Southwest Region University Transportation Center

Congress Avenue Regional Arterial Study: Variability of Departure Headways

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16. Abstract

Virtually all aspects of arterial street performance are dominated by traffic signal operation. Efficiency of signal operations is synonymous with green intervals having exactly the correct duration. That is, greens that are too short or too long can significantly reduce operational efficiency by causing traffic delays to traffic streams using the intersection. Green interval duration is dependent upon many things, but two of the most significant are queue start-up time and intervehicle headways.

These two highly significant traffic flow parameters have been measured through a carefully designed field testing program. Estimates of appropriate parameter values are developed. Actual variation of both parameters across a wide range of geometric, traffic, and other conditions was captured. Predictive models are developed and their use in signal timing optimization is described.

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CONGRESS AVENUE REGIONAL ARTERIAL STUDY: VARIABILITY OF DEPARTURE HEADWAYS

by

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Research Report SWUTC 95/60019-2

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June 1995

EXECUTIVE SUMMARY

This report is the second of four which document work performed as part of the Southwest Region University Transportation Center (SWRUTC) study "Demonstration of Enhanced Arterial Street Traffic Flow, Reduced Fuel Consumption and User Costs Through Application of Super Street Technology". This study constitutes an effort to demonstrate user benefits through development and application of state-of-the-art traffic engineering technology. Specifically, it is an effort to produce an improvement program for Congress Avenue in Austin, Texas which will upgrade its functional class from "major arterial" street to "regional arterial status" and quantify associated user benefits. One extremely important study component is development of new technology which can solve basic problems encountered during improvement plan preparation.

This report is an attempt to extend the state of present knowledge regarding traffic flow through signal controlled intersections. Congress Avenue or any other regional arterial street will have signal controlled intersections which will dominate traffic performance. Specifically, this work uses field traffic flow data to characterize queue start-up times and inter-vehicle headways. These data are used to develop predictive models which, in turn, can be used to significantly improve signalization efficiency. Model development and recommendations are fully described. Conclusions developed through this effort will play a very significant role in the overall Congress Avenue improvement program.

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ABSTRACT

Virtually all aspects of arterial street performance are dominated by traffic signal operation. Efficiency of signal operations is synonomous with green intervals having exactly the correct duration. That is, greens that are too short or too long can signficantly reduce operational efficiency by causing traffic delays to traffic streams using the intersection. Green interval duration is dependent upon many things, but two of the most significant are queue start-up time and intervehicle headways.

These two highly signficant traffic flow parameters have been measured through a carefully designed field testing program. Estimates of appropriate parameter values are developed. Actual variation of both parameters across a wide range of geometric, traffic, and other conditions was captured. Predictive models are developed and their use in signal timing optimization is described.

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CHAPTER 1. INTRODUCTION

The number of waiting vehicles that can cross a signalized intersection in a given period of time depends on how soon the vehicles begin to move after the signal changes to green and how fast each individual vehicle in the queue reacts to the acceleration of the vehicle immediately ahead. This process continues until all cars in the queue are progressing or have progressed through the intersection. The dissipation of a queue of vehicles after the signal changes to green depends on the reaction time and acceleration characteristics of each individual driver and vehicle. Thus, the total time for a group of vehicles to pass through a signalized intersection can vary considerably depending on the alertness and aggressiveness of the individual drivers, their familiarity with the intersection in question, and the acceleration characteristics of the vehicles which the drivers control.

The fact is accepted that all drivers, when exposed to the same situation, have different reaction times. That different drivers when placed behind the wheel of identical cars will accelerate from a standing start at different rates is also accepted. These characteristics inject a certain expected variability into any field data collected concerning vehicle performance at intersections.

This study attempts to provide some information in the lost time aspect within each cycle. The lost time is defined as the excess time needed for a number of vehicles to pass through a signalized intersection compared with that which would be needed if the signal did not exist.

The main objective of this study is to obtain a set of reliable and unbiased departure headways for different queue positions at signalized intersections. It was also intended to collect as much information as possible so that major factors affecting departure headways could be identified. However, because of limitation on data collection sites and their associated condition variables, this part of the study was limited to examining only a few selected factors, including lane width, lane position (inside lane versus outside lane, etc.), time of day (morning peak versus afternoon peak), and posted speed limit (30mph, 35mph, 40mph, or 45mph). For this reason, departure headways were collected for a number of signalized intersection approaches in Austin, Texas. Statistical analyses were performed in order to define the elements of the approaches that influence the time headway values.

Chapter 2 presents, in a chronological sequence, summaries of studies conducted in the past to estimate capacity and saturation flow at signalized intersections. The techniques used for the data collection in this study are described in Chapter 3. The statistical analyses that were performed are presented in Chapter 4 along with the results. Finally, in Chapter 5, the conclusions of this study are outlined.

CHAPTER 2. LITERATURE REVIEW

When the green signal phase begins on an approach to an intersection, stopped vehicles take some time to begin moving; but after a few seconds, the queue discharges at a more or less constant rate termed saturation flow. A basic model of the variation of queue discharge rate with time in a fully saturated green period is illustrated in Figure 2.1. A fully saturated green period is one in which the queue discharge rate remains fairly constant until the green period ends. Saturation flow may vary as a function of items such as layout of the intersection (lane width, grade, etc.), number of turning vehicles, and types of vehicles in the traffic stream.

Estimation of saturation flow values is of prime importance when determining signalized intersection capacity. The objective of this study was to collect a large sample of field data so that reliable saturation flow values could be computed and factors affecting saturation flow could be identified. These saturation flow values may be used as input for determining intersection capacity and when using computer models, to simulate and optimize signal systems.

An extensive literature review was conducted on the topics of intersection capacity and saturation flow. This review identified various methods which had been used to measure saturation flow. It also identified many factors which had been found to affect saturation flow.

GREENSHIELDS [REF. 2, 3]

One of the earliest efforts to quantify vehicle flow characteristics at urban street intersections was made by Bruce D. Greenshields in New York City and New Haven, Connecticut, in 1946 . Entering headway was one of the major items investigated.

Method of Field Observation

As a means of recording field data, pictures of traffic were taken at short successive time intervals ("time motion" method). The equipment which was mounted on high buildings adjacent to intersections, consisted of an Eastman Cine-Kodak Special 16 mm camera with an auxiliary timing mechanism. Pictures were taken at a rate of 88 per minute. Due to wartime restrictions at the time the project was started, it was impossible to purchase a timing mechanism, so one was made by using the spring motor from a hand-cranked phonograph. The time was found to be accurate to at least one in one thousand.

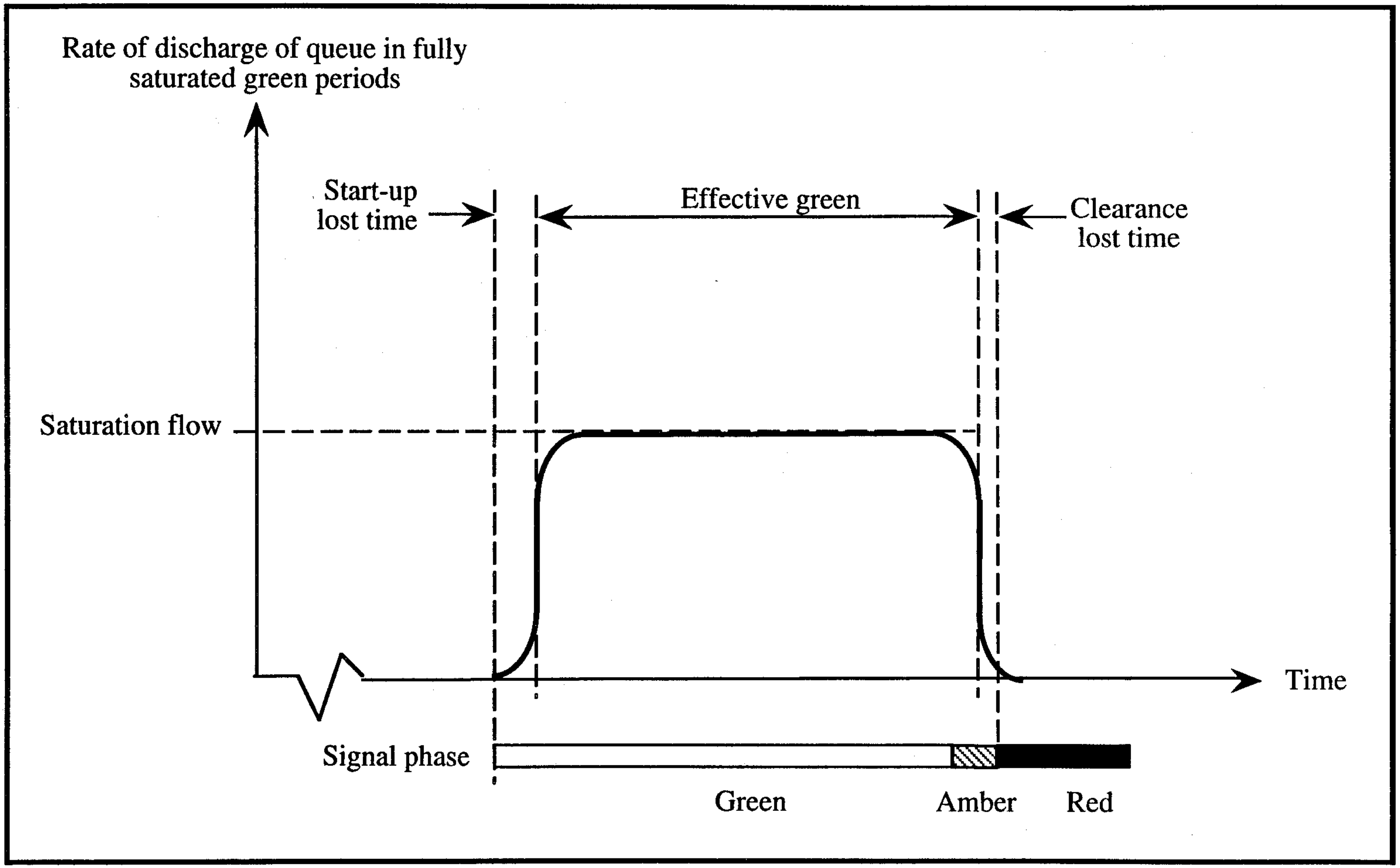

Figure 2.1. Queue discharge rate in a fully saturated green period
Source: Adapted from Ref. 1

The distances vehicles move during the intervals between successive pictures were determined by measuring vehicle location relative to grid lines drawn on a screen. The accuracy to which the positions of the vehicles could be read on the grid depended upon the clarity of the image and the distance of the vehicle from the camera. Most of the films were read to one foot which was considered accurate enough for most purposes but in some cases readings to one-fourth foot were possible. Any single reading as recorded might be in error by 1/88 minutes or 0.682 seconds since this is the most precise measurement of the timing mechanism.

Starting Reaction Time

Starting reaction time as considered in this study was the interval between the signal change to green and the movement of the first waiting vehicle. The average reaction time varied from 0.63 to 2.86 seconds. No average value was indicated that would apply to all intersections.

Reaction Time Between Successive Vehicles

Reaction time between successive vehicles is defined as the time elapsed between starting movements of two successive vehicles.

The interval (reaction time) between the movements of successive vehicles was found to range from 1.09 to 1.79 seconds, with an average reaction time of 1.25 seconds.

It was observed that a second-in-line vehicle may start as soon as the first-in-line. Since a reaction time of zero is impossible, the second driver must react to the signal rather than to the movement of the first vehicle. It was found, however,that after the first two vehicles the average reaction time for a waiting queue of vehicles is constant.

Headways Between Vehicles Entering an Intersection

The average time for the first ten waiting vehicles to enter after the change of the signal to green is: 3.8, 3.1, 2.7, 2.4, 2.2, 2.1, 2.1, 2.1, 2.1, and 2.1 seconds. It can be noted that the interval between vehicles after the first five is a constant equal to 2.1 seconds.

BARTLE, SKORO AND GERLOUGH [REF. 4]

In 1952, Richard M. Bartle, Val Skoro and D. L. Gerlough investigated the starting delay and time spacing of vehicles entering signalized intersection in Los Angeles. The method proposed differed from that of Greenshields in that it focused on the entire intersection approach rather than a single lane.

Data Collection

Starting delay, designated D, was defined as the time in seconds required for the first vehicle to enter the intersection after the display of the green signal.

Time spacing, designated S, was the average time headway in seconds between successive vehicles in an entering platoon. In the measurement of S the number of lanes was disregarded, and the entire intersection approach was considered a unit.

This study was intended to examine the variability of starting delay, D, and time spacing, S, both from intersection to intersection and from day to day at the same intersection. Observations at a given intersection were made on five consecutive weekdays between 4:15 and 5:45 p.m. During each study period the observer recorded data for thirty-one signal cycles.

Starting delay was recorded as the time from the first display of green to the entrance of the first vehicle into the intersection. A vehicle was considered to have entered when its rear wheels crossed the pedestrian crosswalk line nearer the center of the intersection.

Time spacing applies only to platoon movement and is computed by dividing total time in a signal cycle used for platoon movement by one less than the number of cars in the platoon. The time for platoon movement was recorded as the time from the entrance of the first vehicle into the intersection until the entrance of the last car of the platoon. The observer also recorded the number of vehicles entering during this time. Average time spacing for a given cycle was determined by dividing the time for platoon movement by one less than the number of vehicles entering during that time. Determining the end of a platoon was a judgment on the part of the observer. Observers were instructed to consider a platoon ended whenever any one lane was empty or whenever traffic entered the intersection without being restricted in any way by cars immediately ahead. Observers were urged, if necessary, to cut off platoons early in order to be certain that all cars counted were actually traveling in platoons. Data were not recorded for individual lanes; data were based on all cars entering from all lanes in one direction.

Data Analysis

Starting delays were in most cases normally distributed. Departure from normality, where it existed, was in the form of positive skewing with a long tail of high values of D.

The mean starting delays at the thirteen intersection approaches studied ranged from 2.91 to 4.40 seconds. The effect of location on mean starting delay was tested for significance by analysis of variance. The hypothesis tested was that all thirteen mean starting delays were equal. The hypothesis of equal means can be rejected at the 0.005 level of significance, and the effect of location is thus found to be significant. The effect of day of data-taking on D was not significant at the 5 percent significance level. Low mean starting delay is associated with low mean standard deviation. The mean value for starting delay, D, for the thirteen approaches studied is 3.83 seconds with an average standard deviation of 1.27 seconds within each day. The average standard deviation among five means for different days is 0.27 sec. Comparison of mean delay values with various intersection characteristics does not reveal any one factor which appears to have a consistently important effect in increasing or decreasing the value of D.

Average time spacing for the approaches studied ranges from 0.95 to 1.63 sec., and mean values are significantly different among different approaches studied, but the mean value obtained for one weekday will not usually differ significantly from that for other weekdays. The data show that S is a function of intersection characteristics and that significant differences in time spacing values exist among different intersection approaches. Examination of the data indicates that the two factors having the greatest effect on time spacing S for the intersection approaches studied are (1) street width and (2) parking conditions. Within the range of street widths studied, smaller values of S were found for streets with no parking than for streets of the same curb-to-curb width but with parking. The difference between S values for the two parking conditions appears to decrease as the width increases. Better curves presumably can be drawn when data have been collected for more intersections and on a wider range of street widths.

HELM [REF. 6]

In England, Brian Helm studied the saturation flow of traffic at light-controlled intersections in 1957-58 .

In his study he was concerned with the behavior of vehicles in queues. During the red period, 95 percent of vehicles stopped with the front bumper within 10 feet of either side of stop line. On the average, vehicles remained stationary for a further 2.4 seconds after the start of red-amber. Each successive vehicle within the queue started to move 1.5 seconds after the vehicle in front. Queues of light vehicles (cars, light vans, and motorcycles, where a full vehicle space was used, having a length between 138 and 180 in.) were discharged from intersections at steady rates after the exit of the second vehicle of 1.88±0.07 seconds/vehicle.

CAPELLE AND PINNELL [REF. 7]

Donald G. Capelle and Charles Pinnell made a capacity study of signalized diamond interchanges in 1961 . A part of that study was vehicle operational characteristics.

Data Collection

The Wayside Drive and Cullen Boulevard interchanges on the Gulf Freeway in Houston were selected for study sites. Both of these interchanges were conventional-type diamond interchanges.

All of the traffic operational data were collected by filming traffic operations at each of the study intersections with a 16-mm motion picture camera. The filming was performed from a vantage point provided by a hydraulic platform truck. The platform on this truck extended to a height of 35 ft and additional elevation was gained by taking advantage of the terrain. The truck was located in an inconspicious area and it was felt that the presence of the truck and photographer had little effect on the behavior of traffic in the intersection being filmed. The movies at each study site were taken at a camera speed of ten frames per second which permitted accurate determination of vehicle time-headways and delay. The studies were conducted during both the morning and evening periods of peak flow on an average weekday.

As an aid to the determination of vehicle delays and time-headways from the motion pictures, reference lines were placed perpendicular to traffic lanes at each intersection approach. The purpose of these lines was twofold: (a) to regulate and fix the region where approaching vehicles would stop when waiting for a green indication; and (b) to aid in determining when each vehicle entered the intersection area.

Data on traffic operation were extracted from the film through the use of a specially constructed projector. A special control attached to the projector permitted the film to be advanced or reserved in single-frame increments and an interconnected frame counter allowed the operator to determine the number of frames between specific events on the film. By using the constant camera speed, elapsed time between events could be determined.

Data Analysis

It was found that the time-headway between vehicles as they started from a stopped position one behind the other decreased progressively until they reached an average minimum (Table 2.1 & 2.2). Data from this study indicated that an average time-headway could best be obtained by averaging the time-headway values of the third through the last entering vehicle.

When a traffic signal interrupts a flow of traffic, the vehicles stopped by a signal are delayed during the time the signal is red plus the time required for the vehicles to get started and underway again. This latter delay is commonly called starting delay. A generally accepted definition of starting delay is the time required for the first vehicle in a queue to commence motion and enter an intersection after the traffic signal displays a green indication. This time does represent a large portion of the starting delay experienced during each signal phase, but the operational studies showed that it does not represent the total time required for a line of vehicles to attain a reasonable degree of momentum. This is best illustrated by plotting the average time-headway values of a line of vehicles from a stopped position. The time-headway decreases rapidly for the first two vehicles in line with a lesser decrease for each succeeding vehicle. This indicates that the starting delay of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first two vehicles in line.

The operational characteristics of over 4,000 vehicles were recorded. Table 2.1 is a general summary of the data gathered. The operational data given in Table 2.2 indicated little difference in the operating characteristics of (a) straight through movements, (b) left turn movements, and (c) right turn movements. There was no significant difference in the starting delay and time-headway measurements of the straight, single left turning, and single right turning movements. However, there was a significant difference in the double left or two-abreast-type turning movement which necessitated separate consideration of these types of movements.

SCHWARZ [REF. 8]

Heinz Schwarz examined the influence of the amber light on starting delay at intersections in 1961 .

Starting delay data were collected at seven intersections in Chicago, both with and without the amber light preceding green. The experiment was well designed statistically, making possible the testing of two hypotheses with a known degree of significance.

The first hypothesis was that the standard deviations of starting delay, both with and without preceding amber, were equal. At none of the seven intersections was there evidence to reject this hypothesis (a=0.05).

The second hypothesis was that mean starting delay with preceding amber equals mean starting delay without amber. This hypothesis was rejected at all seven intersections (a=0.01). Starting delay averaged 2.97 sec with amber preceding green and 4.17 without the amber.

TABLE 2.1. OPERATIONAL DATA

	Starting Delay (sec)	Average Time-Headway (sec)
Wayside Interchange		
Wayside Drive:		
Through movement	5.9	2.2
North frontage road:		
Lane 1 - 85% left	5.8	2.1
Lane 2 - 99% straight	5.7	1.9
Lane 3 - 82% right	5.8	2.1
South frontage road:		
Lane 1 - 57% right	6.8	2.4
Lane 2 - 65% straight	6.5	2.2
Lane 3 - 100% left	6.5	2.4
Cullen Interchange		
Cullen Boulevard:		
Through movement	5.6	2.1
North frontage road:		
Lane 1 - 68% left	5.3	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 56% right	5.8	2.1
South frontage road:		
Lane 1 - 53% right	5.6	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 79% left	5.6	2.0

TABLE 2.2. SUMMARY

Type of movement	Starting Delay (sec)	Average Time-Headway (sec)
Through	5.8	2.1
Single left turn	5.8	2.1
Single right turn	5.8	2.1
Two-abreast-type turning:		
Inside lane	6.5	2.4
Outside lane	6.5	2.2

WILDERMUTH [REF. 9]

In Zurich,Switzerland, Bruno R. Wildermuth analyzed average vehicle-headways at signalized intersections under different conditions in 1962 [Ref. 9].

Method of Measuring

The following basic requirements were set up in order to establish sound information on average headways:

(1) The approaching traffic volume has to be of such an amount that every green interval is fully utilized.

(2) A free flow of traffic through the intersection and the exit street must be provided.

(3) The amount of motor-cycle and bicycle traffic should not be of any significance.

(4) No parking or stopping should be permitted at or near the intersection either on the approach nor on the exit roadway.

(5) The grades of the approach, the intersection or the exit street shall not have any noticeable slopes.

(6) The lane width of the approach, through the intersection and on the exit street should be at least 10 feet (3.00 meters). For European conditions, with most cars being small, a lane width of 10 feet seems about equivalent to the 12 foot lane, when most cars are of the standard American size.

Traffic signals were operated manually (by a police officer) during morning, noon, and evening-peaks. During peak-hours the length of the green intervals was measured with a stop-watch and the number of light cars (passenger cars and light delivery-trucks) as well as the number of heavy vehicles (trucks and busses) were counted. Separate measures of the same kind were taken on traffic lanes from which vehicles either went straight through or turned.

Analyses

The influence of the length of the green interval on average headways was determined first. For this analysis only samples without any heavy vehicles were applied. They were distributed into eleven groups each representing a different phase length. The mean average headway of each group was computed from 50 to 140 observations. The results are shown in Table 2.3. It can be seen that the length of the green phase does have an affect on average headways.

TABLE 2.3. AVERAGE HEADWAYS AND LENGTH OF PHASES*

Length of phase group (sec)	smallest measured (sec)	largest measured (sec)	Mean (sec)
10	2.00	3.33	2.348
15	1.875	3.00	2.145
20	1.66	2.50	2.060
25	1.66	2.50	2.038
30	1.63	2.50	2.025
35	1.75	2.33	1.998
40	1.695	2.50	1.998
45	1.73	2.25	1.977
50	1.72	2.27	2.035
55	1.77	2.29	2.046
60	1.77	2.40	2.049

*For light vehicles on through lanes only (no heavy trucks or busses, no turning traffic)

LEONG [REF. 10]

In Australia (driving on the left side of the road), H. J. W. Leong made an extensive investigation of urban intersection capacity in 1964 .

Data Collection

The data collected included the measurement of saturated vehicle headways and vehicle delays under various conditions of traffic flow. The equipment used for such measurements included a tape-switch, suitably modified for detection of lateral placement, a photo-cell for recording the length of the 'green' period at the traffic signals and an Esterline Angus 20-pen recorder.

The tapeswitch was placed parallel to and about 4 ft downstream from the stop line to avoid vehicles stopping with their front wheels over the tapeswitch. Coded marks were transferred to the 20-pen recorder by electrical switches, manually operated to indicate vehicle classification, whether vehicles turned left or right, the presence of parked vehicles, and the stopping of buses. By measuring the recorded time interval between successive vehicles for saturated conditions, the saturation flow for each lane was obtained.

A number of signalized urban intersections were selected where the width of approach pavements varied from 12 to 31 ft. The sites chosen were of reasonably level profile with straight approaches and departures and where there was little or no pedestrian interference to vehicular traffic flow. All sites could be termed as being within an 'intermediate' urban area.

Data Analysis

The vehicle headways observed were plotted against vehicle position, following the commencement of the green period. From this study, it may concluded that at signalized intersections in the Sydney metropolitan area, based on 'through' passenger cars only, saturation flow is only attained after the fourth vehicle in each signal cycle, following which the saturated vehicular headways are independent of vehicle positions. The commercial and turning vehicles have been excluded on the assumption that the presence of either affects only the headway of that vehicle together with the headways of the two cars following immediately behind it. In Table 2.4 the headways in the study sites are shown. In order to investigate whether the headways are different for vehicles travelling in different lanes, t-tests were carried out on the values of average headways given in Table 2.4. If acceptance is based on the 5 percent probability level, it may be stated that, in general, there is no significant difference between the saturation flow in either lane. As no significant difference was found in the headways of different lane widths, results have been grouped for the purpose of considering lane width in the last column of Table 2.4. From an analysis of these results, the average headway at the intersection studies was 2.12 seconds for through passenger cars and was independent of lane position or lane width.

On the assumption that vehicle headways remain constant after the fourth vehicle has crossed the stop line, the starting lost time was measured and the results are shown in Table 2.5. From this Table it may be noted that the average lost time on starting was 1.12 sec. for all sites. This result does not take into account the effect of commercial vehicles and should therefore be used only where the percentage of commercial vehicles is low.

In the statistical analysis it was assumed firstly that the effect of commercial and turning vehicles may be removed by excluding the headways of these vehicles and the two cars followed immediately behind them. Secondly, it was assumed that the passenger car headway distribution is normally distributed. The assumption that passenger car headways are normally distributed is not strictly true. For practical purposes, however, the discrepancy has been neglected on the basis that the departure from normal is not great enough to give significant inaccuracy to the solution, particularly for testing the significance of the means. Attempts have been made to compare the observed headway distributions with some known mathematical models. It has been found by the chi square tests that the saturated passenger car headway distribution can neither be represented by the normal nor by the Erlang distribution. It has also been found that inclement weather and upgrades of 4 percent or more in approach to an intersection are two factors which reduce saturation flow.

GEORGE AND HEROY [REF. 11]

In 1966, Earl T. George, Jr. and Frank M. Heroy, Jr. conducted a study to gain more factual information on vehicle and driver characteristics at signalized intersections.

Data Collection

The objective of the study was to determine the time required for each vehicle in a line of stopped vehicles to begin its forward motion after the beginning of the green signal at a signalized intersection. Thus, the time lag from the beginning of a green period to the start of forward motion of vehicles for each position from an intersection stop line was measured.

A total of five traffic lanes at two isolated intersections in New Orleans, Louisiana were studied to determine what effect the location had on this time interval. Locations were selected that had protected left-turn lanes, or "slots", so that the relationship between turning vehicles and through vehicles could be determined. Causes for any undue delay in the responsiveness of the drivers and vehicles to the green traffic signal were also observed.

TABLE 2.4. SUMMARY OF THE OBSERVED HEADWAY RESULTS*

Coded intersection No.	Type of Lane	Lane Width	Sample Size	Average headway after the 4th vehicle in each light cycle (sec)	Mean Headway (sec)
	median lane	7'3"	425	2.124	
IS.3	centre lane	9'0"	298	2.195	2.0944
	kerb lane	9'9"	-	-	
	median lane	6'6"	152	2.179	
IS.4	centre lane	9'6"	133	2.018	2.1035
	kerb lane	10'0"	-	-	
IS.5	median lane	10'9"	239	2.214	2.2390
	centre lane	10'3"	200	2.268	
	median lane	10'6"	-	-	
IS.6	centre lane	9'6"	549	1.884	2.1596*
	kerb lane	9'10"	311	2.160	
	median lane	9'6"	504	1.701	
IS.7	centre lane	9'6"	454	1.839	2.0483*
	kerb lane	9'6"	-	-	
	median lane	9'3"	278	2.067	
IS.11	centre lane	9'3"	347	2.078	2.0731
	kerb lane	8'3"	-	-	
	median lane	9'5"	390	2.038	
IS.12	centre lane	11'5"	428	1.975	2.0055
	kerb lane	10'0"	-	-	
	median lane	9'6"	552	2.186	
IS.13	centre lane	11'0"	489	2.142	2.1654
	kerb lane	10'6"	-	-	
	median lane	10'0"	731	2.100	
IS.14	centre lane	10'6"	519	2.020	2.0665
	kerb lane	10'10"	-	-	
	outer lane	8'6"	898	2.188	
IS.15	centre lane	9'0"	718	2.162	2.1760
	kerb lane	10'3"	-	-	
	outer lane		501	2.310	
IS.17	centre lane		380	2.360	
	kerb lane		-	-	

*The centre lane headways were not included.

TABLE 2.5. STARTING LOST TIME OF PASSENGER CARS

Coded intersection No.	Average Headway for the First 4 Vehicles (sec)	Average Headway After the 4th Vehicle (sec)	Starting Lost Time (sec)
3	2.574	2.174	1.600
3	2.303	2.029	1.096
4	2.310	2.160	0.600
4	2.462	2.060	1.608
5	2.385	1.894	1.964
5	2.444	2.185	1.036
8	2.428	2.150	1.112
9	2.397	2.081	1.264
11	2.262	2.079	0.732
11	2.213	2.062	0.604
12	2.425	2.013	1.648
12	2.272	2.074	0.792
13	2.387	2.150	0.948
13	2.216	2.186	0.120
14	2.380	2.000	1.520
14	2.240	2.100	0.560
15	2.460	2.180	1.120
15	2.610	2.160	1.800

Ten recorders and one supervisor were assigned to this project for five days of data collection. Each recorder, by means of a stopwatch, recorded the elapsed time from the start of the green light to the instant of vehicle motion for his assigned vehicle in the platoon. Data were collected at A.M. peak, off peak, and P.M. peak traffic periods.

Results

The following characteristics were revealed from the study:

(1) No appreciable difference could be found in response of left-turn lanes when compared with through lanes.

(2) The relationship between vehicle position and average time of starting from a stopped position approximated a straight line. Figure 2.2 shows the relationship for the 85 percentile time (15 percent of vehicles exceeded the time shown on the graph). Figure 2.3 shows the relationship for the average time.

(3) There was no significant difference of the average starting response of successive vehicles at peak hour and off-peak hour.

(4) Of the 6,615 samples in positions 1 through 10, 6.17% (444) were delayed by driver inattention, 0.27% (18) were delayed by vehicle failure and 0.23% (15) by other causes.

BETZ AND BAUMAN [REF. 12]

Mathew J. Betz and Richard D. Bauman investigated driver characteristics at intersections in Arizona .

Data Collection

Data were collected on weekdays in the period of Sept. 1964 to April 1966. All data were collected on color film. Two observers, upon commencement of filming, noted the color of the leading car in each queue and the length of the queue. A daylight rear projection screen with an acetate cover was used to view the developed film containing the headway and gap information. Headways were measured with reference to an extension of the curb line which was nearest and parallel to the stop line. All reference lines and measurement points were drawn on the acetate cover.

The film containing the license-plate information was viewed on a screen adjacent to the rear projection screen. Both films were viewed concurrently. One technician read the license-plate of the vehicle and thus classified the origin of the vehicle (in-state or out-of-state) while at the same time the other technician measured the headway or gap of the same vehicle. A frame counter on the projector was used to obtain the time base. Occurrences were estimated to the nearest quarter frame (0.15 sec.).

A total of five intersection approaches were studied. All intersections were normal at-grade designs with right-angle crossings. All locations were signalized.

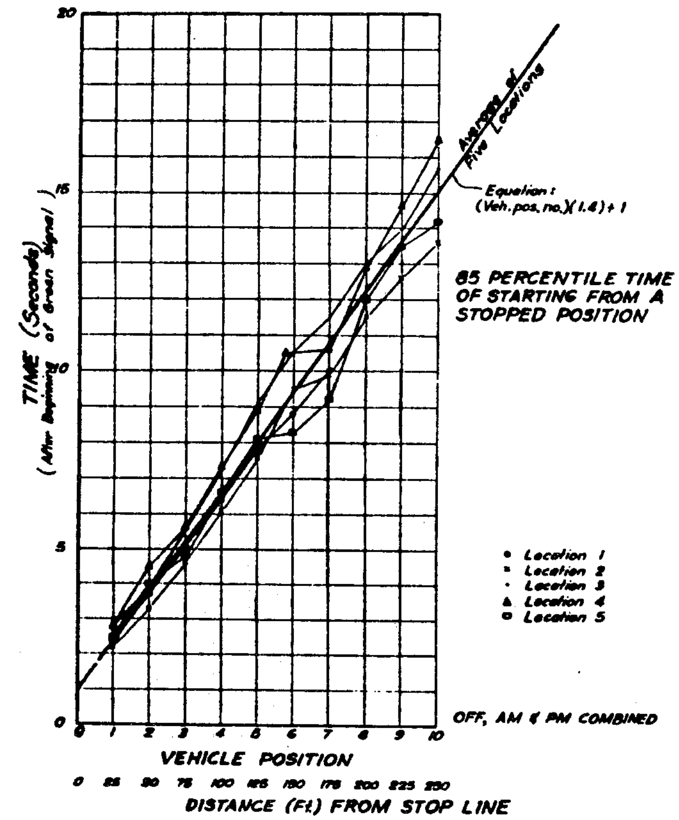

Figure 2.2. Source: Ref. 11

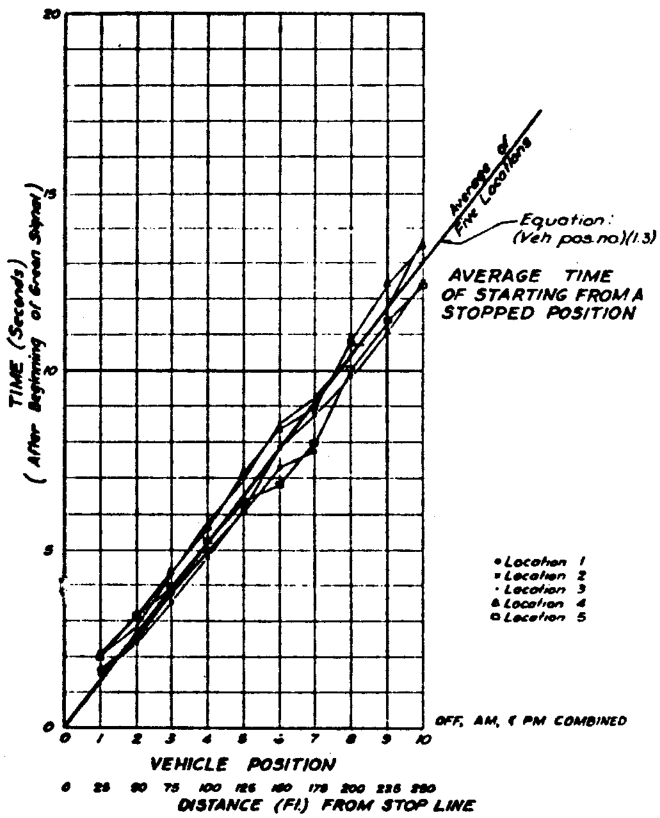

Figure 2.3. Source: Ref. 11

The data are only for vehicles in the inside lane (the through lane nearest the centerline) travelling during off-peak periods. Thus, the headways are based on lanes with uninterrupted through movements. In all cases the time required for the first vehicle in the queue to respond to the light change and advance to the reference line is not included because the distance to the reference line varies from intersection to intersection.

Analysis

The population size of the headway data was 2,517 vehicles. It was determined that there was no significant difference between headway characteristics at four of the approaches tested.

The headways of the second through the eighth vehicle in the queue are 2.48, 2.36, 2.27, 2.11, 2.18, 2.14, and 2.01 seconds respectively, as they can be read from Figure 13, p. 48, Ref. 9, for the 'in-state' category of vehicles.

ANCKER, GAFARIAN, AND GRAY [REF. 13]

In 1967, C. J. Ancker,Jr., A. V. Gafarian and R. K. Gray investigated the oversaturated signalized intersection.

Collection of Data

A simple electronic system was developed with a photocell as the vehicle sensor. A conventional audio tape recorder was selected to serve as the time base and field recording device. An observer, located on a cliff overlooking the test site with a "walkie-talkie", was able to relay the equipment operator information supplementing the electronically recorded data. It was discovered that a single photocell placed in the center of the lanes did not provide an adequate sensor for all vehicles in the lane.

Headway time for the first car in line is defined as the time between onset of the go phase and the instant the front edge of the vehicle crosses the photoelectric cell, two feet downstream of the stop line. Headway times for each of the remaining cars in the queue are the intervals between crossings of the photoelectric cell by front edges of successive vehicles.

Time headway measurements were made when an intersection was oversaturated, i.e., when the queue before the intersection is only partly served during a single green phase. The data were collected from the straight-through middle lane of a three-lane road with no downstream bottleneck. Measurements were made on eight weekdays from approximately 7:00 A.M. to 8:45 A.M. in order to restrict the sample,as far as possible, to commuters and included 406 cycles. Headways were measured for those cars that effectively stopped completely.

Even though large queues of stopped cars would occur it was difficult to get sufficiently large samples on headways beyond 11. The reason for this is that by the time the cars far back get to the intersection, the original queue is destroyed by a lane change, by a car moving into or out of the middle lane of interest. When this occured, observations ceased.

Error Analysis

There are several sources of error in the present method. Tests showed that the magnetic tape recorder contributed a ±2 msec variation in timing over time intervals comparable to the vehicle headways. Also the circuitry used in conjunction with the traffic controller to record the onset of the green phase introduced a relatively constant delay of 10 msec. Errors due to variations caused by a change in the ambient light during the course of a series of measurements were eliminated by manually adjusting the photocell bias with a potentiometer to maintain a constant voltage across the unobstructed cell.

Also, the location of the car relative to the photosensor when it was triggered differed as a function of vehicle velocity. As a result of this, it was felt that the first two or three headways may be as far off as 3 digits in the 2nd decimal, but that all the subsequent headways were much more accurate because of the increase in absolute velocities of the cars.

Data Analysis

Some statistics for the first eleven vehicles are shown in Table 2.6.

The hypothesis of independence of successive pairs seemed well justified and was accepted, after several tests (contingency table, regression, Corner, Spearman rank correlation, and correlation coefficient tests) were run.

The hypothesis of the homogeneity of trend-eliminated distributions for positions three through eleven was accepted at a significance level greater than 0.995.

Also, the hypothesis that populations 7 through 11 have equal means was accepted at less than 0.01 significance.

The shifted Erlang density function was fitted to the data.

A standard chi-square test was used for goodness-of-fit tests. The results of this test clearly show that all samples pass the test and the hypothesis that each comes from the maximum likelihood Erlang density function was accepted.

TABLE 2.6. SOME STATISTICS FOR THE FIRST ELEVEN VEHICLES
(All Times Are In Seconds)

Position	Sample size	Sample mean	Sample variance	Minimum	Maximum	Range
1	371	2.72	0.49	0.56	5.78	5.22
2	347	2.52	0.42	1.32	7.03	5.71
3	352	2.05	0.26	1.05	4.29	3.24
4	316	1.96	0.28	1.11	4.30	3.19
5	276	1.84	0.24	0.85	3.28	2.43
6	243	1.81	0.25	0.88	3.89	3.01
7	224	1.77	0.26	0.95	4.37	3.42
8	187	1.70	0.25	0.63	3.66	3.03
9	154	1.64	0.24	0.83	3.95	3.12
10	135	1.69	0.26	0.56	3.29	2.73
11	115	1.70	0.25	0.84	3.35	2.51

The results of the statistical analyses of the data are that the following hypotheses were accepted:

(1) Headways are independent random variables.

(2) Shifted Erlang density functions are sufficiently good for all positions.

(3) The means decrease monotonically to position seven and do not differ significantly beyond the seventh position.

(4) The density functions for position one, two, and three differ with position three and later positions differing only in location.

CARSTENS [REF. 15]

Robert L. Carstens in 1971 studied some traffic parameters at signalized intersections .

Data Collection

Data gathered from 2,093 changes of a signal aspect from red to green were analyzed for this study. Stopped vehicles were counted, trucks were distinguished from passenger cars, and the direction of movement of every vehicle was noted for each signal change. The elapsed time was determined between the beginning of the green phase and passage over the stop line of the last vehicle that has been stopped awaiting the green. Most of these data were obtained using manual counts and stop watches, the rest from analysis of time-lapse photographs. It is believed that accuracy of the results from the two methods is comparable.

The study sites were 16 lanes in eight approaches at four signalized intersections in Ames, Iowa. The effects of pedestrians and parking (present only on one approach) were insignificant at the times of observation. Posted speed limits on the far side of the approaches varied from 25 to 35 mph. Differences among the results for various lanes, approaches, and intersections were insignificant so that the results reported are a composite of those for all 2,093 changes.

Analysis of Data

The results are as follows:

(a) Average headway at stopline: 2.29 sec. per straight-through passenger car.

(b) Starting delay: 0.35 sec. for one vehicle of any type, 0.55 sec. for two vehicles and 0.75 sec. for three or more vehicles.

The total time following the start of a green signal for various lengths of queues of stopped straight-through passenger cars to cross the stop line was: one, 2.64 sec; two, 5.13 sec; three, 7.62 sec; four, 9.91 sec; five or more, with additional 2.29 sec for each succeeding car.

The results of this study refer to times that the front axles cross a stop line.

BERRY AND GANDHI [REF. 16]

Donald S. Berry and P. K. Gandhi investigated the headway approach to intersection capacity in 1973 [Ref. 16].

Data Collection

Data were taken at one 18-ft approach to a three-phased signalized intersection. There were no left-turning traffic, no opposing flow, no vehicles parked, standing, or stopping, practically no commercial vehicles, and practically no pedestrians to interfere with right-turning vehicles. Those few cycles with buses and pedestrian interference were excluded from the study. All data were taken on weekdays during the evening peak period when about 95 percent of the cycles were loaded. Cycle length was 60 sec. with 17 sec. green and 3 sec. yellow.

Starting delays were measured with a stopwatch that made one revolution in 10 seconds. The stop line, which was 24 ft from the intersection as determined by a prolongation of the curblines, was used as the reference line. A second stopwatch was started as the rear wheels of the first vehicle crossed the stop line and was stopped when the last vehicle in the compact platoon crossed the stop line with its rear wheels. The elapsed time, T, shown on this second stopwatch, was then divided by the number of vehicles, less one, to determine the average headway, h, for the compact platoon. Results for 14 peak periods of data collection are given in Table 2.7.

Statistical tests were performed to examine consistency among results for days having the same conditions. Tests were conducted at the 1 percent significance level under the null hypothesis that the mean headways came from the same population. For the 4 "dry-night" days, the mean values for each of the 4 days were not significantly different from the 4-day average. For the 5 "dry-daylight" days, 1 day had a mean headway significantly different from the 5-day average. Also, mean headway values for the two "wet-night' studies were significantly different from each other, perhaps because of the differences in intensity of rainfall.

Data Analysis

Statistical tests were also performed using the null hypothesis that the mean headways for each set of adverse weather and visibility conditions were the same as for dry-daylight conditions. Comparisons were made between dry-night and dry-daylight conditions and between wet-night and dry-night conditions. The null hypothesis had to be rejected for all significance levels above 1 percent in both cases, indicating that adverse weather significantly increased headways.

TABLE 2.7. RESULTS OF 14 PEAK-PERIODS

Weather, Visibility, Date	Number of Loaded Cycles	Starting Delay D (sec)	Headways average (sec)	Headways st. deviation
dry-day				
03/22/71	60	2.379	1.107	0.047
03/23/71	60	2.607	1.086	0.049
03/25/71	60	2.490	1.074	0.039
03/29/71	60	2.485	1.071	0.047
04/15/71	60	2.457	1.089	0.054
Average		2.483	1.085	0.049
dry-night				
11/17/70	60	2.434	1.167	0.098
11/18/70	60	2.483	1.178	0.083
11/21/70	60	2.555	1.176	0.040
11/22/70	60	2.458	1.178	0.084
Average		2.482	1.175	0.099
wet-night				
11/16/70	60	2.670	1.256	0.070
02/04/71	60	2.762	1.318	0.135
Average		2.716	1.287	0.112
Snow-day				
03/18/71	60	2.714	1.282	0.059
03/19/71	60	2.683	1.255	0.063
Average		2.698	1.269	0.062
Snow-night				
02/12/71	60	2.638	1.283	0.064

KING AND WILKINS [REF. 17]

In a study of the relationship between signal design and departure headways, Gerhart F. King and M. Wilkinson recorded discharge headways of straight-through passenger cars at 39 signalized intersections in 5 states, in 1976.

Collection of Data

Queue discharge headway data were recorded by manual input to a chart recorder. The observer pressed a button when the signal changed to green and when a vehicle passed the stop line (or a screen line established as the location of the front wheels of the first car in queue). Data were recorded for all passenger cars on each cycle that (1) Were stopped in queue at the beginning of the green interval; (2) Proceeded straight through the intersection; and (3) Were not impeded by pedestrians, cross traffic, or opposing left turners. Data were collected for approximately 30 cycles at each location.

This manual input method has an element of error because of the observer's reaction time. To compensate for this error the reaction time was assumed to be almost uniform for all inputs. This assumption was validated by film. Queue discharge data were manually collected at one location by using a chart recorder while, at the same time, the queue discharge process was filmed at 5 frames/sec. Both sets of data were reduced, and the queue discharge distribution was determined. No significant differences between the two distributions were detected.

Data Anslysis

The filmed data were reduced on a frame-by-frame basis. The queue discharge headway data recorded on the charts were reduced by measuring the times between the onset of green spike and the first vehicle spike. Then the time between each succeeding vehicle passage was measured. The sample size at each queue position decreased from approximately 30 at the first position to zero at more distant queue positions.

The computed mean and standard deviation of the discharge headways for each queue position for each approach are shown in Table 2.8. The observed general trend was a decrease of discharge headway as queue position increases and then a leveling off to approximately 2.2 seconds by the fifth position.

FAMBRO, MESSER, AND ANDERSEN [REF. 18]

D. B. Fambro, C. J. Messer, and D. A. Andersen measured unprotected left-turn headways at signalized intersections, in 1977.

TABLE 2.8. OVERALL ANALYSIS OF QUEUE DISCHARGE HEADWAYS

Location No.	Position 1			Position 2			Position 3			Position 4			Position 5			Positions 6 to 8		
	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.
1	28	2.26	0.55	28	2.49	0.76	23	2.70	0.64	22	2.64	0.87	15	2.64	0.83	22	2.58	0.89
2	31	2.32	0.79	31	2.90	0.81	29	2.48	0.51	24	2.22	0.52	15	2.22	0.77	18	2.17	0.80
3	29	3.36	0.97	29	3.17	0.78	28	2.72	0.64	17	2.60	0.40	10	2.22	0.78	4	2.05	0.55
4	28	3.67	1.56	28	3.09	0.83	28	2.71	0.52	16	2.51	0.68	7	2.59	0.56	1	2.00	0.00
5	36	2.30	0.88	35	3.00	0.68	30	2.40	0.55	23	2.20	0.58	17	2.29	0.69	20	2.19	0.63
6	33	2.31	0.69	33	2.78	0.60	22	2.27	0.31	11	2.79	0.63	4	1.85	0.39	2	2.45	0.35
7	29	2.73	0.92	29	3.48	0.73	23	2.53	0.63	18	2.52	0.67	13	2.33	0.55	4	2.30	0.73
8	30	2.87	1.26	29	3.04	0.79	20	2.65	0.60	16	2.27	0.67	2	1.85	0.21	0	-	-
9	30	2.85	1.29	29	3.40	0.67	20	3.08	0.62	7	3.04	0.87	5	2.26	0.24	3	2.03	0.49
10	30	2.82	0.95	30	3.09	0.66	28	2.89	0.75	23	2.53	0.62	14	2.38	0.88	10	2.41	0.60
11	31	2.58	1.02	29	3.08	0.85	20	2.69	0.73	12	2.65	0.62	2	2.50	0.71	0	-	-
12	41	1.80	0.66	41	2.79	0.66	24	2.23	0.51	14	1.95	0.52	7	1.76	0.38	7	1.63	0.72
13	30	2.19	0.70	30	2.79	0.56	30	2.16	0.57	24	2.13	0.43	19	1.98	0.38	19	2.07	0.76
14	30	3.31	1.33	29	3.47	0.90	28	2.63	0.91	22	2.47	0.45	11	2.32	0.67	4	1.93	0.46
15	27	2.61	1.17	27	2.70	0.66	27	2.51	0.54	20	2.57	1.26	10	2.23	0.45	4	2.83	1.09
16	37	2.64	0.79	32	2.53	0.43	18	2.57	0.81	9	3.21	1.09	4	2.23	0.17	2	2.70	0.28
17	33	2.55	0.96	32	2.75	0.50	31	2.45	0.59	28	2.22	0.37	18	2.34	0.53	14	2.49	0.65
18	30	2.14	0.69	20	2.81	0.58	11	2.56	0.55	7	2.64	0.82	4	2.00	0.36	2	1.50	0.71
19	20	2.13	0.98	20	2.88	0.69	20	2.25	0.58	16	2.08	0.46	12	1.99	0.46	21	2.20	0.66

TABLE 2.8. OVERALL ANALYSIS OF QUEUE DISCHARGE HEADWAYS (CONTINUED)

Location No.	Position 1			Position 2			Position 3			Position 4			Position 5			Position 6 to 8		
	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.	No.	Mean (sec)	Std. Dev.
20	31	2.78	1.44	31	3.25	0.58	31	2.62	0.64	21	2.22	0.55	15	2.21	0.68	9	2.01	0.35
21	29	2.97	1.16	29	3.29	0.76	29	2.30	0.32	22	2.52	0.89	14	2.05	0.30	8	2.54	0.58
22	31	3.33	1.22	31	3.05	0.81	31	2.44	0.51	22	2.66	0.54	12	2.22	0.67	7	2.27	0.56
23	30	3.07	1.31	30	2.87	0.74	30	2.40	0.75	21	2.24	0.55	11	2.57	0.81	2	1.55	0.35
24	29	2.94	0.87	29	3.22	0.78	29	2.36	0.72	20	2.20	0.61	13	2.12	0.56	8	2.06	0.50
25	28	2.24	0.71	23	3.42	0.68	17	3.21	1.06	11	3.05	0.65	7	2.51	0.61	2	2.05	0.21
26	30	2.11	0.90	30	2.79	1.39	28	2.39	0.64	26	2.30	0.62	14	2.19	0.64	13	2.19	0.50
27	27	2.09	0.78	27	2.66	0.68	25	2.30	0.71	24	2.27	0.60	22	2.25	0.82	34	2.08	0.55
28	35	2.28	0.87	35	2.83	0.80	31	2.42	0.81	30	2.18	0.47	20	2.25	0.57	19	1.89	0.47
29	30	2.15	0.73	19	3.05	0.52	13	2.79	0.65	9	2.16	0.30	2	2.00	0.99	1	1.50	0.00
30	30	2.82	1.43	19	2.93	0.58	11	2.45	0.45	7	2.39	0.43	4	1.93	0.30	1	2.10	0.00
31	30	2.54	1.03	20	3.30	0.58	13	2.79	0.70	7	2.73	0.66	0	-	-	0	-	-
32	30	4.02	1.04	18	3.41	0.97	13	2.84	0.62	5	2.44	0.50	3	2.87	0.46	0	-	-
33	30	2.96	1.14	21	3.10	0.98	17	2.76	0.65	9	2.02	0.45	4	2.23	0.53	4	1.85	0.42
34	30	2.72	1.01	19	3.62	0.93	14	2.34	0.52	8	2.05	0.48	6	2.33	0.37	3	2.30	0.90
35	33	2.57	1.31	23	3.04	0.55	19	2.57	0.59	10	2.23	0.70	4	2.25	0.24	2	2.30	0.71
36	31	2.23	0.61	27	2.99	0.79	20	2.29	0.56	9	2.22	0.47	5	1.60	0.20	5	2.22	0.60
37	31	2.25	1.15	27	2.85	0.50	20	2.42	0.51	14	2.18	0.36	9	1.83	0.43	10	1.96	0.46
38	30	2.04	0.88	30	2.63	0.52	27	2.37	0.50	25	2.15	0.54	23	1.99	0.41	59	1.96	0.55

Data were collected at twelve intersections by using a portable videotape recording system. Two members of the research team were required to operate the system in the field. A playback unit and a monitor were used to replay the tapes. A stopwatch was used to time vehicle movements and signal intervals.

Headways between left-turning vehicles were found by measuring the time between completion of the turn movement of successive vehicles turning through the same gap. Only those cycles during which more than one vehicle turned through the same gap resulted in usable data. The average headway for vehicles turning from left-turn lanes was 2.48 seconds for the 311 headways measured at the six different locations. Slightly higher were the average turn headways for the intersections without left-turn lanes, 2.62 seconds from the 35 measured headways.

STEUART AND SHIN [REF. 19]

Gerald N. Steuart and Bu-Yong Shin studied the effect of small cars on the capacity of signalized urban intersections, in 1978.

The size of passenger cars, the major independent variable in this study, has a wide range of possible values. The passenger car fleet was stratified into three vehicle size categories as follows:

- Small car: a passenger vehicle having four cylinders and a vehicle length in the range of 12 feet (Volkswagen, Toyota, Capri, etc.).
- Medium-sized car: a passenger vehicle having six or eight cylinders and a vehicle length in the range of 15 feet (Nova, Mustang, Valiant, etc.).
- Full-sized car: American standard passenger cars having vehicle length in the range of 17.5 feet (Impala, LTD, Newport, etc.).

Data Collection

Twelve signalized intersections were chosen for data collection in the metropolitan area of Toronto, with nine intersections in the central business district and three in the suburbs. The data obtained were intended to reflect the variety of operating conditions at urban intersections.

A tape switch was placed close to (6 feet beyond) the stop line of an approach lane to record the passing of a vehicle's axle. The tape switch was constructed with two metal tapes separated by punctured elastic insulation to allow metal contact upon compression. At some intersections a second tape switch was placed a known distance from the first to provide sufficient information to determine the speed of vehicles. The tape switches in combination with manual switches were used to activate a standard multichannel event recorder.

The stop line was chosen as the location to record headways for two reasons. The first was to make the data base comparable with the headways recorded by Miller in Australia. Secondly, the stop line was chosen on the assumption that any relationship between vehicle size and headway would be most evident closest to the stationary queue before the additional factors such as the driver's desired speed would add more variation to the observed headways.

At each location on the lane being studied, two observers were used to record the following information for every headway during saturation flow: (1) Type of vehicle (including the passenger car designation: small, medium, large), (2) Movement of vehicle (through, right turn, left turn), (3) Position of vehicle in the queue, (4) Type of previous vehicle, (5) Movement of previous vehicle, (6) Headway to previous vehicle (seconds), and (7) Speed at stop line (at some locations).

The data allow time headways to be measured from the successive passage of their front axles or rear axles of the vehicles in the traffic stream. The analysis uses both measurements in an attempt to isolate the possible effects of vehicle size on headways during saturation flow. Next Table 2.9 shows the headways for different groups. The analysis shows that vehicle size has a significant effect on headway, a finding which is contrary to the research results with small cars in a free flowing traffic stream.

Analysis of Data

The data base was reduced somewhat for this analysis because of some inconsistencies caused primarily by definition problems and small sample sizes. The medium sized car was difficult to define and preliminary analysis showed inconsistent results for this vehicle size. The data for medium sized cars was omitted from the remainder of the analysis and this vehicle size category only served to separate the small- and large-sized car categories.

The headway of a queue leader is taken to be the time from the beginning of the green period to the passing of the rear axle of the queue leader over the control point. The average headway of the first vehicle in the queue is 2.75 seconds for a small car and 3.17 seconds for a full-sized car. A test of the hypothesis that these mean values are equal was not accepted and with the large sample size established that the average headways were significantly different.

The headway of vehicles after the queue leader is dependent on the size of the preceding vehicle. Table 2.9 summarizes the data according to position in queue for small cars and full-sized cars using the rear axle to rear axle definition of headways. Early vehicles in the queue, before the fifth vehicle, show a significantly smaller headway between two small cars than between two full-sized cars. The results indicate that position in the queue has a lesser effect on small cars than on other vehicle combinations. Vehicles positioned later than ninth in the queue are assumed to have headways independent of the vehicle size. The average headway for all cars later than ninth in the queue, independent of vehicle size, is 1.94 seconds.

TABLE 2.9. HEADWAYS (REAR AXLE TO REAR AXLE) CLASSIFIEDBY POSITION IN QUEUE

Position in queue		Small car following a small car	Small car following a full-sized car	Full-sized car following a small car	Full-sized car following a full-sized car
	Mean headway	2.07	2.31	2.30	2.38
2	Sample variance	0.19	0.46	0.18	0.37
	Sample size	6	19	15	43
	Mean headway	1.88	2.13	2.23	2.44
3	Sample variance	0.36	0.12	0.11	0.36
	Sample size	7	12	13	48
	Mean headway	1.88	2.09	2.15	2.20
4	Sample variance	0.22	0.44	0.14	0.32
	Sample size	7	14	11	56
	Mean headway	1.87	2.05	2.01	2.09
5-9	Sample variance	0.33	0.33	0.33	0.25
	Sample size	16	54	51	192
	Mean headway	2.13	1.78	2.04	1.95
>10	Sample variance	0.52	0.26	0.31	0.32
	Sample size	11	31	26	143

AGENT AND CRABTREE [REF. 20, 21]

In 1982 Kenneth R. Agent and Joseph D. Crabtree made an analysis of saturation flow at signalized intersections.

Data Collection

Data collection consisted of measuring time intervals between the signal turning green and the rear wheels of each vehicle in the queue crossing the stop bar. The stop bar was selected as the screenline because it was felt that it would give the best and most consistent results.

The majority of data was collected in Lexington, Kentucky. Approaches were selected so that a range in values for the variables would be available for data analysis. In other words, an attempt was made to select approaches in different areas having a range in such variables as lane width and gradient. Various cities across the state were selected for data collection. All necessary measurements were made at subject intersections. Approach grade was obtained using an Abney hand level meter.

For each vehicle, the time from start of green to the rear wheels crossing the stop bar was recorded. A description of the vehicle and/or its action was recorded when appropriate. Vehicles that were interrupted were excluded from the subsequent analysis.

Data were collected for each vehicle in the queue and recorded as a function of the vehicle's queue position. When vehicles changed lanes or entered the queue from an adjacent driveway, thereby disrupting normal movement, data collection was discontinued for that cycle. Data were collected only for those vehicles that were part of the queue when the signal indication turned green or became a part of the queue before reaching the stop bar.

All times for individual vehicles were obtained with a split/cumulative timer that displayed time to the nearest 0.01 second. The timer had a digital display which was easy to read. The timer was started at the beginning of green and a button was pushed when each vehicle crossed the stop bar. Elapsed time since starting of the timer was displayed for each vehicle and noted to the nearest 0.1 second on the data sheet.

Data Analysis

Data were transferred from data sheets to a computer file. The field for "headway" on the data records was calculated by taking the time recorded when that vehicle's rear wheels crossed the stop bar and subtracting the corresponding time for the preceding vehicle. The headway for the first vehicle was the time between the onset of green and the rear wheels of that vehicle crossing the stop bar.

For two situations, the headway field was left blank because of irregularities in traffic flow. One was when the first vehicle stopped beyond the stop bar. In that case, the headways for the first two vehicles were left blank. The other situation was when an interruption code was encountered. In that case, headways were left blank for the interrupted vehicle and the following vehicle.

Analysis was performed by limiting values of all but one important variable, allowing that variable to vary, and observing the effect of that variance on saturation flow. The analysis was a careful, step-by-step process, with results of each step affecting limitations applied to successive steps. Where necessary, assumptions were made and later verified. If assumptions were found to be invalid, certain steps of the analysis were repeated with necessary corrections. Additional data also were collected to fill in gaps that became apparent during analysis. The data file contained a total of approximately 47,000 headways, of which approximately 32,000 were collected in Lexington.

RESULTS

Beginning Lost Time

This analysis included all through vehicles at locations in Lexington with grades of -3.0 to +3.0 percent, speed limits of 45 mph, and cycle lengths of 90 to 120 seconds. Results of this analysis are shown in Table 2.10.

TABLE 2.10. BASE VALUE FOR BEGINNING LOST TIME

Queue Position	Total Headways	Average Headway (seconds)
1	478	3.03
2	476	2.65
3	474	2.47
Above 3	3,306	2.25

Vehicle Position in the Queue

The first step in data analysis was to determine how average headway varied with vehicle queue position. The purpose of this first step was to determine how many initial vehicles had to pass at the beginning of a green phase before headways became fairly constant.

For these summaries, data for which lane width was 10 to 15 feet and grade was from minus three to plus three percent were included. The first summary prepared was for through (non-turning) passenger cars in Louisville and Lexington. The results of that summary, presented in Figure 2.4, indicated that headways became fairly constant after the first three vehicles, i.e., beginning with vehicle number four.

Lane Width

This analysis excluded data taken in cities with populations under 20,000, locations in a CBD, locations with heavy pedestrian activity, and locations with parking on the approach within 200 feet of the stop bar. Included were approach grades of minus three to plus three percent and speed limits of 35 to 45 mph. Only through and left-turning passenger cars were included.

The analysis indicated that lane width did not have an effect on saturation flow for lane widths of 10 feet or more. For lane widths between nine and ten feet, a five percent reduction in saturation flow was found compared to lane widths of 10 or more feet. No lane widths below nine feet were observed.

Gradient

Summaries for this analysis included locations having lane widths from 10 to 15 feet and speed limits of 35 to 45 mph. The first summary was for through and left-turning passenger cars. Results are shown in Table 2.11. Increasing grade increased average headway, although the top grade category (grade greater than three percent) did not show the expected increase.

City Size

Average headways decreased with increasing population. The decrease amounted to an eight percent difference over a population range of 20,000 to 500,000. However, for populations under 20,000, average headway values increase substantially.

Vehicle Type and Turning Maneuvers

Lost time was 21 percent higher for trucks and buses than for passenger cars. Left-turning vehicles (at locations with exclusive left-turn phasing) had an eight percent higher lost time than through vehicles, and right-turning vehicles had a five percent lower lost time.

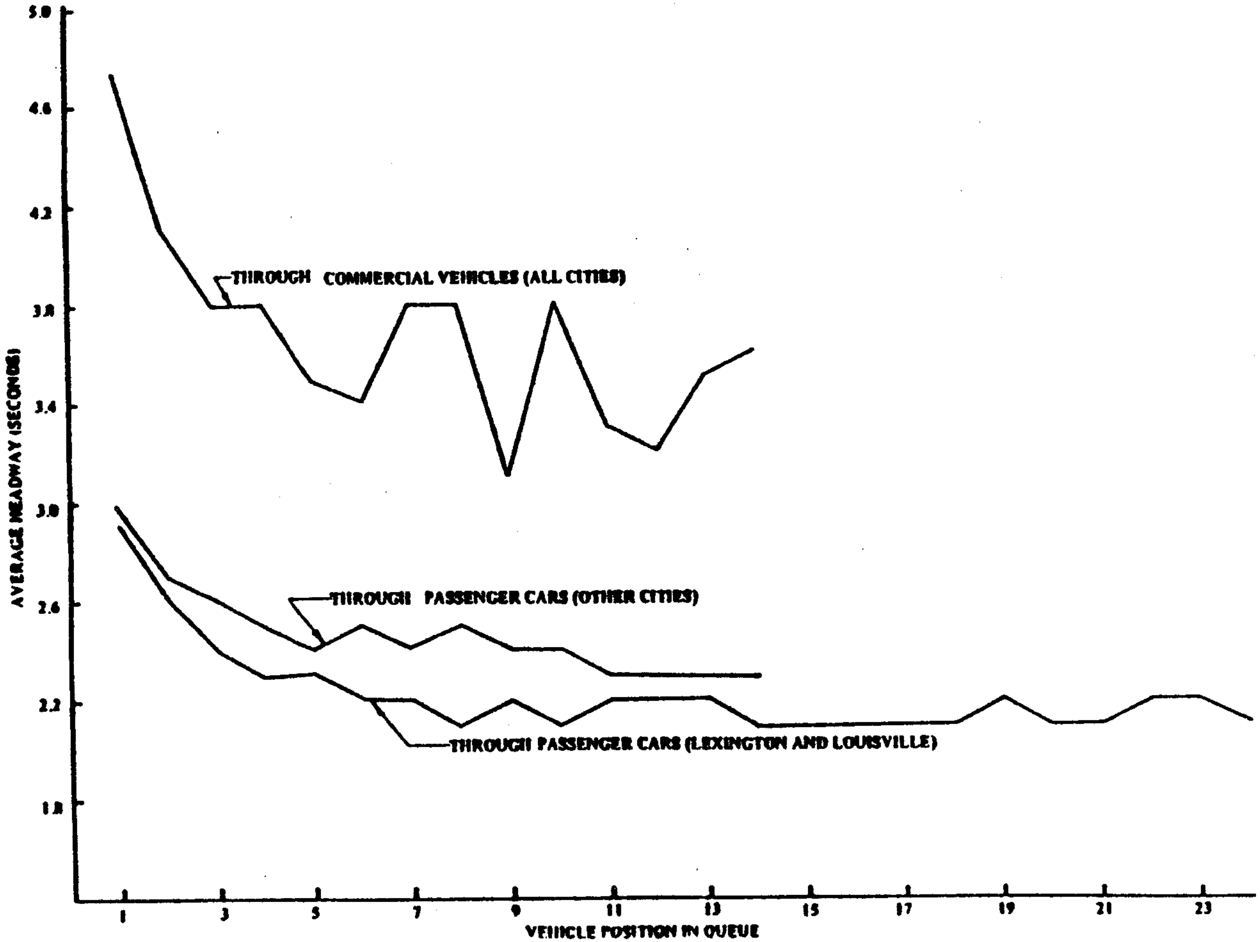

Figure 2.4. Average headway versus vehicle position in queue.

Source: Ref. 20

TABLE 2.11. EFFECT OF GRADIENT ON SATURATION FLOW

Grade (percent)	Total Headways	Average Headway (seconds)
PASSENGER CARS		
Less than -3	1,481	2.09
-3 to -1.1	4,154	2.12
-1 to +1	10,763	2.19
+1.1 to +3	1,465	2.23
Greater than +3	798	2.22
COMMERCIAL VEHICLES AND BUSES		
Less than -1	88	3.10
-1 to +1	361	3.48
Greater than +1	51	3.48

Speed Limit

Data were collected for city size, location in city, lane width, grade, pedestrian activity, and parking. Only through and left-turning passenger cars were included. Results of that summary are in Table 2.12. As speed limit increased, saturation flow also increased. However, the effect was relatively small, amounting to approximately four percent over the range from 35 to 55 miles per hour.

TABLE 2.12. EFFECT OF SPEED LIMIT ON SATURATION FLOW

Speed Limit (mph)	Total Headways	Average Headway (seconds)
35	4,616	2.19
40	1,042	2.15
45	10,726	2.17
50	489	2.13
55	391	2.11

Peak Versus Non-Peak Conditions

No significant difference was observed.

LU [REF 22]

In 1984 Yean-Jye Lu studied left-turns at signalized intersections .

Data Collection

A time recorder and stop watches were used to collect left-turn discharging headways at a protected signalized intersection in Austin, Texas. Observations were made by a person at one corner of the intersection with a stopwatch during the afternoon peak hours.

Vehicle size for this study is classified into three categories (1) Small car:four-cylinder vehicle; (2) Large car: six- or eight-cylinder vehicle; and (3) Truck or bus.

Results

The perception-reaction time (PRT) for this study is the interval from the onset of the green light until the first vehicle in the queue moves. The average PRTs for small cars, large cars, and trucks or buses are 1.21 sec, 1.10 sec, and 1.24 sec, respectively. Since the difference was significant at the 0.19 level, there is no evidence to claim that the PRTs are significantly different between vehicle sizes at the 95 percent confidence level.

Queue discharge headway is the time interval from the moment the front bumper of the leading vehicle of a queue reaches the intersection line until the front bumper of the following vehicle reaches the intersection line. The first vehicle in the queue required an average discharge headway of 2.43 seconds which includes an average perception-reaction time of 1.17 seconds and an average traveling time of 1.26 seconds for traveling from the stop line to the intersection line. The second vehicle required the longest discharge time of 2.62 seconds while the third and fourth vehicles required 2.10 and 2.09 seconds respectively. Moreover, the average queue discharge headway reached a steady state value of about 1.8 seconds when the queue position was fifth or greater.

LEE AND CHEN [REF. 23]

Entering headway at signalized intersections in a small metropolitan area was investigated by J. Lee and R. L. Chen in 1986.

Data Collection and Reduction

Sixteen signalized intersections along four major streets in Lawrence, Kansas were selected for study.

For all selected intersection approaches, vehicle movements were recorded by using a portable video camera system that has a built-in timer with 0.1-sec accuracy. All field videotaping of traffic movements was conducted from June to September 1984. The actual filming at each intersection included 2 hr each day with 1 hr each for a.m. and p.m. peak hours. Within the filming period, half an hour was spent on one approach and the other half for the opposite approach of the major streets. Approximately 5,000 single-lane traffic platoons entering the intersections were recorded.

The tapes were first examined in the laboratory to screen out the cases that were not suitable for this study, including the following: platoons within which vehicles did not stop before entering an intersection; platoons with trucks; platoons with turning vehicles; and platoons in which the movements of cars were impeded by pedestrians, cross traffic, or turning vehicles. In other words, only platoons containing unimpeded, straight-through passenger cars stopped before entering an intersection were considered valid cases. The valid cases, totaling 1,899 traffic platoons, were later viewed on television screen to extract the entering headway values.

For the first vehicle of a queue, its entering headway was taken to be the time elapsed between the start of a green indication and the time at which the car's rear bumper cleared the stop line. For the remaining cars in the queue, the entering headway values were taken to be the elapsed time, rear bumper to rear bumper, as the successive vehicles passed an intersection stop line.

Data Analysis and Major Results

The first step in the analysis was to derive the basic statistics out of all the entering headway data collected. The number of vehicles in the queue varied from 1 to 12. The mean entering headways (in seconds) for vehicles 1 through 12 and other statistics are summarized in Table 2.13.

It is to be noted that because the study locations are limited in their variety of conditions, the results on which the statements below are based may be biased. Therefore, the second-part findings should be interpreted as only preliminary. With this in mind, the following summary was made:

TABLE 2.13. STATISTICS OF ENTERING HEADWAY DATA COLLECTED
(All Times Are In Seconds)

Veh. no.	Valid cases	Mean	Standard error	Standard dev.	Median	Mode	Variance	Max.	Min.
1	1899	3.802	.019	.845	3.70	3.5	.714	7.8	1.6
2	1252	2.555	.018	.640	2.50	2.2	.410	5.5	1.2
3	822	2.352	.021	.612	2.30	2.1	.375	5.0	1.1
4	526	2.214	.026	.587	2.10	1.9	.345	4.4	0.9
5	327	2.163	.035	.629	2.10	1.8	.395	5.0	0.9
6	191	2.026	.040	.550	1.90	1.7	.302	4.5	1.0
7	127	1.972	.047	.527	1.90	1.6	.277	3.5	1.0
8	78	1.938	.054	.475	1.85	1.5	.225	3.9	1.1
9	44	1.941	.086	.573	1.85	1.5	.328	3.5	1.2
10	24	1.783	.074	.363	1.75	1.6	.132	2.4	1.1
11	13	1.638	.109	.393	1.60	1.3	.154	2.7	1.2
12	7	1.757	.092	.244	1.70	1.6	.060	2.1	1.5

- Signal types have no significant influence on entering headways at signalized intersections.
- Time of day, signified by a.m. and p.m. peak hours, does not appear to have any influence on entering headways.
- The inside lane of an approach has slightly lower entering headways than does the outside lane (the difference is statistically significant).
- The entering headway at approaches with speed limits of 20 mph are significantly higher than those at approaches with higher speed limits (more than 30mph). For approaches with speed limits higher than 30 mph, the influence of speed limits on the entering headway is not noticeable.
- In general, streets that have higher speed limits and less roadside frictions have lower entering headway values.
- When queue length increases, the general observation is that the entering headway values decrease.

SHANTEAU [REF. 24]

Robert M. Shanteau, in 1988, used cumulative curves to measure saturation flow and lost time . In his research he made some field measurements.

A lane of an approach that was (almost) always saturated for a number of cycles (e.g., at least 5-10) was found. An approach was used that had fixed green and yellow intervals. For each phase, the elapsed time from the moment the light turned green until each vehicle entered the intersection was measured. An accuracy to the nearest second was acceptable (and all that could be obtained by hand). Table 2.14 shows entry times for an intersection in Concord, California.

The most recent study on departure headways was made by Massoum Moussavi and Mohammed Tarawneh in 1990 [Ref. 25].

Data Collection

The departure headways of approximately 10,000 vehicles from straight-through, exclusive left, and exclusive right-turn lanes at 22 intersections in six cities in the state of Nebraska were collected. A laptop microcomputer with 128K memory, one floppy disk drive, a built-in clock, and rechargeable battery was used to collect the field data. Three interactive programs were used to collect, retrieve, and analyze the data. The observer was pressing predefined keys from the keyboard at the passage of successive vehicles.

Statistical Analysis

The mean departure headway, standard deviation, variance, coefficient of variation, standard error of the mean, and the maximum and minimum values of departure headways were calculated for each lane group, each intersection, and each city in this study. The values of these parameters for the six cities are shown in Tables 2.15 through 2.20.

In general, the departure headways for different queue positions in large cities are smaller than those for small cities. The variance and standard deviation of the first queue position is higher than the the variance and standard deviation of the other queue positions in all cities. The variance and standard deviations of the departure headways for the fourth vehicle and above do not vary significantly from each other.

TABLE 2.14. TIME HEADWAYS FOR 15 VEHICLES (IN SECONDS)

Vehicle	Cycle Number									
Number	1	2	3	4	5	6	7	8	9	10
1	1	2	2	2	3	3	2	2	2	2
2	2	2	2	3	2	2	3	3	4	2
3	2	3	2	3	3	3	2	2	2	1
4	2	2	3	1	2	3	3	2	2	2
5	2	2	3	2	2	3	2	4	2	2
6	2	3	2	2	2	2	2	3	1	3
7	2	1	3	3	2	2	3	2	2	2
8	2	2	1	3	2	2	2	1	3	2
9	2	3	2	1	2	1	2	2	2	2
10	3	3	2	2	2	2	2	1	2	2
11	3	2	1	2	2	3	2	2	2	1
12	1	1	1	1	2	2	2	2	1	3
13	2	2	3	3	2	-	1	-	-	1
14	1	2	1	-	-	-	1	-	-	2
15	2	-	1	-	-	-	1	-	-	2

TABLE 2.15. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN FREMONT

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	3.10	0.64	0.41	20.77	0.086	5.00	1.43
2	1.87	0.036	0.0013	1.94	0.0048	2.75	0.99
3	2.00	0.41	0.17	20.70	0.058	3.08	1.42
4	2.12	0.60	0.36	28.47	0.10	3.90	1.04
5	2.10	0.52	0.27	24.56	0.098	3.13	1.31
6	1.95	0.34	0.11	17.29	0.084	2.31	1.43
7	1.73	0.38	0.15	22.04	0.16	2.20	1.09

TABLE 2.16. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN BELLEVUE

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	2.04	1.16	1.34	56.74	0.13	5.16	0.61
2	1.91	0.65	0.43	34.15	0.074	3.51	0.88
3	2.04	0.52	0.27	25.63	0.060	3.68	0.93
4	2.00	0.51	0.26	25.28	0.062	4.17	0.99
5	1.85	0.71	0.51	38.54	0.10	3.18	0.88
6	1.92	0.55	0.31	28.90	0.11	3.35	1.15
7	1.69	0.60	0.36	35.67	0.16	2.97	0.88

TABLE 2.17. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN LINCOLN

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	3.74	1.18	1.39	31.49	0.16	5.66	2.03
2	2.12	0.66	0.43	30.91	0.088	4.01	1.27
3	2.04	0.64	0.41	31.23	0.091	4.94	1.04
4	2.02	0.47	0.22	23.27	0.11	2.86	1.31
5	1.64	0.25	0.063	15.30	0.11	2.14	1.48

TABLE 2.18. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN COLUMBIA

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	3.21	0.84	0.71	26.26	0.14	5.33	1.04
2	2.12	0.78	0.60	36.65	0.13	4.89	0.72
3	2.23	0.58	0.34	26.22	0.10	3.46	1.16
4	2.04	0.44	0.19	21.38	0.11	2.86	1.42
5	1.87	0.59	0.35	31.63	0.17	2.80	0.77

TABLE 2.19. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN NORFOLK

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	2.91	0.90	0.81	30.91	0.15	5.44	1.54
2	1.92	0.81	0.66	42.27	0.14	3.41	0.88
3	2.21	0.44	0.19	19.81	0.076	3.90	1.21
4	2.13	0.60	0.37	28.39	0.13	3.90	1.48
5	1.84	0.37	0.14	20.11	0.13	2.69	1.54

TABLE 2.20. STATISTICAL ANALYSIS OF DEPARTURE HEADWAYS AT SIGNALIZED INTERSECTIONS IN OMAHA

Queue Position	Mean (sec)	Standard Deviation	Variance	Coefficient of Variation	Standard Error of Mean	Max. (sec)	Min. (sec)
1	2.41	0.94	0.88	38.72	0.037	6.65	0.66
2	2.28	0.60	0.36	26.23	0.023	6.53	0.97
3	2.09	0.56	0.31	26.64	0.022	4.80	1.09
4	1.96	0.51	0.26	26.02	0.020	6.77	0.72
5	1.90	0.47	0.22	24.96	0.018	5.80	0.99
6	1.87	0.46	0.21	24.82	0.018	4.47	0.20
7	1.82	0.40	0.16	22.52	0.017	4.78	0.98
8	1.79	0.50	0.25	28.36	0.023	5.53	0.99
9	1.72	0.37	0.14	21.50	0.019	3.40	0.93
10	1.67	0.47	0.22	27.80	0.026	6.48	0.93
11	1.66	0.49	0.24	28.80	0.031	5.10	0.98

Testing the Variability of Departure Headways

The Chi-Square test of goodness was used to test the distribution of the measured departure headways. The results of the test indicated that the departure headways for each queue position follows the normal distribution.

The results of the independent sample t-test indicated that the mean departure headways for different queue positions from different approaches in the same intersection are significantly different from each other. The results of the t-test also indicated that the mean departure headways for different queue positions for different cities are also different at 95 percent confidence level.

CONCLUSIONS

The results of this study indicated that there is a high degree of variability among the departure headways that were measured. The reason could be the differences in traffic and geometric conditions.

The departure headway for queue position number one ranges from 2.04 to 3.74 seconds in six Nebraska cities under this study. The departure headway for queue position number two ranges from 1.87 to 2.44 seconds. The departure headway for queue positions number three through seven ranges from 2.00 to 2.33 seconds, from 1.96 to 2.36 seconds, from 1.64 to 2.10 seconds, from 1.87 to 1.95 seconds, and from 1.69 to 1.82 seconds, respectively. The average departure headways that were obtained in this study are 2.90, 2.04, 2.10, 2.04, 1.87, 1.91, and 1.75 seconds for the first through the seventh vehicle in a stopped signalized intersection queue.

An overall review of the studies described above indicates that there was a high degree of variability in the observed departure headways. The major reasons for this are probably the following

a. different study objectives;
b. various methods of data collection were used;
c. different definitions for the reference line, the starting delay, and the vehicle headways were employed;
d. different study methods were employed;
e. the studies were conducted over a long period of time during which driver behavior and vehicle characteristics changed;
f. the sample size of the data differed from study to study and sometimes it was not reliably large; and
g. different locations in a wide variety of cities were studied.

CHAPTER 3. DATA COLLECTION

Field data describing stopped queue start-up and inter-vehicle headway times were collected at locations encompassing a broad range of conditions. The data collection activities are described in this chapter.

SITE SELECTION

All data collection sites were located in Austin, Texas. Five intersections along Congress Avenue were selected for study. Congress Avenue is a primary arterial street extending from the CBD to the suburbs (direction North-South), with a length of about 12 miles and commercial development along both roadsides. The intersection approaches and their associated characteristics examined in this study are summarized in Table 3.1.

The sites chosen had straight intersection approaches. The grades of the approaches, the intersections, and the exit lanes were of reasonably level profile. All intersections but one (Congress Avenue and Riverside Drive) were right-angle crossings. All intersections were signalized and controlled by pretimed, multi-phase signal systems. Parking was not permitted and did not exist for at least 600 feet ahead of any intersection.

FIELD OBSERVATION TECHNIQUE

All data were collected by Stilianos Efstathiadis. Data collection consisted primarily of measuring time intervals. For all selected intersection approaches, the time of individual vehicle movements was recorded by using a portable time recording device developed at The University of Texas at Austin. The observer pressed a button when the signal changed to green and when a vehicle passed the reference line. It is felt that the presence of the observer had little or no effect on the behavior of traffic in the intersections being measured.

Definitions

Headway time for the first car in line was defined as the time between onset of the green phase and the instant the front wheels of the car crossed the reference line. Headway times for each of the remaining cars in the queue were the intervals between crossings of the reference line by the front wheels of successive vehicles.

TABLE 3.1. INTERSECTION APPROACH CHARACTERISTICS

Intersection	Approach code	Direction	Lane position (from centerline)	Speed limit (mph)	Lane width (feet)	District area
Congress Av. & Riverside Dr.	1 S 3	Southbound	3	30	10.5	Business
Congress Av. & Riverside Dr.	1 S 4	Southbound	4	30	11.5	Business
Congress Av. & Riverside Dr.	1 N 3	Northbound	3	30	10.0	Business
Congress Av. & Riverside Dr.	1 N 4	Northbound	4	30	11.0	Business
Congress Av. & Oltorf St.	6 S 2	Southbound	2	35	10.0	Intermediate
Congress Av. & Oltorf St.	6 S 3	Southbound	3	35	10.0	Intermediate
Congress Av. & Oltorf St.	6 N 2	Northbound	2	35	9.5	Intermediate
Congress Av. & Oltorf St.	6 N 3	Northbound	3	35	9.5	Intermediate
Congress Av. & Ben White Blvd.	9 S 1	Southbound	1	40	12.0	Commercial
Congress Av. & Ben White Blvd.	9 S 2	Southbound	2	40	18.0	Commercial
Congress Av. & Ben White Blvd.	10 N 1	Northbound	1	40	13.0	Commercial
Congress Av. & Ben White Blvd.	10 N 2	Northbound	2	40	17.0	Commercial
Congress Av. & Stassney La.	11 S 2	Southbound	2	40	12.0	Commercial
Congress Av. & Stassney La.	11 N 2	Northbound	2	40	16.5	Commercial
Congress Av. & William Cannon Dr.	12 S 2	Southbound	2	45	18.0	Open
Congress Av. & William Cannon Dr.	12 N 2	Northbound	2	45	18.5	Open

SELECTION OF REFERENCE LINE

For a period of time, each investigated lane was observed in order to establish the area at which most vehicles were stopping. The reference line was selected as the location at which, most of the time, the front bumper of the first car in the queue stopped. This line was the designated reference line on the assumption that most vehicles, in the first position of any queue, would stop with their front wheels only a short distance behind this line. Most of the time, the reference line was the stop line of the investigated lane.

This line was selected as the screenline because it was felt that it would give the best and most consistent results for the departure headways. That is before any additional factors such as the driver's desired speed and behavior would add more variation to the observed headways.

The same reference line was used for each lane for all the cycles for which data were collected. This way all measurements were consistent for any one lane.

Eligible Vehicles

Data were collected for each vehicle in the queue and recorded as a function of the vehicle's position in the queue. When the original queue was destroyed by a lane change (a vehicle moved into or out of the lane of interest) normal queue departure movement was disrupted, therefore data collection was discontinued for that cycle.

Headways were measured for only those cars that were effectively stopped either during the red phase or during the green phase, because the wave of motion (starting wave) had not traveled back far enough.

Data were recorded for all cars on each cycle that proceeded straight through the intersection and were not impeded by pedestrians, cross traffic, or opposing left turners. Thus, the headways are based on lanes with uninterrupted through movements. Measurements were taken only if free flow of traffic downstream from the intersection was provided.

Vehicle Codes

The characteristics of each vehicle were classified; that is, a coded description of each vehicle was recorded when appropriate. Vehicle size for this study was classified into two major categories: (a) passenger cars (which also included mini vans and small trucks); and (b) trucks, busses, or vehicles with trailers. Because most of the vehicles were in the first category, passenger cars were established as the default value. The observer noted on the data sheet only when a vehicle from the other category was measured.

Observation Problems

A major problem was encountered with using the designated reference line because vehicles sometimes stopped past that line. In such a situation, the time for that vehicle was recorded but an interruption code (i.e. 1) was noted on the data sheet. When a vehicle stopped with its front wheels on the reference line, no interruption code was noted, and the time measured was its start-up reaction time. Also, more frequently for some intersections than for others, there were occasions when the first vehicle in the queue willingly delayed its departure because there was still opposing flow. In these cases, the time for that vehicle was recorded, but an interruption code (i.e. 2) was added.

Time Recording Device

All times for individual vehicles were obtained with a time recording device. The time recording device was in effect a storage stopwatch which had the following characteristics: (1) it was portable; (2) it used an independent DC power supply (a small motorcycle battery); (3) it incorporated solid state electronic component reliability; (4) it could store up to 32 time intervals; and (5) it had selectable time increments of 1.0 second, 0.1 second, and 0.01 second.

The user first placed the time increment switch to the desired basis for timing. The 0.01 second time increment was selected for this study. Next, the user set the record/display switch to the record function. Timing of the first event was initiated by pressing the zero button. Each time the event button was depressed thereafter, the duration of the current event was stored in a 32-position register stack and timing of a new event was begun. The current stack position was displayed for the user.

After storing the desired number of timed intervals, the user positioned the record/display switch to the display function and read the stored data. The device had a digital display which was easy to read. The entire stack could be examined in sequence by repeatedly depressing the event button. Displayed information was transferred to a data sheet.

Time of Measurement

The studies were conducted during both the morning and evening periods of peak flow on average weekdays. The morning and evening periods of peak flow were identified after traffic counts were conducted at each approach under consideration. The morning peak period was from 7:00 a.m. to 8:30 a.m. and the evening peak period from 4:30 p.m. to 5:45 p.m. The weather at all times of data collection was dry and sunny or cloudy, i.e. the pavement was dry.

Period of Data Collection

All data were collected from July 1991 to March 1992. In all, more than 8,000 vehicle headways were recorded.

ESTIMATION OF SAMPLE SIZE

The required size of the sample that should be collected had to be estimated first. The size of a sample of data needed to give a mean within some desired range of accuracy is dependent on the desired confidence level and the standard deviation of the population. Because no comparable data were available, the standard deviation of the population had to be assumed. A 'pilot' study was conducted and an estimate of the standard deviation of the population could be obtained from that sample, so it became possible to determine whether the size of the available sample was sufficiently large. The following equation was used to determine the required sample size.

$$n = \frac{K^2 * V^2}{D^2} \tag{3.1}$$

where n = required sample size

D = error that can be tolerated (as a percentage)

K = number of standard deviations from mean to produce desired confidence

V = coefficient of variation, that is s/m where s is the standard deviation and m is the mean of the population

The above equation stands true in the case that the population is approximately normally distributed and the resultant sample size is not very large compared to the total number of cases in the population.

As it can be seen from Equation 3.1, some variables are unknown before any sample is collected. For this reason, a small exploratory sample was obtained; the sample standard deviation and the sample average were used as approximations of population values. After some initial data were collected, it was found that the average coefficient of variation was approximately 0.28.

For this study the error that could be tolerated, D, was chosen not to exceed 10% or 0.10, and K was found from a Standard Normal Probability Table to be 1.645.

After replacing the above values at Equation 3.1, the required sample size was found to be around 22. It was determined that in order to detect a headway time difference of 10 percent, with 90 percent confidence, a sample size of at least 22 was required.

Interpreting this result, any data sample of size 22 observations or more would have a mean value within 10 percent from the true population mean in 90 cases out of 100. These sample sizes are strict minimums, and more observations were obtained where feasible.

The data were transferred from the data sheets to a computer file by entering at a remote terminal. The format used is shown in Table 3.2. Wherever irregularities in traffic flow were observed, headway fields were left blank and interpreted as missing. Missing values were recorded as dots.

TABLE 3.2. DATA RECORD FORMAT

Column	Variable name	Comments
1-2	INTERS	Intersection number (1-12)
3-4	DIR	Direction (North or South)
5-6	LANE	Lane position (1-4)
7-8	TIME	Time period (A.m. or P.m. peak period)
9-80	P1-P19	Headway time for positions 1-19

CHAPTER 4. DATA ANALYSIS AND RESULTS

INTRODUCTION

Two parameters of traffic performance can be used to represent several important characteristics of intersection operation. The two parameters investigated in this study are starting delay and time spacing between vehicles departing from a stopped position to enter a signalized intersection. Information on the variability of these parameters at an intersection and among intersections may have useful applications in studies of intersection capacity and signal timing.

When a traffic signal interrupts the flow of traffic, the vehicles stopped by the signal are delayed during the time the signal is red plus the time required for the vehicles to get started and underway again. This latter delay is commonly called starting delay. A generally accepted definition of starting delay is the time required for the first vehicles in a queue to commence motion and enter an intersection after the traffic signal displays a green indication. Time spacing is the average time headway in seconds between successive vehicles in an entering platoon.

ERROR ANALYSIS

The manual input method used for time increment measurements has an element of error because of the observer's reaction time.

The error at each time headway measured was estimated. In order to accomplish that, the observer measured his experimental error. The hypothesis was that zero seconds time headway existed. Thus, the time calculated at two consecutive presses of the event button would give us an estimate of the error.

This procedure was repeated 105 times and an average was calculated. The average error was found to be 0.155 seconds with standard deviation 0.014 seconds and standard error of the mean 0.0014 seconds.

As a result, all times measured in this study may contain an average measurement error of 0.155 seconds. Since this error was very small, the observer reaction time was assumed to be uniform for all inputs.

Upon examination of the data, some very low values for vehicle headways were discovered. Computations were performed, and some values were found to be impossible. In order to edit the data set, some assumptions were made. First it was assumed that the space interval (front-axle to front-axle) between vehicles would be at least 20 feet. Second, the maximum speed that the vehicles of each position in the queue could attain, when they reached the reference line, was as follows: 10 mph for the second vehicle in the queue, 15 mph for the third and fourth, 20 mph for the fifth, 25 mph for the sixth, 27 mph for the seventh, 30 mph for the eighth, 32 mph for the ninth, 34 mph for the tenth and 35 mph for the positions eleven through nineteen.

The following editing was done for the extremely small values. Whenever values smaller than the following were found, they were interpreted as missing for the following analysis. The minimum acceptable headway for the first vehicle in the queue was 1.00 sec. as the PIJR time (Perception, Identification, Judgment, and Reaction time) for an average driver in urban area [Ref. 26, 27]. The minimum headway values for positions two through nineteen were 1.37 sec, 0.91 sec, 0.91 sec, 0.68 sec, 0.54 sec, 0.51 sec, 0.45 sec, 0.42 sec, 0.40 sec, and 0.39 sec thereafter.

EXPLORATORY DATA ANALYSIS

Due to the limited number of trucks, busses and trailers (vehicles of the second category), the analysis was concentrated only on passenger cars and light trucks (vehicles of the first category). The headway times of any heavy vehicles were excluded from the analysis. It was assumed that these vehicles influence only their headways and not of their following cars. This was examined and it was found to be valid. The headways of cars following vehicles of the second category did not have any significant difference from the cars of the same position following vehicles of the first category. Thus, the following analysis is concentrated only on passenger cars and light trucks and the results are valid only for these vehicles.

Also the time headways for vehicles at the first position in the queue with interruption codes 1 or 2 (see Chapter 3) were excluded from the subsequent analysis.

The following analysis is the first step to relate the headway values and their variabilities to physical and traffic conditions.

For reasons discussed earlier, there is a marked tendency for the number of sample points to decrease as one goes from the front of the waiting line towards the rear. In much of the statistical analysis that follows, the shape of the density functions is important. From a practical point of view, by this it is meant that the first two moments be reasonably accurate. For the density functions that are used in this study, if the sample size is sufficiently large to ensure a given relative accuracy of the variance then the relative accuracy of the mean will be much greater.

A normality test from the SAS/STAT software [Ref. 28] was used to test the distribution of the departure headways. Departure headways were in most cases normally distributed.

Departure from normality, where it existed, was in the form of positive skewing with a long tail of high values.

A preliminary analytical procedure involved computing selected sample statistics. In the following Tables 4.1 through 4.16 the number of observations, median, mean, standard deviation, minimum, maximum, range, standard error of the mean, variance, and coefficient of variation of the vehicles' headways are given for each specific lane that was investigated. In Table 4.17 the descriptive statistics for all the collected data are summarized. In these tables and throughout this chapter, the approach codes from Table 3.1 are used for identification instead of the actual approach names.

The mean value of headway shown in this tabulation generally decreases from front to rear of the queue and there is some tendency to approach a constant value at position three or four, depending on the case.

An overall review of the studies in the literature review suggests that all efforts are fragmented in terms of study methods, location characteristics, and technical objectives. Therefore, comparison of these studies is limited to a general observation that their results do not entirely agree. It is also noticed that by far the most comprehensive study was that reported by Greenshields et al. conducted 45 years ago. It is questionable whether the results could represent current traffic characteristics. In addition, the limited depth of the studies does not allow a clear identification of factors that can affect the headway values.

It is interesting to note that all the entering headways from various studies appear to follow a similar pattern except for the first two vehicles. It is a fact that studies with the first vehicle having a low headway value have used a different definition of entering headway than the others. This study is the only one that defines the reference line as an imaginary line in front of the usual first vehicles stopping place. So, the departure headway for the first vehicle is closer to its PIJR time.

Further analysis suggests that there are logical reasons for the observation that the departure headways measured in this study were lower than in almost all other studies. This study was one of the very few that was conducted in the last 5 years. Cars are smaller than ever before and have better acceleration characteristics. The results are very similar to the last study in the literature review, by M. Moussavi and M. Tarawneh [Ref. 25], conducted in 1990, and with the studies of C.J. Ancker et al. [Ref. 13, 1967], R.M. Bartle et al. [Ref. 4, 1952] and B. Helm [Ref. 6,1957]. Surprisingly, there is one study conducted by D.S. Berry and P.K. Gandhi [Ref. 16, 1973], that has reported significantly lower headway values than this one.

It may be presumed that more aggressive driver behavior and the advent of automatic transmissions have contributed to more rapid response to the green signal by today's drivers.

TABLE 4.1. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND RIVERSIDE DRIVE, SOUTHBOUND, LANE NO. 3, P.M. PEAK PERIOD (APPROACH CODE: 1S3)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	33	1.96	2.01	0.45	1.31	2.86	1.55	0.08	0.20	22.25
2	52	2.42	2.49	0.53	1.39	4.21	2.82	0.07	0.28	21.22
3	54	2.17	2.17	0.41	1.38	3.25	1.87	0.06	0.17	18.84
4	54	1.85	1.98	0.51	1.23	4.38	3.15	0.07	0.26	25.56
5	50	1.80	1.84	0.48	0.92	3.15	2.23	0.07	0.23	25.88
6	44	1.94	2.02	0.35	1.38	2.77	1.39	0.05	0.12	17.44
7	43	1.80	1.85	0.47	0.84	3.00	2.16	0.07	0.22	25.48
8	40	1.72	1.72	0.31	0.99	2.37	1.38	0.05	0.10	18.22
9	37	1.73	1.75	0.31	1.26	2.95	1.69	0.05	0.10	17.92
10	35	1.58	1.66	0.34	1.23	2.50	1.27	0.06	0.11	20.38
11	32	1.53	1.55	0.26	1.04	2.16	1.12	0.05	0.07	17.03
12	30	1.58	1.60	0.28	1.01	2.09	1.08	0.05	0.08	17.41
13	26	1.59	1.62	0.22	1.18	2.14	0.96	0.04	0.05	13.74
14	23	1.55	1.55	0.26	0.91	2.23	1.32	0.05	0.07	16.58
15	12	1.61	1.61	0.10	1.47	1.78	0.31	0.03	0.01	6.17
16	12	1.56	1.55	0.17	1.21	1.83	0.62	0.05	0.03	10.85
17	6	1.72	1.56	0.23	1.32	1.94	0.62	0.09	0.05	14.46
18	4	1.59	1.64	0.24	1.41	1.97	0.56	0.12	0.06	14.68
19	3	1.71	1.70	0.19	1.51	1.89	0.38	0.11	0.04	11.16

TABLE 4.2. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND RIVERSIDE DRIVE, SOUTHBOUND, LANE NO. 4, P.M. PEAK PERIOD (APPROACH CODE: 1S4)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	30	2.15	2.23	0.66	1.17	3.71	2.54	0.12	0.43	29.48
2	60	2.62	2.69	0.66	1.56	4.31	2.75	0.08	0.43	24.36
3	60	2.22	2.21	0.57	1.23	3.82	2.59	0.07	0.32	25.71
4	60	1.92	2.04	0.54	1.14	3.81	2.67	0.07	0.29	26.61
5	59	2.04	2.00	0.39	1.32	2.83	1.51	0.05	0.15	19.43
6	58	1.89	1.97	0.68	1.08	5.57	4.49	0.09	0.46	34.24
7	56	1.78	1.89	0.58	1.06	3.64	2.58	0.08	0.34	30.83
8	49	1.79	1.92	0.63	1.15	4.69	3.54	0.09	0.39	32.53
9	47	1.78	1.88	0.54	0.96	4.11	3.15	0.08	0.29	28.86
10	43	1.63	1.71	0.45	0.99	3.32	2.33	0.07	0.21	26.53
11	43	1.77	1.72	0.49	0.54	2.78	2.24	0.08	0.24	28.76
12	41	1.77	1.82	0.56	1.03	3.87	2.84	0.09	0.32	30.98
13	40	1.75	1.90	0.62	0.85	3.35	2.50	0.10	0.38	32.58
14	34	1.57	1.65	0.49	0.50	2.62	2.12	0.08	0.24	29.75
15	27	2.06	2.13	0.84	0.87	4.29	3.42	0.16	0.71	39.54
16	20	1.86	1.84	0.63	0.89	3.67	2.78	0.14	0.40	34.38
17	13	1.60	1.60	0.47	1.01	2.52	1.51	0.13	0.22	29.27
18	8	1.95	2.06	0.65	1.17	3.33	2.16	0.23	0.43	31.78
19	5	1.85	1.81	0.35	1.25	2.23	0.98	0.16	0.12	19.55

TABLE 4.3. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND RIVERSIDE DRIVE, NORTHBOUND, LANE NO. 3, A.M. PEAK PERIOD (APPROACH CODE: 1N3)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	39	2.01	1.99	0.33	1.22	2.69	1.47	0.05	0.11	16.52
2	42	2.37	2.33	0.40	1.65	3.29	1.64	0.06	0.16	17.32
3	42	2.11	2.06	0.44	1.21	2.94	1.73	0.07	0.19	21.22
4	41	1.87	1.91	0.38	1.20	2.87	1.67	0.06	0.14	19.93
5	40	1.94	1.94	0.37	1.21	2.98	1.77	0.06	0.13	18.81
6	37	1.69	1.67	0.32	1.02	2.25	1.23	0.05	0.10	19.18
7	32	1.84	1.81	0.26	1.22	2.34	1.12	0.05	0.07	14.46
8	32	1.92	1.77	0.33	1.00	2.69	1.69	0.06	0.11	18.78
9	30	1.79	1.74	0.30	1.04	2.23	1.19	0.05	0.09	17.06
10	27	1.81	1.83	0.27	1.12	2.43	1.31	0.05	0.08	14.98
11	24	1.80	1.76	0.23	1.21	2.15	0.94	0.05	0.05	12.90
12	19	1.84	1.81	0.27	1.05	2.14	1.09	0.06	0.07	14.94
13	12	1.81	1.66	0.41	0.61	2.02	1.41	0.12	0.17	24.72
14	9	1.92	1.90	0.20	1.51	2.26	0.75	0.07	0.04	10.50
15	6	1.84	1.70	0.32	1.18	1.97	0.79	0.13	0.10	18.57
16	4	1.87	1.76	0.29	1.34	1.96	0.62	0.14	0.08	16.20

TABLE 4.4. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND RIVERSIDE DRIVE, NORTHBOUND, LANE NO. 4, A.M. PEAK PERIOD (APPROACH CODE: 1N4) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	45	2.03	2.03	0.41	1.08	3.04	1.96	0.06	0.17	20.10
2	48	2.60	2.32	0.36	1.55	3.02	1.47	0.05	0.13	15.49
3	49	2.04	2.10	0.37	1.32	3.01	1.69	0.05	0.13	17.46
4	46	1.89	1.92	0.39	0.91	2.74	1.83	0.06	0.16	20.48
5	43	1.88	1.87	0.35	1.12	2.66	1.54	0.05	0.12	18.52
6	40	1.91	1.87	0.36	1.09	2.67	1.58	0.06	0.13	19.39
7	40	1.90	1.88	0.27	1.29	2.43	1.14	0.04	0.07	14.43
8	40	1.80	1.77	0.33	1.12	2.41	1.29	0.05	0.11	18.53
9	37	1.87	1.86	0.36	1.17	2.90	1.73	0.06	0.13	19.33
10	35	1.82	1.80	0.26	1.12	2.51	1.39	0.04	0.07	14.49
11	28	1.71	1.71	0.18	1.21	2.01	0.80	0.03	0.03	10.35
12	24	1.75	1.73	0.34	1.04	2.90	1.86	0.07	0.11	19.53
13	16	1.72	1.68	0.21	1.11	1.95	0.84	0.05	0.04	12.43
14	7	1.57	1.60	0.37	1.21	2.25	1.04	0.14	0.14	23.03
15	6	1.72	1.82	0.50	1.34	2.60	1.26	0.20	0.25	27.35
16	6	1.42	1.51	0.58	0.99	2.58	1.59	0.23	0.33	38.06
17	4	1.34	1.70	0.83	1.18	2.94	1.76	0.42	0.69	48.95
18	4	1.36	1.29	0.34	0.86	1.59	0.73	0.17	0.11	26.02
19	2	1.41	1.41	0.49	1.06	1.76	0.70	0.35	0.25	35.10

TABLE 4.5. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND OLTORF STREET, SOUTHBOUND, LANE NO. 2, P.M. PEAK PERIOD (APPROACH CODE: 6S2) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	21	1.95	2.27	1.01	1.29	6.05	4.76	0.22	1.02	44.57
2	44	2.49	2.49	0.69	1.21	4.68	3.47	0.10	0.48	27.90
3	44	2.17	2.24	0.58	1.18	4.92	3.74	0.09	0.34	26.08
4	44	2.10	2.17	0.50	1.22	3.35	2.13	0.08	0.25	23.08
5	43	1.88	1.96	0.46	1.11	3.31	2.20	0.07	0.21	23.64
6	42	2.00	2.03	0.47	1.21	3.09	1.88	0.07	0.22	23.17
7	40	1.92	2.03	0.71	0.93	3.95	3.02	0.11	0.50	34.83
8	39	1.81	1.82	0.44	1.04	2.97	1.93	0.07	0.20	24.30
9	36	1.82	1.81	0.51	0.65	3.23	2.58	0.08	0.26	27.98
10	37	1.85	1.90	0.60	0.81	3.25	2.44	0.10	0.36	31.72
11	36	1.74	1.78	0.41	0.98	3.01	2.03	0.07	0.17	22.97
12	36	1.69	1.81	0.68	0.91	3.86	2.95	0.11	0.46	37.52
13	29	1.67	1.81	0.68	0.72	3.53	2.81	0.13	0.46	37.59
14	22	1.58	1.60	0.59	0.72	2.79	2.07	0.13	0.34	36.68
15	12	1.67	1.51	0.40	1.02	2.57	1.55	0.11	0.16	26.20
16	2	1.76	1.76	1.00	1.05	2.46	1.41	0.71	0.99	56.81
17	2	1.06	1.06	0.08	1.00	1.11	0.11	0.05	0.01	7.37

TABLE 4.6. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND OLTORF STREET, SOUTHBOUND, LANE NO. 3, P.M. PEAK PERIOD (APPROACH CODE: 6S3)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	33	1.86	1.89	0.29	1.37	2.64	1.27	0.05	0.09	15.49
2	39	2.11	2.32	0.71	1.47	4.06	2.59	0.11	0.50	30.39
3	41	1.86	1.88	0.49	0.96	3.11	2.15	0.08	0.24	26.08
4	43	1.89	1.98	0.57	0.91	3.43	2.52	0.09	0.32	28.70
5	43	1.89	2.04	0.61	0.81	3.61	2.80	0.09	0.37	30.01
6	44	1.87	1.94	0.60	0.82	3.84	3.02	0.09	0.36	30.91
7	44	1.92	1.92	0.41	1.01	3.01	2.00	0.06	0.16	21.15
8	42	1.97	1.93	0.49	0.48	2.84	2.36	0.08	0.24	25.39
9	43	1.90	1.84	0.38	1.00	2.66	1.66	0.06	0.14	20.52
10	42	2.00	2.02	0.51	0.88	3.10	2.22	0.08	0.26	25.23
11	37	2.02	1.89	0.50	0.42	2.74	2.32	0.08	0.25	26.21
12	21	1.93	2.01	0.49	1.54	3.83	2.29	0.11	0.24	24.22
13	11	1.89	1.90	0.32	1.38	2.54	1.16	0.10	0.10	16.85
14	7	1.56	1.67	0.33	1.21	2.11	0.90	0.13	0.11	19.94
15	4	1.51	1.56	0.37	1.17	2.06	0.89	0.19	0.14	23.86

TABLE 4.7. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND OLTORF STREET, NORTHBOUND, LANE NO. 2, A.M. PEAK PERIOD (APPROACH CODE: 6N2)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	43	2.00	1.98	0.28	1.29	2.56	1.27	0.04	0.08	14.20
2	43	2.42	2.41	0.37	1.72	3.27	1.55	0.06	0.14	15.35
3	40	2.16	2.15	0.33	1.34	2.75	1.41	0.05	0.11	15.16
4	37	2.02	2.03	0.36	1.28	2.66	1.38	0.06	0.13	17.62
5	36	1.93	1.91	0.28	1.38	2.62	1.24	0.05	0.08	14.78
6	35	1.84	1.86	0.26	1.21	2.51	1.30	0.04	0.07	13.93
7	31	1.79	1.72	0.32	1.00	2.21	1.21	0.06	0.10	18.79
8	30	1.84	1.83	0.24	1.14	2.29	1.15	0.04	0.06	13.06
9	27	1.85	1.82	0.17	1.28	2.13	0.85	0.03	0.03	9.10
10	24	1.79	1.79	0.16	1.34	2.11	0.77	0.03	0.03	8.96
11	20	1.78	1.68	0.34	0.78	2.08	1.30	0.08	0.11	19.92
12	11	1.78	1.68	0.21	1.34	1.91	0.57	0.06	0.04	12.53
13	8	1.76	1.66	0.26	1.20	1.92	0.72	0.09	0.07	15.94
14	3	1.78	1.80	0.05	1.77	1.86	0.09	0.03	0.00	2.74
15	1	1.78	1.78	-	1.78	1.78	0.00	-	-	-
16	1	1.77	1.77	-	1.77	1.77	0.00	-	-	-

TABLE 4.8. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVERAGE AND OLTORF STREET, NORTHBOUND, LANE NO. 3, A.M. PEAK PERIOD (APPROACH CODE: 6N3)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	36	1.99	1.95	0.44	1.01	3.00	1.99	0.07	0.20	22.68
2	39	2.51	2.48	0.39	1.64	3.51	1.87	0.06	0.15	15.71
3	38	2.12	2.13	0.32	1.37	2.81	1.44	0.05	0.10	14.83
4	34	1.97	1.95	0.36	1.31	2.74	1.43	0.06	0.13	18.30
5	33	1.93	1.94	0.36	1.11	3.12	2.01	0.06	0.13	18.46
6	32	1.84	1.87	0.33	1.41	2.86	1.45	0.06	0.11	17.55
7	31	1.83	1.82	0.26	1.19	2.38	1.19	0.05	0.07	14.40
8	29	1.82	1.85	0.26	1.37	2.71	1.34	0.05	0.07	13.98
9	27	1.82	1.82	0.35	1.09	3.19	2.10	0.07	0.12	19.15
10	18	1.82	1.83	0.23	1.41	2.34	0.93	0.05	0.05	12.50
11	11	1.89	1.90	0.28	1.34	2.31	0.97	0.08	0.08	14.54
12	7	1.72	1.74	0.17	1.49	2.01	0.52	0.06	0.03	9.63
13	5	1.92	1.89	0.12	1.71	2.03	0.32	0.05	0.01	6.16
14	4	1.87	1.89	0.26	1.66	2.16	0.50	0.13	0.07	13.52
15	1	1.78	1.78	-	1.78	1.78	0.00	-	-	-
16	1	1.39	1.39	-	1.39	1.39	0.00	-	-	-

TABLE 4.9. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND BEN WHITE BOULEVARD, SOUTHBOUND, LANE NO. 1, P.M. PEAK PERIOD (APPROACH CODE: 9S1) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	36	2.02	2.07	0.52	1.32	4.60	3.28	0.09	0.27	25.21
2	38	2.28	2.31	0.34	1.69	3.23	1.54	0.06	0.12	14.89
3	38	2.07	2.04	0.30	1.31	2.67	1.36	0.05	0.09	14.57
4	38	1.95	1.94	0.30	1.28	2.67	1.39	0.05	0.09	15.35
5	38	1.87	1.89	0.24	1.43	2.48	1.05	0.04	0.06	12.49
6	38	1.92	1.97	0.34	1.43	3.31	1.88	0.05	0.11	17.03
7	26	1.88	1.88	0.25	1.31	2.39	1.08	0.05	0.06	13.04
8	19	1.86	1.80	0.28	1.20	2.18	0.98	0.06	0.08	15.42
9	16	1.89	1.86	0.24	1.21	2.32	1.11	0.06	0.06	12.69
10	12	1.96	1.82	0.30	1.23	2.29	1.06	0.09	0.09	16.37
11	8	1.83	1.84	0.09	1.74	1.96	0.22	0.03	0.01	5.06
12	3	1.81	1.85	0.10	1.78	1.97	0.19	0.06	0.01	5.51
13	1	1.85	1.85	-	1.85	1.85	0.00	-	-	-

TABLE 4.10. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND BEN WHITE BOULEVARD, SOUTHBOUND, LANE NO. 2, P.M. PEAK PERIOD (APPROACH CODE: 9S2) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	40		2.23	0.43	1.35	3.10	1.75	0.07	0.18	19.30
2	42		2.47	0.54	1.54	3.78	2.24	0.08	0.29	21.78
3	43		2.06	0.32	1.23	2.76	1.53	0.05	0.10	15.68
4	43		2.04	0.33	1.25	2.73	1.48	0.05	0.11	16.30
5	43		1.88	0.40	0.70	2.71	2.01	0.06	0.16	21.21
6	42		1.95	0.29	1.42	2.71	1.29	0.05	0.09	14.93
7	40		1.77	0.26	1.23	2.29	1.06	0.04	0.07	14.55
8	38		1.75	0.25	0.99	2.37	1.38	0.04	0.06	14.01
9	28		1.90	0.31	1.36	2.95	1.59	0.06	0.10	16.23
10	17		1.81	0.24	1.37	2.25	0.88	0.06	0.06	13.46
11	10		1.78	0.21	1.37	2.03	0.66	0.07	0.04	11.60
12	6		1.77	0.12	1.56	1.89	0.33	0.05	0.01	6.61
13	3		1.53	0.42	1.25	2.01	0.76	0.24	0.18	27.51
14	3		1.79	0.51	1.29	2.30	1.01	0.29	0.26	28.22
15	2		1.45	0.42	1.16	1.75	0.59	0.30	0.17	28.67
16	1		1.37	-	1.37	1.37	0.00	-	-	-

TABLE 4.11. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND BEN WHITE BOULEVARD, NORTHBOUND, LANE NO. 1, A.M. PEAK PERIOD (APPROACH CODE: 10N1)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	35	2.13	2.09	0.43	1.17	3.01	1.84	0.07	0.19	20.62
2	36	2.44	2.50	0.41	1.91	3.46	1.55	0.07	0.17	16.59
3	36	2.11	2.10	0.41	1.27	3.10	1.83	0.07	0.17	19.55
4	36	1.94	2.03	0.48	1.10	3.19	2.09	0.08	0.23	23.67
5	36	1.82	1.89	0.33	1.31	2.87	1.56	0.06	0.11	17.63
6	34	1.83	1.90	0.45	1.05	3.19	2.14	0.08	0.21	23.95
7	35	1.84	1.88	0.43	0.90	3.31	2.41	0.07	0.18	22.62
8	34	1.78	1.79	0.33	1.11	2.82	1.71	0.06	0.11	18.27
9	33	1.81	1.73	0.27	1.00	2.21	1.21	0.05	0.08	15.89
10	32	1.75	1.70	0.29	1.07	2.37	1.30	0.05	0.08	16.94
11	30	1.82	1.76	0.22	1.18	2.21	1.03	0.04	0.05	12.53
12	29	1.78	1.72	0.22	1.29	2.09	0.80	0.04	0.05	12.55
13	26	1.77	1.76	0.25	1.26	2.46	1.20	0.05	0.06	13.97
14	22	1.77	1.69	0.27	1.04	1.95	0.91	0.06	0.07	15.80
15	15	1.78	1.75	0.27	1.28	2.25	0.97	0.07	0.07	15.38
16	10	1.81	1.75	0.30	1.05	2.03	0.98	0.09	0.09	16.97
17	4	1.81	1.79	0.43	1.23	2.29	1.06	0.22	0.19	24.33
18	1	1.74	1.74	-	1.74	1.74	0.00	-	-	-

TABLE 4.12. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND BEN WHITE BOULEVARD, NORTHBOUND, LANE NO. 2, A.M. PEAK PERIOD (APPROACH CODE: 10N2) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	38	2.13	2.12	0.35	1.40	3.08	1.68	0.06	0.12	16.65
2	40	2.55	2.59	0.51	1.83	4.27	2.44	0.08	0.26	19.68
3	40	2.13	2.17	0.38	1.49	3.02	1.53	0.06	0.14	17.52
4	40	2.02	2.04	0.33	1.46	3.03	1.57	0.05	0.11	16.39
5	40	1.88	1.88	0.41	1.08	3.66	2.58	0.06	0.17	21.79
6	37	1.84	1.83	0.35	1.22	3.10	1.88	0.06	0.12	19.10
7	38	1.80	1.78	0.32	1.08	2.59	1.51	0.05	0.11	18.21
8	36	1.79	1.78	0.32	1.28	2.56	1.28	0.05	0.10	17.96
9	35	1.85	1.76	0.28	1.05	2.13	1.08	0.05	0.08	15.60
10	32	1.84	1.84	0.22	1.18	2.25	1.07	0.04	0.05	11.74
11	26	1.86	1.76	0.22	1.27	2.03	0.76	0.04	0.05	12.54
12	19	1.83	1.79	0.39	1.04	2.97	1.93	0.09	0.15	21.61
13	15	1.79	1.77	0.15	1.53	2.03	0.50	0.04	0.02	8.29
14	8	1.73	1.66	0.32	0.95	2.02	1.07	0.11	0.10	19.46
15	8	1.81	1.83	0.13	1.64	2.01	0.37	0.05	0.02	7.00
16	5	1.81	1.81	0.06	1.73	1.88	0.15	0.03	0.00	3.10
17	4	1.61	1.75	0.22	1.42	1.91	0.49	0.11	0.05	12.82
18	2	1.93	1.93	0.11	1.86	2.01	0.15	0.07	0.01	5.48

TABLE 4.13. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND STASSNEY LANE, SOUTHBOUND, LANE NO. 2, P.M. PEAK PERIOD (APPROACH CODE: 11S2)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	56	2.07	2.07	0.47	1.11	3.23	2.12	0.06	0.22	22.53
2	67	2.29	2.41	0.51	1.59	3.64	2.05	0.06	0.26	21.23
3	71	2.10	2.19	0.48	1.42	3.50	2.08	0.06	0.23	21.66
4	71	1.95	2.05	0.48	1.14	4.00	2.86	0.06	0.23	23.19
5	70	1.96	2.00	0.51	0.89	3.52	2.63	0.06	0.26	25.70
6	69	1.83	1.91	0.53	1.16	3.29	2.13	0.06	0.28	27.82
7	64	1.85	1.91	0.43	1.09	3.37	2.28	0.05	0.19	22.70
8	50	1.81	1.87	0.56	1.00	4.12	3.12	0.08	0.32	29.97
9	41	1.62	1.69	0.44	0.90	3.02	2.12	0.07	0.19	25.95
10	28	1.67	1.68	0.36	0.95	2.72	1.77	0.07	0.13	21.66
11	18	1.65	1.75	0.50	1.15	2.95	1.80	0.12	0.25	28.57
12	10	1.75	1.83	0.47	1.02	2.55	1.53	0.15	0.22	25.44
13	4	1.79	1.73	0.30	1.33	2.01	0.68	0.15	0.09	17.08
14	3	1.55	1.40	0.30	1.06	1.60	0.54	0.17	0.09	21.26

TABLE 4.14. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND STASSNEY LANE, NORTHBOUND, LANE NO. 2, A.M. PEAK PERIOD (APPROACH CODE: 11N2) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	57		1.89	0.49	1.20	3.17	1.97	0.06	0.24	25.84
2	60		2.51	0.42	1.58	3.43	1.85	0.05	0.17	16.63
3	59		2.05	0.42	1.22	3.02	1.80	0.06	0.18	20.69
4	56		1.99	0.45	1.21	3.01	1.80	0.06	0.20	22.50
5	52		1.87	0.45	1.03	3.04	2.01	0.06	0.20	23.86
6	48		1.79	0.34	0.97	2.88	1.91	0.05	0.12	19.16
7	42		1.83	0.35	1.08	2.83	1.75	0.05	0.12	18.92
8	35		1.75	0.32	1.19	2.60	1.41	0.05	0.10	18.26
9	29		1.67	0.40	0.95	2.86	1.91	0.07	0.16	23.90
10	20		1.64	0.37	0.97	2.21	1.24	0.08	0.14	22.52
11	16		1.71	0.40	1.12	2.63	1.51	0.10	0.16	23.50
12	11		1.64	0.39	0.95	2.04	1.09	0.12	0.15	23.53
13	9		1.42	0.35	0.85	2.01	1.16	0.12	0.12	24.48
14	4		1.27	0.30	0.93	1.65	0.72	0.15	0.09	23.39

TABLE 4.15. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND WILLIAM CANNON, SOUTHBOUND, LANE NO. 2, P.M. PEAK PERIOD (APPROACH CODE: 12S2)
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	37	1.86	1.84	0.36	1.17	2.81	1.64	0.06	0.13	19.51
2	40	2.18	2.38	0.50	1.63	3.91	2.28	0.08	0.25	21.11
3	38	1.97	2.02	0.34	1.36	3.32	1.96	0.05	0.11	16.75
4	35	1.91	1.98	0.40	1.22	3.28	2.06	0.07	0.16	20.20
5	33	1.92	1.97	0.30	1.37	2.70	1.33	0.05	0.09	15.07
6	25	1.86	1.95	0.45	1.20	3.43	2.23	0.09	0.20	22.82
7	16	1.78	1.78	0.27	1.35	2.35	1.00	0.07	0.07	15.13
8	12	1.71	1.82	0.30	1.47	2.25	0.78	0.09	0.09	16.76
9	5	1.69	1.67	0.39	1.10	2.04	0.94	0.17	0.15	23.17
10	3	1.70	1.63	0.39	1.21	1.97	0.76	0.22	0.15	23.68
11	1	1.65	1.65		1.65	1.65	0.00			

TABLE 4.16. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR CONGRESS AVENUE AND WILLIAM CANNON, NORTHBOUND, LANE NO. 2, A.M. PEAK PERIOD (APPROACH CODE: 12N2) [ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	47	1.84	1.79	0.41	1.00	2.57	1.57	0.06	0.17	23.10
2	47	2.53	2.41	0.39	1.57	3.12	1.55	0.06	0.15	16.33
3	47	2.10	2.10	0.30	1.22	2.79	1.57	0.04	0.09	14.20
4	45	1.95	1.93	0.35	1.20	3.00	1.80	0.05	0.12	18.16
5	41	1.87	1.89	0.38	1.13	2.61	1.48	0.06	0.15	20.27
6	37	1.77	1.85	0.46	1.00	3.28	2.28	0.08	0.21	24.96
7	31	1.67	1.64	0.41	1.00	2.61	1.61	0.07	0.17	24.78
8	23	1.73	1.76	0.27	1.31	2.56	1.25	0.06	0.08	15.61
9	23	1.81	1.72	0.35	1.05	2.23	1.18	0.07	0.12	20.43
10	21	1.84	1.71	0.36	0.99	2.16	1.17	0.08	0.13	20.77
11	20	1.70	1.76	0.41	1.15	2.41	1.26	0.09	0.16	23.09
12	14	1.74	1.78	0.61	1.08	3.47	2.39	0.16	0.37	34.12
13	11	1.66	1.58	0.28	1.18	1.97	0.79	0.08	0.08	17.60
14	9	1.70	1.74	0.29	1.24	2.11	0.87	0.10	0.08	16.37
15	8	1.66	1.64	0.23	1.25	2.01	0.76	0.08	0.05	14.19
16	3	1.43	1.40	0.37	1.02	1.75	0.73	0.21	0.13	26.14

TABLE 4.17. DESCRIPTIVE STATISTICS OF INTER-VEHICLE QUEUE DEPARTURE TIME HEADWAYS FOR ALL COLLECTED DATA
[ALL TIME MEASUREMENTS ARE IN SECONDS]

Position	No. Obs.	Median	Mean	Std Dev	Min.	Max.	Range	Std Error	Variance	CV (%)
1	663	2.01	2.04	0.51	1.00	6.05	5.05	0.02	0.26	24.82
2	778	2.41	2.46	0.51	1.39	4.68	3.29	0.02	0.26	20.86
3	777	2.09	2.12	0.43	0.96	4.92	3.96	0.02	0.18	20.19
4	750	1.95	2.00	0.44	0.91	4.38	3.47	0.02	0.19	21.96
5	718	1.89	1.93	0.42	0.70	3.66	2.96	0.02	0.17	21.66
6	674	1.86	1.90	1.44	0.82	5.57	4.75	0.02	0.20	23.36
7	621	1.83	1.85	0.42	0.84	3.95	3.11	0.02	0.18	22.69
8	560	1.81	1.82	1.40	0.48	4.69	4.21	0.02	0.16	21.92
9	505	1.82	1.79	0.37	0.65	4.11	3.46	0.02	0.14	20.94
10	436	1.79	1.79	0.38	0.81	3.32	2.51	0.02	0.14	21.28
11	370	1.79	1.75	0.38	0.42	3.01	2.59	0.02	0.14	21.39
12	288	1.78	1.77	0.45	0.79	3.87	3.08	0.03	0.20	25.31
13	221	1.75	1.75	0.43	0.61	3.53	2.92	0.03	0.19	24.90
14	163	1.66	1.65	0.39	0.50	2.79	2.29	0.03	0.16	23.84
15	104	1.72	1.80	0.54	0.87	4.29	3.42	0.05	0.29	29.78
16	66	1.72	1.70	0.45	0.89	3.67	2.78	0.06	0.21	26.68
17	33	1.57	1.61	0.45	1.00	2.94	1.94	0.08	0.20	28.05
18	19	1.74	1.78	0.54	0.86	3.33	2.47	0.12	0.29	30.25
19	10	1.78	1.70	0.34	1.06	2.23	1.17	0.11	0.12	20.06

THE PAIRWISE INDEPENDENCE OF SUCCESSIVE HEADWAYS

One of the objectives of the study was to determine the position in the queue after which the headways approach a constant value. Tests for independence between the interarrival time samples for the vehicles in positions one through nineteen were performed. The significance probability, i.e., P[|test statistic| ≥ |observed value of the statistic|], will be reported throughout. The significance probability tells at what significance levels the hypothesis would be rejected. The 0.05 level was set as the significance level.

As explained earlier, data points are occasionally missing in the raw data. All of the tests to be discussed below are pairwise comparisons for independence, i.e., positions 1 with 2, 2 with 3, and so on. Whenever a gap occurs in the data, there are of course two missing pairs [e.g., if the nth data point is missing, the pairs (n-1, n) and (n, n+1) do not exist]. This gap poses no practical difficulties, but simply reduces the sample size for analysis.

As shown in Table 3.1, the data were collected from sixteen different lanes. At first it was decided that each lane would be considered as an independent individual case and for a better understanding of the data set a separate analysis for all the data together would be made.

The SAS/STAT software [Ref. 28] was used for the analyses. A procedure was used that was considered to be the most appropriate for unbalanced situations; that is, models where there are unequal numbers of observations for the different combinations of the treatment variables. The first step was to perform analysis of variance (ANOVA) for unbalanced data design. The hypothesis tested was that all mean headways for each queue position were equal, considering each lane a different situation. The analysis for each lane separately and all lanes together is presented in Table 4.18. The hypothesis of equal means can be rejected at the 0.001 level of significance, and the effect of position within the queue on mean headways between pairs of vehicles is thus found to be significant.

The next step was to determine which position pairs were significantly different. In order to accomplish this the least significant difference test (LSD) was conducted. The comparisons were made at the 0.05 significance level. The results are shown in Tables 4.19 through 4.23.

The general observation is that the start-up lost time of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first 2 or 3 vehicles in line, depending on the case. This result is within the range of values reported in the literature.

TABLE 4.18. ANALYSIS OF VARIANCE BETWEEN POSITIONS IN THE QUEUE

Lane	Source	DF	SS	Mean Square	F value	Pr > F*
	Model	18	42.441659	2.357870	15.26	0.0001
1S3	Error	571	88.205968	0.154476		
	Corrected Total	589	130.647626			
	Model	18	50.882200	2.826789	8.41	0.0001
1S4	Error	734	246.743881	0.336163		
	Corrected Total	752	297.626080			
	Model	15	14.429288	0.961953	8.28	0.0001
1N3	Error	420	48.789956	0.116167		
	Corrected Total	435	63.219244			
	Model	18	18.653643	1.036314	8.44	0.0001
1N4	Error	501	61.519352	0.122793		
	Corrected Total	519	80.172995			
	Model	16	32.154110	2.009632	5.80	0.0001
6S2	Error	511	176.999097	0.346378		
	Corrected Total	527	209.153206			
	Model	14	8.190242	0.585017	2.24	0.0061
6S3	Error	479	125.321800	0.261632		
	Corrected Total	493	133.512042			
	Model	15	17.349593	1.156640	13.48	0.0001
6N2	Error	374	32.087099	0.085794		
	Corrected Total	389	49.436692			
	Model	15	14.615447	0.974363	8.66	0.0001
6N3	Error	330	37.145386	0.112562		
	Corrected Total	345	51.760833			
	Model	12	6.556041	0.546337	5.18	0.0001
9S1	Error	298	31.457892	0.105563		
	Corrected Total	310	38.013932			
	Model	15	20.031293	1.335420	10.70	0.0001
9S2	Error	385	48.039079	0.124777		
	Corrected Total	400	68.070373			
	Model	17	22.295954	1.311527	10.22	0.0001
10N1	Error	466	59.799079	0.128324		
	Corrected Total	483	82.095034			
	Model	17	27.140782	1.596517	13.57	0.0001
10N2	Error	445	52.371411	0.117689		
	Corrected Total	462	79.512193			
	Model	13	25.840802	1.987754	8.42	0.0001
11S2	Error	608	143.582096	0.236155		
	Corrected Total	621	169.422899			
	Model	13	33.707210	2.592862	15.43	0.0001
11N2	Error	484	81.357353	0.168094		
	Corrected Total	497	115.064562			
	Model	10	9.177496	0.917750	6.18	0.0001
12S2	Error	234	34.734835	0.148440		
	Corrected Total	244	43.912331			
	Model	15	22.013781	1.467585	10.07	0.0001
12N2	Error	411	59.887464	0.145712		
	Corrected Total	426	81.901245			
	Model	18	333.916769	18.550932	96.77	0.0001
ALL	Error	7737	1483.194744	0.191702		
	Corrected Total	7755	1817.111513			

* The minimum value that the statistical software used can give is 0.0001

TABLE 4.19. PAIRWISE TESTS FOR CONGRESS AVENUE AND RIVERSIDE DRIVE INTERSECTION

	1 S 3		1 S 4		1 N 3		1 N 4	
Pairs	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.
(1,2)	.0001	0.02%	.0004	0.05%	.0001	0.02%	.0001	0.02%
(2,3)	.0001	0.02%	.0001	0.02%	.0003	0.04%	.0020	0.3%
(3,4)	.0128	1.3%	.1036	N.S.	.0460	4.7%	.0147	1.5%
(4,5)	.0711	N.S.*	.7434	N.S.	.6397	N.S.	.4649	N.S.
(5,6)	.0335	3.4%	.7898	N.S.	.0004	0.05%	.9736	N.S.
(6,7)	.0492	5%	.4214	N.S	.0911	N.S.	.8558	N.S
(7,8)	.1251	N.S.	.7380	N.S.	.6601	N.S.	.1428	N.S.
(8,9)	.7708	N.S.	.6844	N.S.	.7505	N.S	.2280	N.S.
(9,10)	.3717	N.S.	.1833	N.S	.3237	N.S.	.4588	N.S.
(10,11)	.2329	N.S.	.9540	N.S.	.4487	N.S.	.2840	N.S.
(11,12)	.6251	N.S.	.4131	N.S.	.6678	N.S.	.8356	N.S.
(12,13)	.8166	N.S.	.5551	N.S	.2521	N.S.	.7086	N.S
(13,14)	.5372	N.S.	.0618	N.S.	.1070	N.S	.6017	N.S.
(14,15)	.6751	N.S.	.0011	0.2%	.2519	N.S.	.2591	N.S.
(15,16)	.7319	N.S.	.0873	N.S.	.7794	N.S.	.1261	N.S.
(16,17)	.9594	N.S.	.2417	N.S.			.4055	N.S
(17,18)	.7676	N.S.	.0779	N.S			.1028	N.S.
(18,19)	.8330	N.S.	.4461	N.S.			.7049	N.S.

* N.S. = Non-Significant

TABLE 4.20. PAIRWISE TESTS FOR CONGRESS AVENUE AND OLTORF STREET INTERSECTION

	6 S 2		6 S 3		6 N 2		6 N 3	
Pairs	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.
(1,2)	.1098	N.S.*	.0004	0.05%	.0001	0.02%	.0001	0.02%
(2,3)	.0268	2.7%	.0001	0.02%	.0001	0.02%	.0001	0.02%
(3,4)	.5884	N.S.	.3980	N.S.	.0860	N.S.	.0210	2.2%
(4,5)	.0896	N.S.	.5882	N.S.	.0777	N.S.	.9061	N.S.
(5,6)	.5690	N.S.	.3724	N.S.	.4934	N.S.	.4096	N.S.
(6,7)	.9774	N.S.	.8350	N.S	.0505	N.S.	.5250	N.S
(7,8)	.1079	N.S.	.9165	N.S.	.1648	N.S.	.7364	N.S.
(8,9)	.9523	N.S.	.4250	N.S.	.9468	N.S	.7808	N.S.
(9,10)	.5128	N.S.	.1125	N.S	.6869	N.S.	.9163	N.S.
(10,11)	.3723	N.S.	.2781	N.S.	.2447	N.S.	.6188	N.S.
(11,12)	.7901	N.S.	.3901	N.S.	.9572	N.S.	.3200	N.S.
(12,13)	.9729	N.S.	.5617	N.S	.8740	N.S.	.4210	N.S
(13,14)	.2056	N.S.	.3520	N.S.	.4625	N.S	.9770	N.S.
(14,15)	.6820	N.S.	.7342	N.S.	.9450	N.S.	.7746	N.S.
(15,16)	.5898	N.S.			.9808	N.S.	.4117	N.S.
(16,17)	.2348	N.S.						
(17,18)								
(18,19)								

* N.S. = Non-Significant

TABLE 4.21. PAIRWISE TESTS FOR CONGRESS AVENUE AND BEN WHITE BOULEVARD INTERSECTION

	9 S 1		9 S 2		10 N 1		10 N 2	
Pairs	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.
(1,2)	.0012	0.2%	.0026	0.27%	.0001	0.02%	.0001	0.02%
(2,3)	.0003	0.04%	.0001	0.02%	.0001	0.02%	.0001	0.02%
(3,4)	.1985	N.S.*	.8001	N.S.	.4340	N.S.	.1025	N.S.
(4,5)	.4676	N.S	.0340	3.5%	.0915	N.S	.0358	3.6%
(5,6)	.2920	N.S.	.3210	N.S.	.9210	N.S.	.5209	N.S.
(6,7)	.3030	N.S	.0170	1.8%	.8431	N.S.	.5547	N.S
(7,8)	.4141	N.S.	.8321	N.S.	.3201	N.S.	.9822	N.S.
(8,9)	.5765	N.S.	.0827	N.S	.4395	N.S.	.8227	N.S.
(9,10)	.7499	N.S.	.3925	N.S.	.7860	N.S.	.3883	N.S.
(10,11)	.8950	N.S.	.8447	N.S.	.5229	N.S.	.4281	N.S.
(11,12)	.9653	N.S.	.9360	N.S.	.6659	N.S	.8294	N.S.
(12,13)	.9929	N.S	.3339	N.S.	.6497	N.S.	.8719	N.S
(13,14)			.3618	N.S.	.4595	N.S.	.4677	N.S.
(14,15)			.2995	N.S	.5835	N.S.	.3047	N.S.
(15,16)			.8443	N.S.	.9782	N.S.	.9034	N.S.
(16,17)					.8596	N.S.	.7861	N.S
(17,18)					.9056	N.S	.5283	N.S.
(18,19)								

* N.S. = Non-Significant

TABLE 4.22. PAIRWISE TESTS FOR CONGRESS AVENUE AND STASSNEY LANE, WILLIAM CANNON DRIVE INTERSECTIONS

	11 S 2		11 N 2		12 S 2		12 N 2	
Pairs	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.	Prob>F	signif.
(1,2)	.0001	0.02%	.0001	0.02%	.0001	0.02%	.0001	0.02%
(2,3)	.0092	0.95%	.0001	0.02%	.0001	0.02%	.0001	0.02%
(3,4)	.0891	N.S.*	.4219	N.S.	.6833	N.S.	.0353	3.6%
(4,5)	.4892	N.S.	.1226	N.S.	.9302	N.S.	.6591	N.S.
(5,6)	.3023	N.S.	.3763	N.S.	.8702	N.S.	.6448	N.S.
(6,7)	.9652	N.S.	.6354	N.S.	.1491	N.S.	.0233	2.4%
(7,8)	.6915	N.S.	.3498	N.S.	.7751	N.S.	.2675	N.S.
(8,9)	.0770	N.S.	.4466	N.S.	.4762	N.S.	.7370	N.S.
(9,10)	.9091	N.S.	.8203	N.S.	.8721	N.S.	.9377	N.S.
(10,11)	.6153	N.S.	.6167	N.S.	.9582	N.S	.7119	N.S.
(11,12)	.6867	N.S.	.6783	N.S.			.8582	N.S.
(12,13)	.7307	N.S.	.2368	N.S.			.2062	N.S
(13,14)	.3791	N.S	.5244	N.S			.3586	N.S.
(14,15)							.5819	N.S.
(15,16)							.3536	N.S.
(16,17)								
(17,18)								
(18,19)								

* N.S. = Non-Significant

TABLE 4.23. PAIRWISE TESTS FOR ALL THE DATA TOGETHER

Pairs	Probability >F	significance level
(1,2)	0.0001	0.02%
(2,3)	0.0001	0.02%
(3,4)	0.0001	0.02%
(4,5)	0.0011	0.2%
(5,6)	0.3117	N.S.*
(6,7)	0.0238	3%
(7,8)	0.2009	N.S.
(8,9)	0.3244	N.S.
(9,10)	0.9990	N.S.
(10,11)	0.2526	N.S.
(11,12)	0.6542	N.S.
(12,13)	0.5579	N.S.
(13,14)	0.0378	4%
(14,15)	0.0078	0.8%
(15,16)	0.1585	N.S.
(16,17)	0.3440	N.S.
(17,18)	0.1849	N.S.
(18,19)	0.6253	N.S.

* N.S. = Non-Significant

The number of vehicles after which the headway values approach a constant value, differs from study to study. Helm [Ref. 6,1958] and Capelle [Ref. 7, 1961] reported that vehicles after the 2nd depart at steady rates; Agent et al. [Ref. 20, 1982] vehicles after the 3rd; Leong [Ref. 10, 1964], Carstens [Ref. 15, 1971], and King et al. [Ref. 17, 1976] vehicles after the 4th; Greenshields et al. [Ref. 2, 1946] vehicles after the 5th; Ancker et al [Ref. 13, 1967] vehicles after the 7th; and Steuart et al. [Ref. 19, 1978] vehicles after the 9th approach a constant departure headway value.

EQUATION DEVELOPMENT

It was intended to develop an equation which would relate the time required for a number of stopped vehicles at a signalized intersection to pass the reference line with the start-up lost time, *L*, and the time headway, *H*, between vehicles. The equation can have the following form.

$$G = L + H*n \qquad \text{for } n \geq a \qquad (4.1)$$

where G = time needed for the front of n vehicles in a single-line stopped queue to cross a designated reference line at a signalized intersection after the signal indication changes to green

L = start-up lost time

H = average time headway between successive vehicles

n = number of vehicles that cross the reference line

a = the number of vehicles that contribute to the start-up lost time

From Tables 4.19 through 4.23 the variable *a* could be determined. On the assumption that vehicle headways remain constant after the front of the *a* th vehicle had crossed the reference line, the start-up lost time was measured, and the results are shown in Table 4.24. In order to estimate the variable *H*, the mean values for the positions from *a* to the end of the queue were calculated. The variable *L* was the difference of the means of the *a* positions and the average time headway for the remaining vehicles, *H*. The equations that were developed are shown in Table 4.25.

The start-up lost time varied from 0.35 to 1.39 seconds. The constant value of headway that was approached by vehicles, after the ones that contributed to the start-up lost time, ranged from 1.76 to 1.94 seconds. When all data were analyzed together, the overall average start-up lost time of 1.34 seconds can be attributed to the first four vehicles and the average headway, H, after the fourth vehicle was 1.82 seconds.

The start-up lost times observed in this study are considerably smaller than the ones reported in the literature. Differences can be expected in start-up lost times when different screen line definitions are used. In this study, the elapsed time from the beginning of the green interval includes reaction (PIJR) time, but no acceleration time.

TABLE 2.24. START-UP LOST TIME OF PASSENGER CARS AT SIGNALIZED INTERSECTIONS

Approach code	*a*	Average headway for the first *a* vehicles (sec)	Average headway after the *a* th vehicle (H) (sec)	Start-up lost time (L) (sec)
1S3	3	2.22	1.76	1.39
1S4	2	2.46	1.92	1.08
1N3	3	2.13	1.81	0.95
1N4	3	2.15	1.81	1.02
6S2	2	2.38	1.92	0.92
6S3	2	2.11	1.93	0.35
6N2	2	2.20	1.87	0.65
6N3	3	2.19	1.87	0.95
9S1	2	2.19	1.92	0.54
9S2	2	2.35	1.89	0.92
10N1	2	2.30	1.84	0.91
10N2	2	2.36	1.86	0.99
11S2	2	2.24	1.94	0.60
11N2	2	2.20	1.83	0.74
12S2	2	2.11	1.93	0.36
12S3	3	2.10	1.78	0.96
ALL	4	2.16	1.82	1.34

TABLE 4.25. EQUATION (4.1), FOR EACH LANE SEPARATELY AND ALL LANES TOGETHER

Approach code	Time, G, (sec) required for *n* stopped vehicles to cross the reference line	
1S3	$1.39 + 1.76 * n$ when	$n \geq 3$
1S4	$1.08 + 1.92 * n$	$n \geq 2$
1N3	$0.95 + 1.81 * n$	$n \geq 3$
1N4	$1.02 + 1.81 * n$	$n \geq 3$
6S2	$0.92 + 1.92 * n$	$n \geq 2$
6S3	$0.35 + 1.93 * n$	$n \geq 2$
6N2	$0.65 + 1.87 * n$	$n \geq 2$
6N3	$0.95 + 1.87 * n$	$n \geq 3$
9S1	$0.54 + 1.92 * n$	$n \geq 2$
9S2	$0.92 + 1.89 * n$	$n \geq 2$
10N1	$0.91 + 1.84 * n$	$n \geq 2$
10N2	$0.99 + 1.86 * n$	$n \geq 2$
11S2	$0.60 + 1.94 * n$	$n \geq 2$
11N2	$0.74 + 1.83 * n$	$n \geq 2$
12S2	$0.36 + 1.93 * n$	$n \geq 2$
12S3	$0.96 + 1.78 * n$	$n \geq 3$
ALL	$1.34 + 1.82 * n$	$n \geq 4$

In 1973, Kittelson [Ref. 17] investigated the effect of two screen lines on queue discharge headways. Time-lapse photography was used at 5 frames/sec at a single-lane approach adjacent to the Evanston campus of Northwestern University. His films have data for analyzing effects of five screen-line definitions on starting delay for the first vehicle and on headways for subsequent vehicles. The choice of a screen-line definition affects headways for both queue position 1 (starting delay) and queue position 2.

Other location factors, noted by Kittelson, that affect the length of the start-up time for the first queued vehicle for different screen-line definitions include:

1. the distances between the stop line, the crosswalk lines, and the point of intersection entry;

2. the extent to which drivers tend to stop behind the stop and crosswalk lines when stopping;
3. the extent to which the side-street yellow signal is visible to the drivers; and
4. whether a yellow signal is displayed after the red and just prior to the green, as in some European countries.

Schwarz [Ref. 17] studied starting delays in 1961 at seven intersections in Chicago before and after elimination of a "get ready to go" yellow varying from 1.7 to 2.6 seconds. Using the screen-line as the crosswalk or downstream line and measuring headways when the rear wheels of the vehicle crossed that line, he found that starting delays with the advance yellow averaged 1.20 sec. lower (2.97 versus 4.17) than with the red-green sequence. The differences were significantly different. Distances from stop lines to his crosswalk screen lines varied from 5.6 to 11.5 m (18.4 to 37.8 ft).

Agent and Crabtree [Ref. 21] reported that results showed no apparent relationship between lost time, at the beginning of the phase, and distance from stop bar to intersection.

As it can be seen From Table 4.25, the difference in the time required for n vehicles to cross the reference line between the different locations increases as n becomes larger.

FACTORS AFFECTING THE HEADWAYS

The need for reliable and readily obtainable estimates of vehicle headways has generated a considerable amount of research devoted to the determination of the factors affecting vehicle headways. Table 4.26 summarizes the various elements typically considered within each of the factors [Ref. 20, 29].

In this study, the first step was to test the hypothesis that all mean start-up lost times and headways for each lane were equal. The results are shown in Table 4.27. The hypothesis can be rejected at the 0.001 significance level, which means that all mean start-up lost times and headways in each lane are not equal. Note that the minimum value that the statistical software used could report was 0.0001.

Bartle et al. [Ref. 4, 1952] found that the effect of location was significant for both the starting delay and the average time spacing. On the other hand, Carstens [Ref. 15, 1971] reported that the differences among the results for various lanes, approaches and intersections were insignificant.

Figure 4.1. Time to pass the nth vehicle from the reference line.

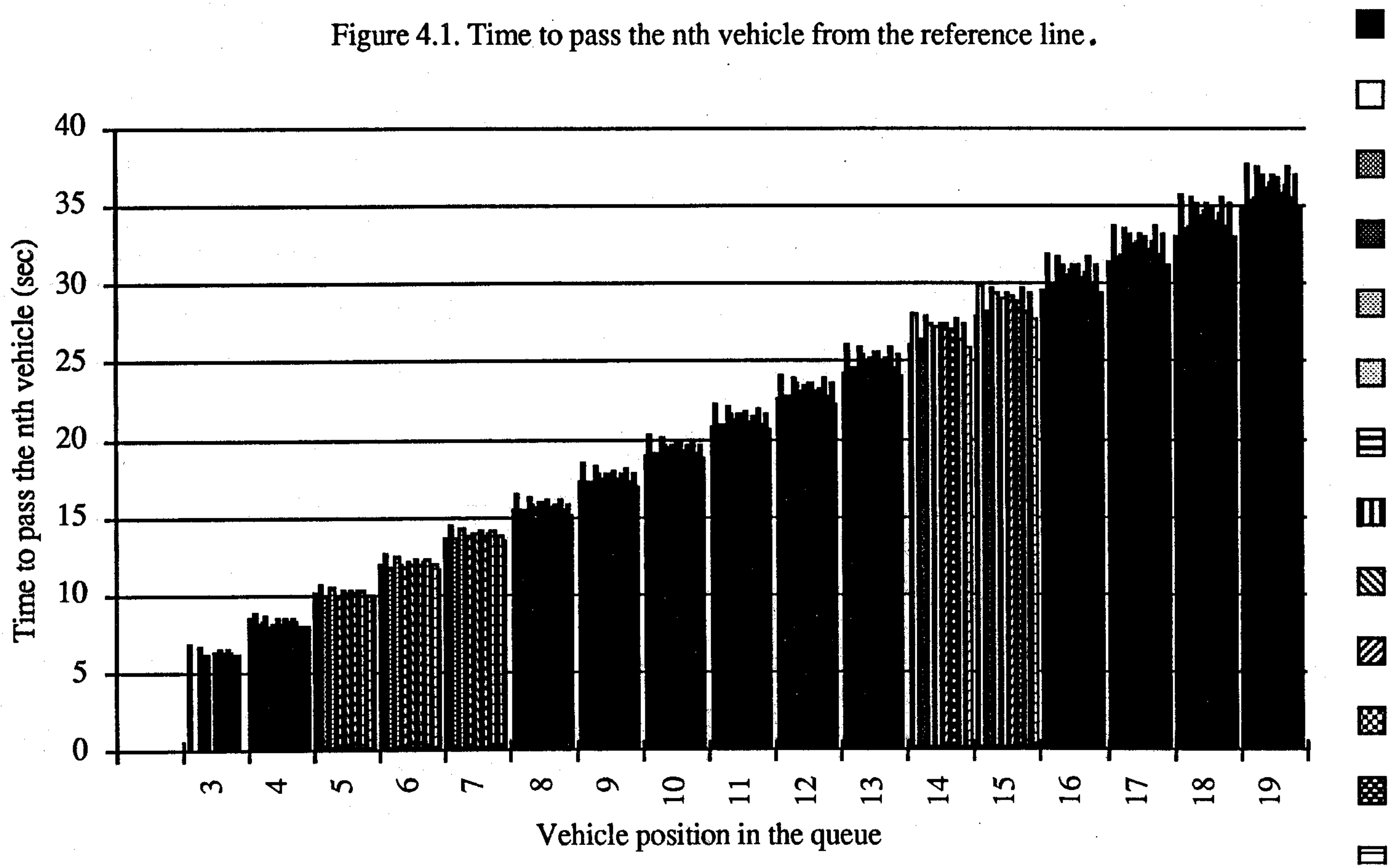

TABLE 4.26. SUMMARY OF PRINCIPAL FACTORS AFFECTING HEADWAYS AT SIGNALIZED INTERSECTIONS

Factors	Elements affecting vehicle headways
Geometrics	Width of approach
	Width of lanes
	Number of lanes
	Grade
	Radius of turns
	Length of turn bay
Operating conditions	Signal timing and phasing arrangements
	Peaking characteristics
	Parking activities
	Bus stop operations
Traffic characteristics	Traffic composition
	Turning movements
	Pedestrian activity
Environmental and other factors	Weather
	Driver behavior
	Area population
	Roadway surface conditions
	Adjacent land uses

TABLE 4.27. ANALYSIS OF VARIANCE

	Source	DF	SS	Mean Square	F value	Pr > F*
Start-up lost time	Model	15	20.871632	1.391442	5.40	0.0001
	Error	1576	406.241520	0.257767		
	Corrected Total	1591	427.113152			
Headway	Model	1	19.409348	1.293957	6.96	0.0001
	Error	5914	1096.744632	0.185889		
	Corrected Total	5915	1116.153980			

* The minimum value that the statistical software used can give is 0.0001

The second part of the conducted analysis, in the study reported herein, focused on finding out whether selected factors were influencing start-up lost times and vehicle headways. Because the variety of existing conditions was limited at the intersections observed, the factors that could be examined included only lane width, lane position, time of day and posted speed limit. Because most of these factors could not be expressed in continuous terms, factor analysis or regression analysis could not be applied. Therefore, a simple comparative approach was engaged.

In this approach, the factors were analyzed one at a time. Analysis was performed by limiting values of all but one important variable, allowing that variable to vary, and observing the effect of that variable on the start-up lost time and the headways.

For the investigation of the lane width factor, it was necessary to group the studied lanes into three groups, in order to have a more dependable number of observations within each group. The groups are the following:

1. Lane width less than 10.5 ft
2. Lane width between 10.5 ft and 13.0 ft
3. Lane width more than 13 ft

The analysis for the lane position effect, consisted from four different groups. The lane position was measured from the centerline of the investigated approach (i.e. lane position 1 indicates that is the lane closest to the centerline, usually called inside lane). The four groups were for lane positions 1, 2, 3, or 4.

The effect of the time of the day was examined through comparisons for A.M. peak period and P.M. peak period.

The data were collected at areas with posted speed limit of 30 mph, 35 mph, 40 mph, and 45 mph. The effect of the speed limit was investigated for the four distinct groups.

Again, analysis of variance (ANOVA) was performed for unbalanced data sets. The hypothesis tested was that all mean start-up lost times and headways were equal for each group. The results are presented on Tables 4.28 through 4.31.

The hypothesis of equal means, for the lane width and the lane position factors can not be rejected at the 0.01 level of significance, and the effects of these factors are thus found to be insignificant for both the start-up lost times and the vehicle headways.

For the analysis of the time of day and speed limit factors, the hypothesis of equal means can be rejected at the 0.01 level of significance and the effects of these factors are thus found to be significant for the start-up lost times and the vehicle headways.

TABLE 4.28. ANOVA FOR THE LANE WIDTH EFFECT

	Source	DF	SS	Mean Square	F value	Pr > F
Start-up	Model	2	1.312778	0.656389	2.45	0.0867
lost time	Error	1589	425.800373	0.267968		
	Corrected Total	1591	427.113152			
	Model	2	1.188875	0.594438	3.15	0.0428
Headway	Error	5913	1114.965105	0.188562		
	Corrected Total	5915	1116.153980			

TABLE 4.29. ANOVA FOR THE LANE POSITION EFFECT

	Source	DF	SS	Mean Square	F value	Pr > F
Start-up	Model	3	2.229422	0.743141	2.78	0.0400
lost time	Error	1588	424.883730	0.267559		
	Corrected Total	1591	427.113152			
	Model	3	1.819848	0.606616	3.22	0.0218
Headway	Error	5912	1114.334132	0.188487		
	Corrected Total	5915	1116.153980			

TABLE 4.30. ANOVA FOR THE TIME OF DAY EFFECT

	Source	DF	SS	Mean Square	F value	Pr > F
Start-up	Model	1	3.524928	3.524928	13.23	0.0003
lost time	Error	1590	423.588224	0.266408		
	Corrected Total	1591	427.113152			
	Model	1	6.294217	6.294217	33.54	0.0001
Headway	Error	5914	1109.859763	0.187667		
	Corrected Total	5915	1116.153980			

TABLE 4.31. ANOVA FOR THE SPEED LIMIT EFFECT

	Source	DF	SS	Mean Square	F value	Pr > F
Start-up lost time	Model	3	4.440086	1.480029	5.56	0.0009
	Error	1588	422.673066	0.266167		
	Corrected Total	1591	427.113152			
Headway	Model	3	4.687441	1.562480	8.31	0.0001
	Error	5915	1111.466539	0.188002		
	Corrected Total	5915	1116.153980			

As it can be seen from Table 4.30, the time of day (A.M. and P.M. peak period) is a significant factor for both the start-up lost time and the headways. Because of the limited variation of the data, there are no observations for Southbound during the morning peak period and for the Northbound during the afternoon peak period. So effects due to time of day and the direction are confounded, that is, one cannot determine which factor influnces the start-up lost time and the headways. Therefore, the outcome of this analysis suggests that the combination of time of day and direction (Northbound A.M. peak period and Southbound P.M. peak period) have a significant effect on start-up lost time and vehicle headways.

The next step was to determine which groups of the speed limit factor were significantly different. In order to accomplish this the least significant difference test (LSD) was conducted for the start-up lost times and the vehicle headways. The comparisons were made at the 0.01 significance level. The results are shown in Table 4.32.

The last analysis conducted was the effect of speed limit on start-up lost times and departing headways. Due to the strong relationship between the speed limit and the area where the intersection was (see Table 3.1), these two factors were combined for analysis. Thus this result can not be conclusive. The start-up lost times are equal for the first three groups and differ for the fourth, which is an open area with a 45 mph posted speed limit. As far the headways are concerned, they are affected by the area factor only for group 1. The outcome of this analysis is that the start-up lost times are not significantly different for business area with 30 mph speed limit, intermediate area with 35 mph speed limit, and commercial area with 40 mph speed limit; and the vehicle headways are not significantly different for intermediate area with 35 mph speed limit, commercial area with 40 mph speed limit, and open area with 45 mph speed limit.

TABLE 4.32. COMPARISON FOR THE SPEED LIMIT FACTOR

	Area groups	DF	F Value	Prob > F
Start-up lost time	30 mph vs 35 mph	1	0.25	0.6145
	35 mph vs 40 mph	1	1.62	0.2028
	40 mph vs 45 mph	1	16.18	0.0001
Headway	30 mph vs 35 mph	1	21.55	0.0001
	35 mph vs 40 mph	1	3.48	0.0621
	40 mph vs 45 mph	1	3.12	0.0775

Before an overall interprentation of the results was made, the information presented in Table 3.1 was examined. It became obvious that because of the limited variety of intersections examined, it would be difficult to draw definite conclusions about the effects of start-up lost times and departing headways on the factors investigated in this study. For example, the locations are scattered along Congress Avenue and have different characteristics such as adjacent land uses and distance from the CBD (Central Business District). Because of these conditions and perhaps a combination of them, the values measured differ considerably between them. However, because the limited variety of sites studied, the individual impacts of the factors could not be isolated by using statistical methods. Therefore, results would be exaggerated if a particular group contained a large amount of collected data.

In a study by Shawaly et al. [Ref. 30], the results showed a considerable variation in the departure patterns of vehicles crossing the stop line during the peak hours not only during a particular peak period, but also on different dates.

SUMMARY

The mean value of headway generally decreased from front to rear of the queue. This was observed through the Tables 4.1 - 4.17.

The hypothesis of equal means was rejected at the 0.001 level of significance, and the effect of position within the queue on mean headways between pairs of vehicles is thus found to be significant. The general observation is that the start-up lost time of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first 2 or 3 vehicles in line, depending on the case. This result is within the range of values reported in the literature.

The start-up lost times ranged from 0.35 to 1.39 seconds and were attributed to the first two or three vehicles. The vehicle headways which varied from 1.76 to 1.94 seconds, approached a constant value after these first vehicles.

When all data were analyzed together, the overall average start-up lost time of 1.34 seconds can be attributed to the first four vehicles and the average headway after the fourth vehicle was 1.82 seconds.

The hypothesis that all mean start-up lost times were equal was tested. The hypothesis was rejected at the 0.001 significance level. The hypothesis that all mean vehicle headways were equal was rejected at the 0.001 level of significance.

The factors that affect the start-up lost times and the vehicle headways were investigated. It was found that the lane width (which varied from 9.5 to 18.0 ft) and the lane position (for lane positions 1 through 4) did not have a significant effect on either the start-up lost time or the vehicle headways. On the other hand, the combined effect of time of day and direction had a significant effect on both the start-up lost time and the vehicle headways. The combined effect of posted speed limit and area had a significant effect for start-up lost time when the speed limit was 45 mph in an open area and for vehicle headways when the speed limit was 30 mph in a business area.

CHAPTER 5. CONCLUSIONS

As the problems of efficiently transporting people and goods in modern urbanized society persist and intensify, there is accentuated concern for provision and improved operation of systems of both individual and mass transportation facilities. Emphasis has concentrated on freeways and mass transit. Nevertheless, surface arterial streets and highways are paramount, as they continue to bear the brunt of enormous traffic demands. Literally hundreds of billions of vehicle-miles are traveled annually on city streets.

In street and highway systems incorporating signalized intersections, the intersection has always been the cardinal element. To improve the operational effectiveness of the system, it is necessary to develop traffic signalization to its highest possible level of efficiency.

The number of waiting vehicles that can cross a signalized street intersection in a given time depends in the simple case on how soon they move after the signal changes to the green indication, and on how fast they accelerate. The driver of the first vehicle reacts to the signal change or the clearance of the intersection and then each driver in turn reacts until the ripple of motion has traveled to the tail car of the queue, with the progress of the wave of motion depending on individual reaction times. It is the total time required to pass a given number of vehicles through the intersection that is of primary interest to the traffic engineer. In analytical categories, this time depends on the integration of individual patterns of reaction times, acceleration, speed and spacing.

The overall review of the studies found in the literature review indicated that there is a high degree of variability among the start-up lost times and departure headways at signalized intersections. Most studies were based on limited data points and they date back 10 to 30 years ago. Since the traffic and vehicle characteristics have changed over time, the validity of the results of these studies, that do not reflect the current traffic and vehicle characteristics, is questionable. Therefore, there is a need to reexamine the departure headways at signalized intersections. The new departure headways could also be tested for their applicability and transferability to other locations.

Field data were obtained primarily to get more extensive knowledge of behavior at intersections than is now available, but they were not intended to be conclusive.

Two parameters of traffic performance can be used to represent several important characteristics of intersection operation. The two parameters, investigated in this study, are start-up lost time and time spacing of vehicles departing from a signalized intersection. Information on the variability of these parameters at an intersection and among intersections may have useful applications in studies of intersection capacity and signal timing.

The mean value of headway generally decreased from front to rear of the queue. The hypothesis of equal means was rejected at the 0.001 level of significance, and the effect of position within the queue on mean headways between pairs of vehicles is thus found to be significant. The general observation is that the start-up lost time of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first 2 or 3 vehicles in line, depending on the case. This result is within the range of values reported in the literature. The start-up lost times ranged from 0.35 to 1.39 seconds and were attributed to the first two or three vehicles; the vehicle headways which varied from 1.76 to 1.94 seconds, approached a constant value after these first vehicles.

It was also found that the lane width and the lane position (for the range that they were studied) did not have a significant effect on the start-up lost times and the vehicle headways. The combined effect of time of day and direction had also a significant effect on the start-up lost time and the vehicle headways. The combined effect of posted speed limit and the area within the city had a significant effect for the start-up lost time when the speed limit was 45 mph in an open area and for the vehicle headways when the speed limit was 30 mph in a business area.

The results of this study suggest that the overall average start-up lost time of 1.34 seconds can be attributed to the first four vehicles and the average headway after the fourth vehicle was 1.82 seconds.

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