. The Transportation Research Board (*2*) recognizes this deficiency and has identified the development of capacity and level-of-service analysis procedures for AWSC intersections as a high priority for future research. Clearly, there is a need for an improved methodology that can be used in the analysis of AWSC intersections. Further, there is a need for a data base to investigate the relationships between vehicle delay, intersection flow rates, and other key variables that affect intersection performance.

In response to this need, in 1987 the University of Idaho and the Idaho Transportation Department jointly initiated a research project to study the traffic flow characteristics of AWSC intersections. This paper describes the results of this study: the accumulation of a data base, the identification of the relationship between delay and traffic flow rates, and the estimation of intersection capacity. The focus, in general, is on the four-way, single-lane approach, AWSC intersection and specifically on the eight sites for which data have been collected and analyzed.

In this paper, previous studies in this area are described, the data collection method and the site characteristics are presented, and the flow rate/delay relationship is analyzed. Procedures for estimating intersection capacity are given, an evaluation method is proposed, and findings and conclusions are summarized.

PREVIOUS WORK

A literature search was undertaken to determine relevant previous research on stop-controlled intersections (*3*). Two of the more important studies, one empirical and the other theoretical, are summarized below.

Empirical Study by Hebert

In 1963, Hebert (*4*) presented the results of a study of three AWSC intersections in Chicago. The objective of his research was to determine the capacities of AWSC intersections under a variety of traffic and operating conditions. He investigated the average departure headway for an approach for three cases:

1. When both the major and minor streets are loaded,
2. When the subject approach is loaded but no vehicles are on the cross street, and
3. When the study approach is loaded and is affected by cross-street vehicles.

Data were gathered using a movie camera operating at a rate of 100 frames/min, or one frame every 0.6 sec. Three intersections were filmed (two single-lane approach intersections and one multilane approach intersection), and the vehicle headways were calculated. It was found that the ratio of major street volume to total intersection volume (defined as "volume split") affects departure headway and, thus, capacity. Hebert's estimates of departure headway and capacity are the basis for Table 10-5 in the HCM.

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Theoretical Model by Richardson

Besides a lack of data, there has also been a paucity of theory to explain the operation of AWSC intersections. Richardson (*5*) addressed this problem by proposing a model based on queuing theory for a single-lane approach, AWSC intersection with no turning movements. Richardson used departure headway data from Hebert's work as a basis for his model.

Richardson proposed an M/G/1 queuing model that assumes Poisson-distributed vehicle arrivals and service rates described by a general distribution. The queue discipline is single server, first come, first served, since a single-lane approach is assumed. Richardson noted that the service time for a vehicle on any approach is bimodal: either there is a vehicle waiting on one of the conflicting approaches or there is not. For the first case, the service time includes the time for both the conflicting vehicle and the waiting vehicle to clear the intersection, or 7.6 sec from Hebert's study for a single-lane approach case. For the second case, the service time, also from Hebert's study, is 4.0 sec. In calculating the actual service time for each approach, there is an interaction between the approaches that relates to the probability of vehicles waiting for service on each of the approaches (which is a function of the demand on each approach). The model calculates the service times iteratively, and convergence to stable values is rapid.

Richardson used his model to forecast intersection capacity and delay under a variety of volume split conditions. The results of his capacity analysis correlate well with the forecasts of Hebert's intersection capacity equation. Further, Richardson postulated that delay on a given approach is a function of, in decreasing order of importance, flow rate on the subject approach, flow rate on the cross streets (conflicting flow rate), and flow rate on the opposing approach. The Richardson model can be programmed on an electronic spreadsheet (e.g., Lotus 123 or Quattro), and delay calculations can be easily performed.

DATA COLLECTION AND SITE CHARACTERISTICS

The review of the literature revealed that there is little data on capacity or delay for AWSC intersections. Thus, one of the first tasks in this study was to assemble a data base of traffic flow rate and delay data.

Delay can be defined in several ways. "Stopped delay," used here, is defined as that time beginning when a vehicle enters a queue and ending when it crosses the stop line at the intersection and leaves the queue. The use of stopped delay is consistent with the signalized intersection methodology of Chapter 9 of the HCM.

One common method to collect delay data is to monitor the length of the vehicle queue at periodic intervals (*1*). This method yields good results for signalized intersections where at least some of the vehicles are stopped for a portion of the signal cycle. For AWSC intersections, delays may often be less than the sampling interval. Thus, to successfully collect delay data at an AWSC intersection, the progress of individual vehicles in the queue, from entrance to exit, must be traced.

A video camera was used for data collection in this study, which required only one person in the field to collect the data. With data in real-time videotape format, intersection operations can be reviewed as often as needed to record additional data or to observe traffic dynamics. The camera is placed to provide an unobstructed view of all intersection approaches and the resultant vehicle turning movements. In addition, the view must include all queue formation and dissipation activity for at least one of the approaches.

Software was developed that allows delay and flow rate data to be entered into the computer while the videotape of the intersection operation is being observed. The demands placed on software used for real-time data entry are severe and require a certain degree of robustness in the program operation (*6*).

While the eight study sites have a common geometry (four approaches with a single lane on each approach), traffic conditions vary widely among the sites. A comparison of the flow rate and delay data for the sites is given in Table 1.

METHODS OF ESTIMATING DELAY

Forecasting Vehicle Delay

One of the most important objectives of this study is to develop a method for forecasting vehicle delay at AWSC intersections. To achieve this objective, the factors contributing to vehicle delay must first be identified and then quantified. Two methods for estimating vehicle delay are described here. The first is empirically based and uses regression analysis to determine the relationship between subject approach delay and subject approach flow rate, conflicting approach flow rates, and opposing flow rate. The second method is a validation of Richardson's theoretical queuing model to determine how accurately the model replicates "real world" intersection operations.

Empirical Estimation of Vehicle Delay

Plots of delay versus subject flow rate for each site show, in general, an exponential relationship between these variables. Figure 1 shows an example of this plot for Site 4.

A basic model form is suggested using Richardson's hypothesis that subject delay is a function of subject, conflicting, and opposing flow rate. Delay/flow rate equations were developed for each of the eight sites individually and for all 297 data points together using this hypothesized relationship. Equation parameters for the best model fit for each site are summarized in Table 2.

Several conclusions can be drawn from an analysis of these models:

- Subject flow rate is the most important contributor to vehicle delay, and the subject flow rate coefficient is numerically the highest coefficient for each model.
- Conflicting flow rate is also a significant factor in forecasting vehicle delay, but somewhat less important than subject flow rate.
- Opposing flow rate is only significant in those models where opposing flow rate exceeds 300 vph. For those sites where opposing flow rate never exceeds 300 vph for any 5-min period, it is not a significant factor in estimating delay.

TABLE 1 FLOW RATE AND DELAY DATA

Mean 5-Minute Data

Site	Location	Intersection Flow Rates	Approach Flow Rates (vph) Subject	Conflicting	Opposing	Delay (sec)
1	Moscow,ID	851	417	309	126	13.5
2	Moscow,ID	960	340	469	151	15.5
3	Portland,OR	922	476	348	98	6.0
4	Aloha,OR	1453	391	659	403	22.2
5	Boise,ID	1140	461	233	233	19.8
6	Boise,ID	865	388	388	185	11.2
7	Boise,ID	975	312	336	327	10.5
8	Spokane,WA	548	202	279	67	4.7

5-Minute Data Ranges

Site	Location	Intersection Flow Rates	Approach Flow Rates (vph) Subject	Conflicting	Opposing	Delay (sec)
1	Moscow,ID	624-1164	288-636	252- 420	72-168	5-40
2	Moscow,ID	700-1140	180-480	360- 600	60-216	7-34
3	Portland,OR	612-1104	288-648	252- 480	12-180	3- 8
4	Aloha,OR	984-2016	204-564	360-1032	192-600	5-73
5	Boise,ID	852-1488	252-660	144- 720	144-324	5-48
6	Boise,ID	420-1404	144-732	36- 660	24-684	5-37
7	Boise,ID	708-1164	156-492	180- 480	204-528	5-20
8	Spokane,WA	276- 804	108-396	96- 468	0-156	3- 7

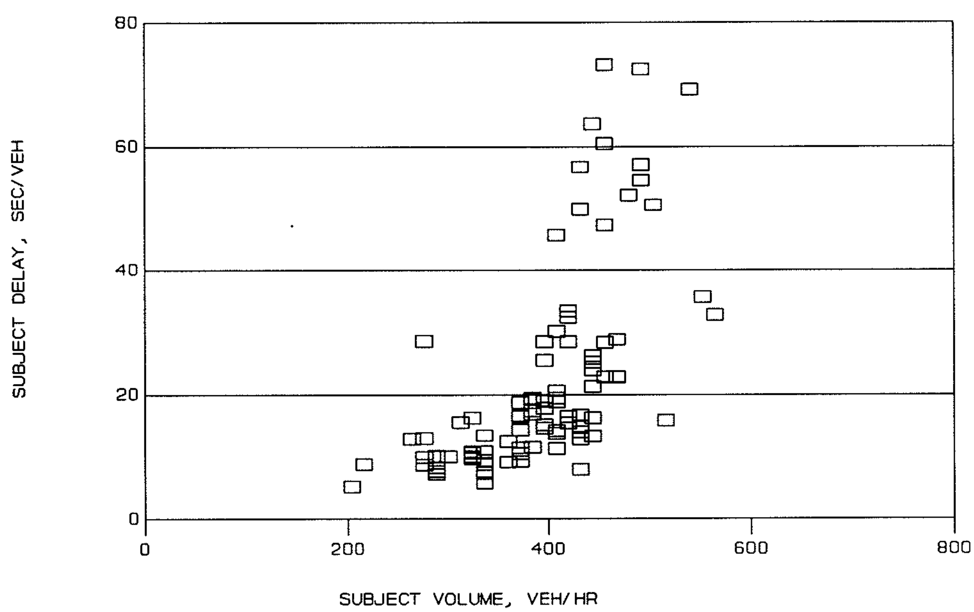

FIGURE 1 Delay versus subject volume, Site 4.

- Contrary to Richardson's hypothesis, the effect of opposing flow rate is approximately equal to that of conflicting flow rate.
- The exponential model form best fits the data for seven of the eight sites and for the overall model. For the other site, flow rates are relatively low and delay remains nearly constant at 5 to 10 sec/veh. The linear form is most appropriate for this case.
- For the sites where high delays were measured (greater than 30 sec/veh for any 5-min period), the models tend to underpredict delays consistently. This leads to the conclusion that there are probably two regions of delay. The first is for low subject approach flow rates (less than 300 to 400 vph) in which delays are constant and average about 5 sec/veh. The second region is for higher flow rates (greater than 400 vph) in which delays increase exponentially.

TABLE 2 DELAY/FLOW RATE MODELS

Site	SubVol	ConVol	OppVol	Constant	R-Sq	Functional Form
1	.00401 ±.00052	.00231 ±.00072	n/s		.99	Exponential
2	.00518 ±.00111	.00181 ±.00082	n/s		.98	Exponential
3	.00243 ±.00080	.00161 ±.00108	n/s		.98	Exponential
4	.00490 ±.00058	.00068 ±.00026	.00131 ±.00055		.98	Exponential
5	.00399 ±.00050	.00216 ±.00051	n/s		.99	Exponential
6	.00240 ±.00029	.00052 ±.00026	.00068 ±.00028	2.99029 ±.25207	.73	Exponential
7	.00377 ±.00064	.00219 ±.00062	.00108 ±.00055		.99	Exponential
8	.00480 ±.00169	.00316 ±.00170	n/s	2.86427 ±.54244	.35	Linear
All	.00375 ±.00019	.00132 ±.00016	.00153 ±.00023		.98	Exponential

Coefficient estimates are shown with their standard errors.
R-Sq is the coefficient of determination, a measure of goodness of fit of the model to the data.
n/s = the variable was not statistically significant and was not included in the final model estimation.

The forms for the linear and exponential equations are shown below:

$$D = a_1 \text{ SUBVOL} + a_2 \text{ CONVOL} + a_3 \text{ OPPVOL} + \text{Constant}$$

$$D = \text{Constant} \times \exp(a_1 \text{ SUBVOL} + a_2 \text{ CONVOL} + a_3 \text{ OPPVOL})$$

Validation of Richardson Model: Theoretical Estimation of Vehicle Delay

The data collected at the eight sites were used to test the validity of the Richardson model. The validation was performed by using the model to calculate vehicle delay based on the measured flow rates and comparing this estimate with measured delays.

As seen from Figure 2, the Richardson model yields good forecasts of subject delay for ranges of subject flow rates up to 400 vph. For this range, residuals (i.e., the difference between model forecasts and actual delay values) are usually 2 sec or less. As subject flow rates increase beyond 400 vph, the model forecasts deteriorate, which is probably the result of two factors. First, the model is based on capacity departure headways estimated by Hebert for three cases of volume splits. These headways have not been verified by other researchers. If they are in error, they will yield incorrect results in the Richardson model. Second, the model does not consider the effects of opposing left turns on subject delay.

METHODS OF ESTIMATING CAPACITY

This section develops and analyzes alternative methods of estimating the capacity of AWSC intersections.

Measuring Capacity

The 1985 HCM defines capacity as the "maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a specified time period under prevailing roadway, traffic, and control conditions." There is a fundamental difference in the notion of capacity for uninterrupted flow and interrupted flow facilities. The HCM specifies that 15-min periods are to be used when measuring capacity on an uninterrupted flow facility. Since traffic flows are subject to instability, primarily because of the interactions between vehicles, there must be some degree of sustainability if a high flow rate is indeed a capacity flow rate and not just a transient effect.

By contrast, the capacity of interrupted flow facilities is controlled not by vehicle interaction but by the nature of the intersection control. Because of the cyclic nature of its operation, intersection capacity is measured on a time scale much less than 15 min. In fact, capacity data for both signalized and unsignalized intersections are based on saturation departure headways. Another issue is the importance of intersection capacity versus approach capacity. The HCM suggests that, for signalized intersections, approach capacity, rather than intersection capacity, is of paramount importance. For unsignalized intersections, however, there is an important interaction between flows on each approach. Hebert showed that

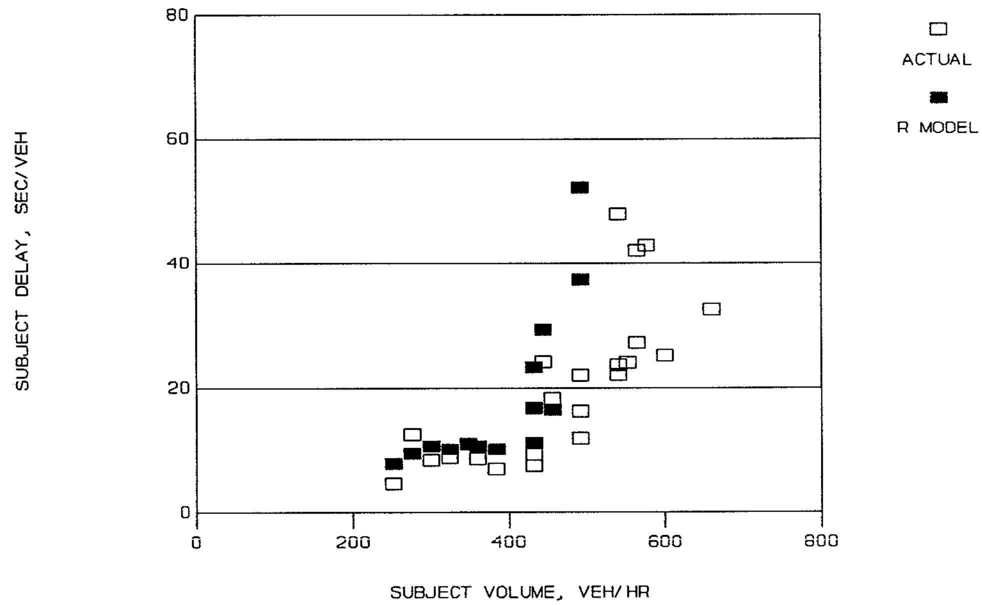

FIGURE 2 Validation of Richardson model, Site 5.

the "capacity" departure headways on one approach of an intersection increase as the volumes on the conflicting and opposing approaches increase.

Finally, when measuring capacity, it is important to understand the basic patterns of traffic flow at AWSC intersections. At low volumes, a vehicle is served as it arrives; while the rule of priority to the vehicle on the right may apply, there is no other apparent pattern to the intersection operation. As demand increases and approaches capacity, a pattern does begin to emerge. For the single-lane approach case, opposite approaches flow simultaneously, and a two-phase operation develops. For multilane approaches, a four-phase operational pattern emerges; that is, each approach travels as a group.

Methods of Estimating Capacity

The method used to estimate the capacity of an AWSC intersection must consider the importance of the interaction of traffic on the various approaches. If the volume on one approach increases, the volumes that can potentially be served on the other approaches are reduced.

On a microscopic basis, the fastest rate at which vehicles can depart is the capacity of that approach. Hebert estimated the departure headway to be 4.05 sec when there is no conflicting flow. This implied a capacity of 890 vph per approach. At the other extreme, he estimated a departure headway of 7.65 sec for a fully loaded intersection, or an approach capacity of 470 vph. Another method for estimating capacity is to examine the empirically based models using the delay/flow rate data for the intersections studied here. When an approach is at capacity, delay approaches very high values. Delay can continue to increase indefinitely as queues continue to increase in length, but there is some maximum acceptable level of delay that drivers will tolerate. Plots of delay versus flow rate show the point where delay begins to increase rapidly; this is the point at which the intersection has reached unstable conditions. The asymptotic value of flow rate might be called the theoretical capacity.

Two methods are explored here: an analysis of the highest flow rate observations and an analysis of departure headway data.

High Volume Observations

One method of estimating capacity is to record the highest measured flow rates at an intersection and to study the conditions present when these high flow rates are observed. A total of 297 5-min observations were made during this study. Of this number, 10 observed intersection flow rates exceeded 1,700 vph, and 33 observations were above 1,600 vph. The highest single observation was 2,016 vph at Site 4. The maximum observed flow rate on an individual intersection approach was 732 vph, recorded at Site 6.

According to Hebert's hypothesis, the highest approach flow rates will be observed on those approaches where volume splits are the highest; this occurs when traffic is concentrated on only one approach. This finding is confirmed in the present study in an analysis of high approach flow rates. In Figure 3, approach flow rates are plotted against the proportion of flow on the subject approach. As expected from Hebert's hypothesis, maximum observed flow rates increase as that approach becomes the dominant flow of the intersection approaches. A regression equation relating departure headway (the reciprocal of flow rate) on the subject approach to the proportion of flow on the subject approach is

$$H = -3.894(\%SUBVOL) + 8.2099 \qquad (1)$$

A similiar pattern emerges between intersection flow rate and the distribution of traffic on the intersection approaches. Figure 4 shows a plot of high intersection flow rates versus

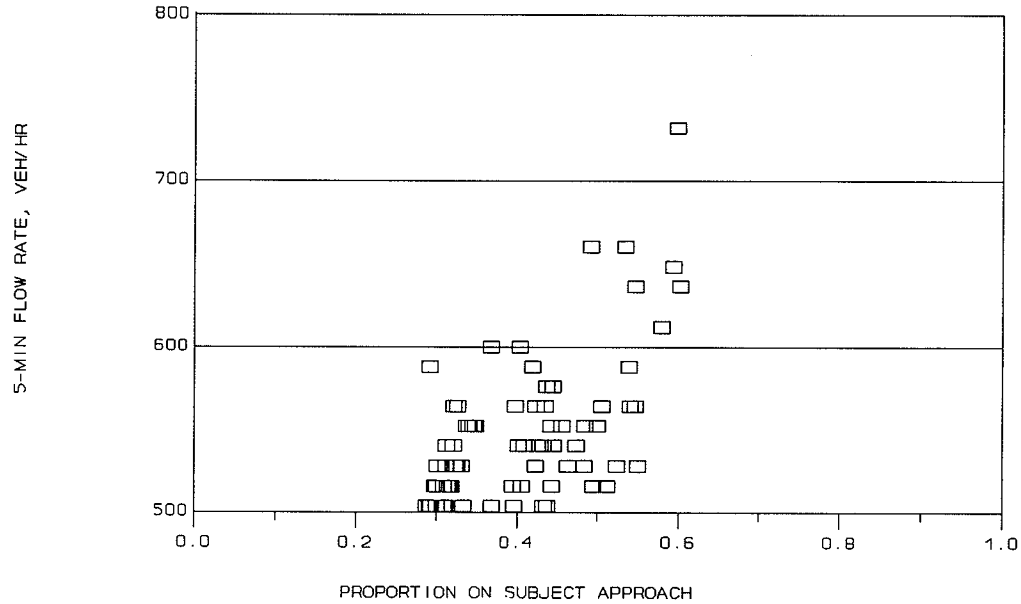

FIGURE 3 Subject approach volumes versus proportion of subject volume, high volume cases.

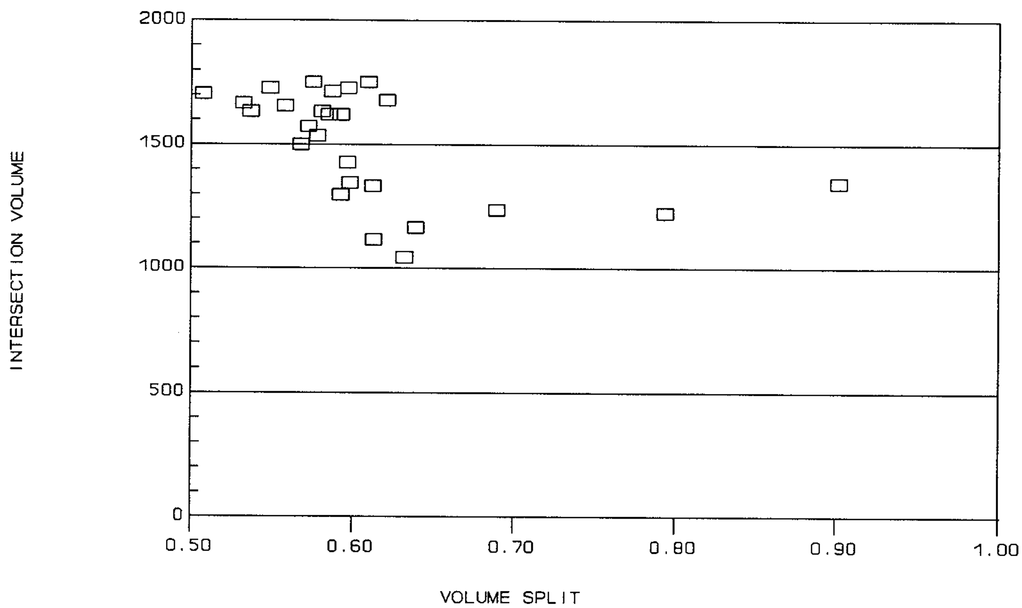

FIGURE 4 Intersection flow rate versus volume split for subject delays above 30 sec.

volume split for cases when delay exceeds 30 sec/veh. These high delay cases approximate capacity conditions.

Headway Analysis

Headway data recorded to one-tenth of a second were collected as part of this study and calculated based on the time between two vehicles departing a given approach.

The conditions of traffic flow on the subject, opposing, and conflicting approaches were noted for each departing vehicle. The subject approach is considered to be loaded or "at capacity" if there is at least one vehicle in queue or waiting to be served. The capacity departure headway for the subject approach can be calculated for several basic situations:

1. When no opposing or conflicting vehicles are waiting, and two subject approach vehicles can travel through the intersection consecutively with no other intervening vehicles;
2. When one opposing vehicle crosses through the intersection between consecutive subject approach vehicles;
3. When one or more conflicting vehicles cross the intersection between two subject approach vehicles; and
4. When both conflicting and opposing vehicles cross the intersection between subject approach vehicles.

So far, headway data have been summarized only for Site 1 (see Table 3).

Even with this limited sample, several important facts are revealed:

1. Departure headways are lowest when there are no vehicles on the conflicting or opposing approaches. The mean value for this case is 4.1 sec, approximately the same value measured by Hebert.
2. Departure headways are somewhat higher when there is a vehicle on the opposing approach. The mean headway is 5.1 sec for this case, but this distribution may be bimodal, indicating that there may be a different effect from straight-through versus left-turning opposing vehicles.
3. Departure headways for Cases 3 and 4 (conflicting vehicles present) average 7.0 and 8.3 sec, respectively.

PROPOSED EVALUATION METHODOLOGY

Elements of an Operational Analysis

One of the basic tools of the HCM is the operational analysis, a technique for determining the capacity and level of service of a highway facility. The operational analysis method has been developed for nearly all types of highway facilities including freeways, multilane highways, rural highways, signalized intersections, arterial systems, and two-way stop-controlled unsignalized intersections. However, there is currently no operational analysis method established for AWSC intersections.

The common features of the operational analysis method include

- Definition of the MOE, a parameter that can be used to assess the performance of the facility over a wide range of operating conditions;
- A method of forecasting the MOE for a given set of traffic and geometric conditions; and
- A method for estimating the capacity of the facility.

Based on the results of this study, an operational analysis method for AWSC intersections should include the following three attributes:

1. Vehicle delay should be used as the MOE. It can be measured easily and adequately describes how well the intersection is performing from the driver's perspective. Use of delay as the MOE will provide for a consistency of analysis and performance evaluation between signalized and non-signalized intersections.
2. Vehicle delay should be estimated as a function of the forecasted volumes for each intersection approach. Delay has been shown to be a function of the flow rates on the subject, conflicting, and opposing approaches. This relationship appears sound from both theoretical and empirical perspectives.
3. Capacity should be estimated as a function of the distribution of flows on each intersection approach.

While it may be premature to suggest an operational analysis methodology based on a study of only eight sites, it is useful to at least describe possible methods for estimating two of the important parameters needed to perform an operational analysis, namely capacity and delay. A set of alternative methods is presented below for estimating delay and capacity for AWSC intersections. It is hoped that these methods can be field tested by other researchers and practicing engineers.

Suggested Methods for Estimating Vehicle Delay

Two methods are proposed for estimating vehicle delay, one analytical and one graphical.

TABLE 3 DEPARTURE HEADWAY DATA, SITE 1

Departure Headways

Case	Number of Observations	Maximum Headway (sec)	Minimum Headway (sec)	Mean Headway (sec)	Estimated Capacity (vph)
1	155	10.8	1.3	4.1	878
2	32	15.2	2.5	5.1	706
3	98	12.4	2.5	7.0	514
4	29	16.1	5.1	8.3	434

Capacities

Case	Subject Headway (sec)	Subject Flow (vph)	Resultant Conflicting Flow (vph)	Resultant Opposing Flow (vph)	Estimated Intersection Capacity (vph)	Percent Subject Volume
1	4.1	878	0	0	878	1.00
2	5.1	706	0	706	1412	.50
3	7.0	514	868	0	1382	.33
4	8.3	434	868	434	1736	.25

Estimating Vehicle Delay: Method 1

The graphical method is based on the plots of subject approach delay versus subject approach, conflicting, and opposing approach flow rates. The graph is based on an idealized description of the delay/flow rate relationship:

- Delay is constant at approximately 5 sec/veh up to a subject flow rate of 300 vph.
- At subject flow rates above 300 vph, the curve branches into several segments, based on the conflicting and opposing flow rates, as delay increases at an exponential rate.

At least with respect to order of magnitude, conflicting and opposing flow rates have been found to affect delay equally. Thus, it is proposed that the sum of these two variables determine the proper curve to be used in estimating delay. The proposed delay estimation curves are shown in Figure 5.

Estimating Vehicle Delay: Method 2

The analytical method is based on an equation relating vehicle delay on the subject approach to the flow rates on the subject, conflicting, and opposing approaches. The equation was developed using data for all eight study sites:

$$D = e^{.00375\ SUBVOL\ +\ .00132\ CONVOL\ +\ .00153\ OPPVOL} \quad (2)$$

Suggested Methods for Estimating Intersection Capacity

Two methods are suggested for estimating intersection capacity: one based on measured departure headways at capacity operations, and the other based on the characteristics of the high flow rates measured as part of this study.

Estimating Intersection Capacity: Method 1

The departure headways calculated for Site 1 under various flow configurations can be used to estimate intersection capacity. Capacity values for four flow cases are given in Table 3. These represent the range of intersection loading conditions described earlier. The subject approach capacity for each case is estimated by the following equation:

$$\text{Subject approach capacity} = \frac{3{,}600}{\text{Mean departure headway}} \quad (3)$$

Using this equation, the subject approach capacity for Case 1 is 878 vph. The intersection capacity for Case 1 is simply the subject approach capacity, since no vehicles are present on any of the other approaches.

The subject approach capacity for Case 2 is estimated to be 706 vph. Because this case assumes that one opposing approach vehicle enters the intersection between two consecutive subject approach vehicles, by symmetry, the opposing approach capacity is also equal to 706 vph. Thus, the intersection capacity is 1,412 vph.

The subject approach capacity for Case 3 is 514 vph, but the estimation of intersection capacity is somewhat more complex because of the method used to estimate departure headways. Since this case includes vehicles from either one or both of the conflicting approaches traveling between consecutive subject approach vehicles, conflicting volumes can range from 514 to 1,028 vph. If the maximum conflicting volume is assumed to occur at capacity conditions, the intersection capacity is estimated to be 1,542 vph.

For Case 4, the subject approach capacity is 434 vph. This case includes one opposing vehicle and either one or two conflicting vehicles (one from each approach) traveling between consecutive subject approaches. Thus, the conflicting and

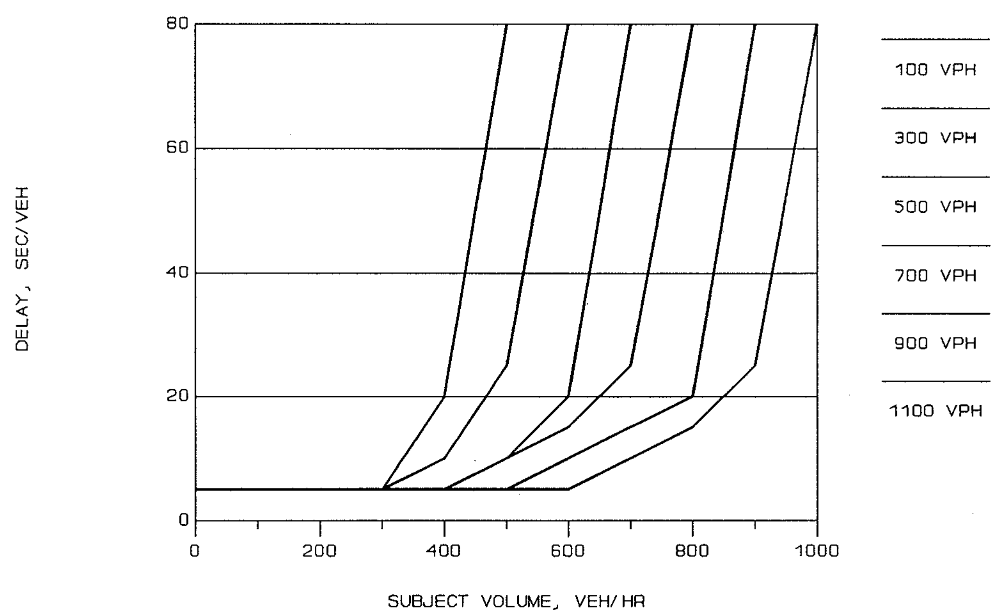

FIGURE 5 Graphical delay estimation method (conflicting and opposing volume range of 100 to 1,100 vph).

opposing approach flow rates can range from 868 to 1,302 vph. If the maximum conflicting volume is assumed to occur at capacity conditions, the intersection capacity is estimated to be 1,736 vph.

This data is used to develop the relationships between subject approach capacity and intersection capacity as a function of the proportion of traffic on the subject approach. A plot of this relationship, shown in Figure 6, can then be used to estimate intersection capacity as a function of conditions on the subject approach.

Estimating Intersection Capacity: Method 2

The proportion of traffic on the subject approach for high flow rate observations was analyzed and an equation relating departure headway (the reciprocal of subject approach flow rates) to this variable was estimated:

$$H = -3.894(\%SUBVOL) + 8.2099 \quad (4)$$

Table 4 gives estimates of subject approach and intersection capacity using Equation 4 for a range of intersection flow conditions (see Figure 7).

FINDINGS AND CONCLUSIONS

The results presented in this paper are the first step in the development of a methodology to determine the level of service of unsignalized intersections and to provide a consistent measure of effectiveness for all types of intersections. The data represent a wide range of delay and flow rate conditions.

Several important findings and conclusions result from this study:

- Delay on a given approach (called the subject approach) is a function primarily of the flow rate on that approach. It is secondarily a function of flow rates on the conflicting and opposing approaches, which contribute to delay in an approximately equal manner.
- Delay remains constant (at approximately 5 sec/veh) for low flow rates, up to subject flow rates of 300 to 400 vph. At this point, delay begins to increase exponentially, and asymptotically approaches the subject approach capacity point.
- The minimum or capacity departure headways on a given approach are a direct function of the traffic patterns on the conflicting and opposing approaches. The lowest departure headways occur when there is no traffic on either the conflicting or opposing approaches. Departure headways increase with the following conditions:
  1. No traffic on either the conflicting or opposing approaches,
  2. Traffic on the opposing approach only,
  3. Traffic on the conflicting approach only, and
  4. Traffic on both conflicting and opposing approaches.
- Methods for estimating delay and capacity are proposed. Two methods for estimating delay are described, one using a graphical approach and the other an analytical approach. Two methods for estimating capacity are proposed, one based on departure headways and the other on conditions observed for high flow rate situations. It is hoped that other researchers will test these methods and suggest improvements where warranted.
- Hebert's hypothesis that the proportion of traffic on the major and minor streets affects intersection capacity has been verified. However, a more accurate indicator is the distribution of traffic on each of the four approaches. Hebert's estimates of capacity are probably low, however, and his assertion that capacity is not affected by the proportion of left-turning vehicles is probably incorrect.
- Richardson's queuing model provides good estimates of vehicle delay for subject flow rates up to 400 to 450 vph. Above this point, the model gives poor results. Two likely reasons are incorrect departure headways as estimated by

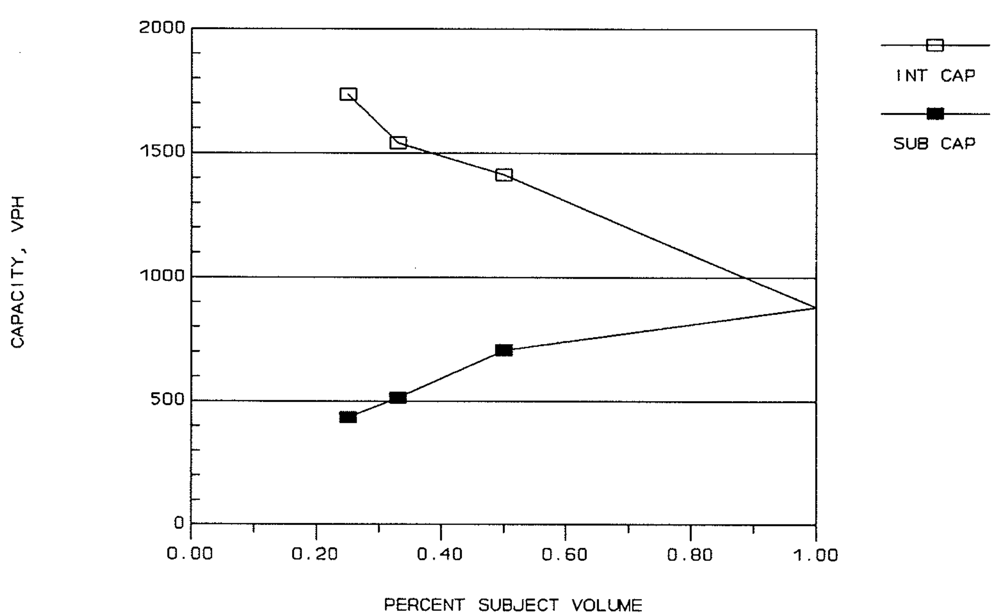

FIGURE 6 Capacity estimation, Method 1.

TABLE 4 CAPACITY AS A FUNCTION OF PERCENT SUBJECT VOLUME

%SubVol	Headway	Approach Capacity	Intersection Capacity	Hebert's Estimate
.25	7.236	497	1990	1882
.30	7.042	511	1704	1678
.35	6.847	526	1502	1547
.40	6.652	541	1353	1463
.45	6.458	557	1239	1416
.50	6.263	575	1150	1398
.55	6.068	593	1079	
.60	5.873	613	1022	

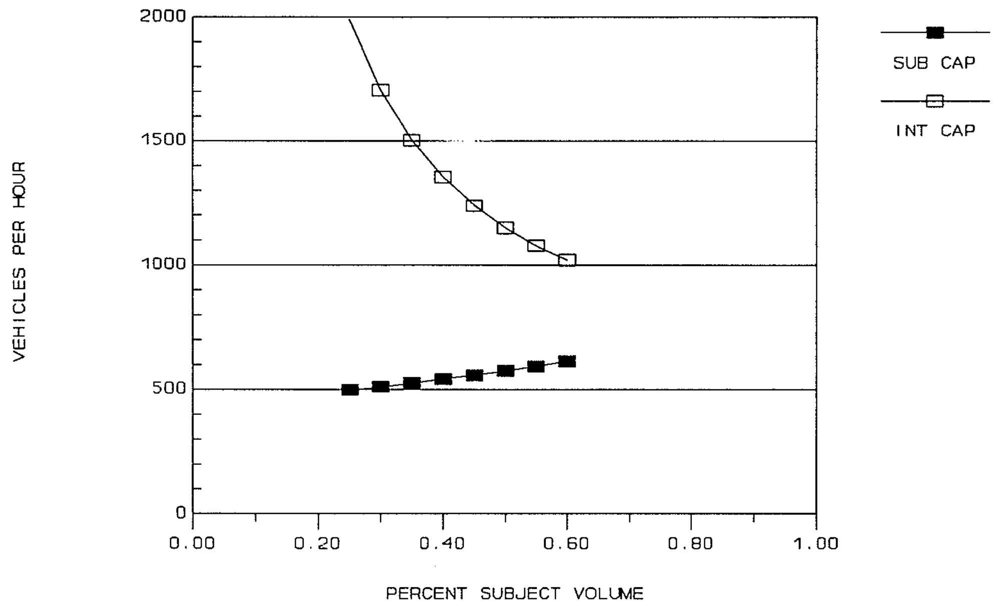

FIGURE 7 Capacity estimation, Method 2.

Hebert and the omission of the effects of opposing left-turning vehicles in the model.

More work must still be done to provide a complete understanding of the operations of AWSC intersections. It is hoped that the work summarized in this paper provides both motivation and a good foundation for future research.

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