THE SIMULATION OF TRAFFIC TO EVALUATE THE EFFICIENCY OF THE INTERSECTION CONTROL SYSTEM

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INTRODUCTION

One important test in the development of new traffic control methods is an evaluation of proposed alternates. It is impractical to test every alternate on a real time basis, and it is not wise to choose an alternate without comparison testing. With the rapid development of high speed computers, simulation of traffic under the various alternatives has proved to be a valuable means of providing effective comparisons at reasonable cost.

Simulation is a technique which enables the study of a complex traffic system in the laboratory rather than in the field. It is usually faster and less expensive than the testing of a real system. In many cases, it enables study of system characteristics prior to construction of the facility. The modern digital electronic computer presently provides the high speed of computation and the logic capabilities that enables it to react in a manner analogous to vehicles traveling on a roadway (4). By proper programming, the behavior of each vehicle represented within the computer may be calculated by following predetermined patterns, derived from the observation of real vehicles or established mathematical theory. By this method, the precise control of the dynamic traffic process can be maintained and many unnecessary variables eliminated (4).

In considering changing an intersection control system, either providing a separate lane for turning traffic or changing the signal phases, the question usually is to determine which change is the most effective and advantageous. Since the traffic characteristics vary considerably at different locations, the correct answer is not easily obtained by simply applying traffic theories. The direct way would be to make actual changes and evaluate the results for comparison. However, this method is impractical because it is time consuming and expensive. Another way is to solve the problem using a computer simulation model.

This research concerns a computer simulation program that will be used to evaluate the single intersection controlled by traffic signal. The intersection considered has one or two lanes on each approach, and may have separate turning lanes. A traffic signal is used to control traffic in all directions.

The first part of the research is the development of the computer simulation program, written in GPSS (General Purpose Simulation System), and run on the IBM 360/65 computer. The output of the program is a table indicating the traveling times required by vehicles on each approaching lane to cross the intersection, and the queue length of stopped vehicles. The reason for using "delay" or "traveling time" rather than "load factor" as the measuring parameter is because it is more realistic from the street user's standpoint. This parameter is the value that will be used as a measure of comparison when any changes occur.

The second part of the research is field observation. Traffic data and statistics are observed from a selected intersection for use as the input information of the computer program. The time-lapse photography method, in addition to stop watches, was used to record the flow of traffic. The data from this method can be analyzed conveniently by replaying in a single-frame movie projector.

The third and last part is the analysis and comparison of the results. Evaluation of program accuracy and further research needed are discussed.

Development of Previous Traffic Simulation Using the Computer

Since World War II, there has been an increasing use of computers to speed and simplify the mathematical processes involved in simulation. These computers are of two general types: analog (continuous variable) and digital (discrete-variable) (31). Both types have been used for traffic simulation; however, analog simulation has some disadvantages because the vehicle is represented as a passive element completely controlled by external factors. Digital simulation, on the other hand, offers tremendous possibilities in this area by handling elements of the simulation in successive steps (36). Although traffic engineers realize that the digital computer permits bringing the traffic facility into the laboratory with unlimited control conditions, nevertheless, there has been comparatively little work done in this area.

In 1956, there were three digital computer simulations reported in the traffic engineering field. Gerlough simulated freeway traffic on a general-purpose computer (37). Goode et al. developed a model of a signalized intersection using the MIDAC digital computer (38). Wong's paper (39) described the simulation of a portion of a multilane boulevard.

In 1959, Gerlough presented another paper (40) describing computer inputs which may be used to represent the operation of traffic streams. The statistical distributions he mentioned are: Poisson, exponential, shifted exponential, and composite exponential. Lewis (41) and Benhard (42) simulated the intersection of two two-lane streets with actuated signal control. Perchonek and Levy (43), and Wohl (44) applied the digital computer to study the problem of freeway on-ramp operations. Glickstein and associates (45) applied computer simulation techniques to the interchange design problem.

Kell presented two subsequent papers (46, 47) on obtaining vehicle delays at intersections by the application of computer simulation. His studies compared orthogonal intersections of two two-lane, two-way streets, having traffic signal control (fixed time, semi-actuated, or full actuated) with intersections having cross street stop sign control.

Since 1962 Lewis has presented two papers describing the basic theory of simulation techniques at intersections. His first paper (4) details his simulation model and its application to determine the vehicular delays at the intersection of a four-lane major street with a two-lane minor street. The computer output (the delays) was used to determine volume warrants for different types of intersection control. In the second paper (48), Lewis proposed an improved headway distribution which consists of a modified binomial distribution, using two different levels of probability, for use in simulation studies.

In 1966, Dart (1) studied the problems of left-turn traffic at signalized intersections on four-lane arterial streets. He applied the computer simulation technique to obtain the left-turn characteristics.

The delay data was used to observe the factual warrants for left-turn channelization.

In recent years, due to the faster and more efficient generation of computer machines, simulations on computer have been widely employed by traffic engineers. The research in this field has been considerably increased. Separate studies of Beilby (49) and Story (50) used similar techniques to simulate traffic flow by digital computer. Klijnhout (51) simulated a single intersection with traffic signal, his program being written in PL1 language for the third generation IBM 360/65 computer. Rumsey and Hartley (33) simulated a model of traffic flow between two adjacent intersections. Their model emphasized the behavior of platoons created by vehicles leaving one signalized intersection toward the neighboring intersection.

Simulation models for large scale street networks have also been developed in the past few years. Davies et al. (11) wrote a street network model using FORTRAN IV to operate on the CDC 6500 computer. Other large scale models which are well-known at the present time include:

1. TRANS model developed by Planning Research Corporation (52).

2. DYNAMO model developed by MIT (53).

3. UNIST simulator developed by the University of Manchester Institute of Science and Technology (54, 55).

As computer speeds have increased and the complexity of simulations have grown, special-purpose computer languages have been developed to aid in the development and debugging of simulation programs. One of the most widely used of these languages now is GPSS (56). To publicize how a substantial reduction in the programming effort can be accomplished by using the simulator, Blum (57, 58) developed a traffic simulation program written in GPSS II and FAP languages for the IBM 7090/94 systems. His program dealt with traffic flow at various types of intersections and was later adapted for large-scale street networks.

Traffic Behavior at Signalized Intersections

When a vehicle, approaching an intersection, reaches the point where the stream of traffic is influenced by the signal at the intersection, the pattern of flow is changed, depending on vehicle interactions and driver responses. The general description of the nature of traffic at four-lane intersections, illustrated in Figure 1, follows the idea of intersection characteristics described by Dart (1).*

In the case of an isolated intersection, before the vehicles reach the intersection, usually several hundred feet in advance, their characteristics are not yet affected by the intersection congestion. Their arrival times will generally be considered random, making the distribution of successive time spacing between vehicles (inter-arrival time) an exponential relationship.

When passing point A in Figure 1 (the reference point for this model), drivers will notice the changing pattern of the preceding vehicles (the build-up of a queue of stopped vehicles or the slowing of vehicles ahead). When the signal phase is yellow or red, the first independent vehicle in that cycle will not begin to decelerate until about half-way or more from point A to the stop line, designated as

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*The following explanation is selected and modified from Reference (1).

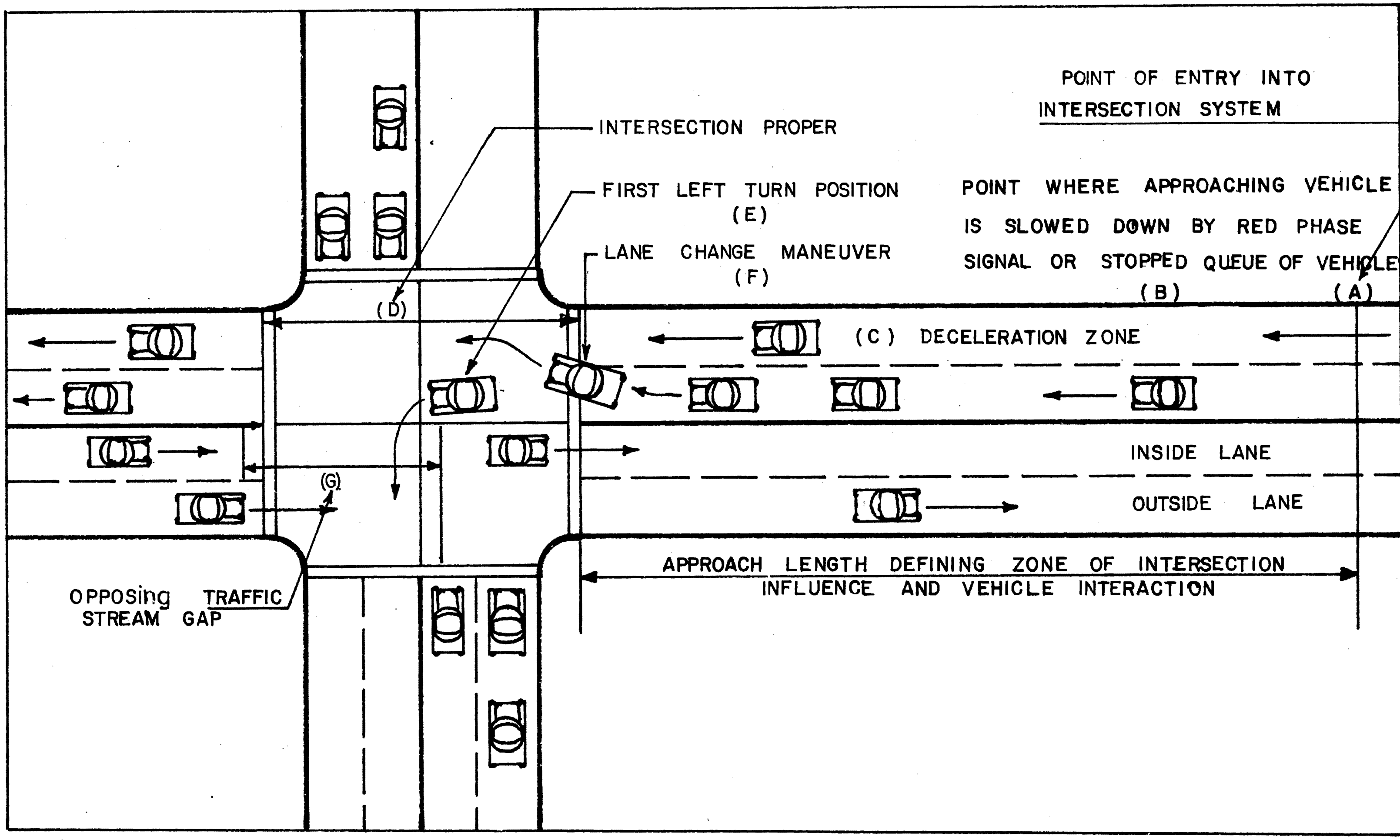


Figure 1. Illustration of Traffic Behaviors at a Typical Signalized Intersection (1)

zone C. This is the zone where the driver, realizing the stop condition ahead, will likely decelerate at a comfortable rate. This zone is also the general area where approaching drivers may suddenly encounter a yellow phase and must quickly decide whether to decelerate at an uncomfortable rate or to continue through the intersection (D, Figure 1). Due to vehicle interaction, the trailing vehicle of a platoon will begin to slow soon after passing point A, usually at the location of point B, between point A and zone C.

There can be interaction between successive vehicles in a traffic lane at any point on the approach between point A and the intersection proper, because of the different speeds of these vehicles. The driver of a faster trailing vehicle will adjust his speed according to the speed differential and headway between the two vehicles. If the driver of the faster vehicle is planning to turn and is already in the proper lane for the turn, then he will likely slow down and follow the slower vehicle in this lane. The driver of a nonturning vehicle in the same situation may not be willing to slow down. When there is a large enough gap in another lane, he may attempt to improve his position by passing the slower vehicle through a lane change maneuver.

If there is no separate left-turn signal phase, the driver of a left-turning vehicle near the intersection in the inside lane must evaluate the opposing traffic streams. Generally, he requires a gap size in the opposing traffic, G, that is large enough for his turning maneuver. He may find such a gap in advance without stopping and make his turn immediately, or he may have to stop near point E and wait for a suitable gap. Under heavy traffic conditions, he may have to wait to turn until after the yellow phase has displayed and all opposing traffic has stopped.

The following vehicle, meanwhile, if it is not a left-turner, may not want to be delayed waiting for the turning vehicle to move and may try to change to the adjacent outside lane, as at point F, if there is a large enough gap for a lane change maneuver.

In the case where there is heavy pedestrian movement crossing the side street, the right-turn vehicle from the outside lane may have to wait until all pedestrians have passed. In this case a build-up of stopped vehicles may occur during the green signal phase, and one or more of the following vehicles, if not turning, may try to make a lane change in the same manner as in the case of the left turn. Still another case may occur where right-turning is allowed during the red signal phase. The first vehicle in the outside lane may turn right after accepting a gap from the cross-street traffic stream.

When the signal changes to green, there is usually a short starting delay before the first vehicle in a queue begins to move into the intersection proper. This delay is the perception and reaction time required for the driver of the first vehicle. Succeeding vehicles will follow with progressively smaller headway until some relatively constant value is reached for the last few vehicles in the queue.

CHAPTER II

VEHICLE BEHAVIOR ASSOCIATED WITH SIGNALIZED INTERSECTIONS

It is obvious that the delay is a most important factor in the determination of intersection efficiency, and thus its causes and characteristics should be studied. Previous studies have indicated that the delay deals primarily with vehicle behavior responding to the intersection control devices. To study the traffic simulation model, it is necessary to realize all of the statistics and characteristics of vehicles associated with the intersection control devices.

Traffic Distribution

There are many variables associated with a traffic system. These concern the characteristics of vehicles, roadway, and drivers. Most of them are of a statistical nature. Due to a lack of knowledge of the distributions and laws of interaction of traffic system components at the present time, the traffic engineer usually fits the distributions to observations of the overall system. The variables usually observed are: flow (rate), inter-arrival times, and speeds. If given the distribution of one variable, it is sometimes possible (when the relationships between variables are known to some extent) to determine the distribution of another through simulation (31).

Greenshields et al. (5) have shown that the vehicle arrivals, with low to moderate flow and with a sufficient number of lanes so that vehicles can pass at will, generally follow the Poisson distribution. Thus, inter-arrival times follow the exponential distribution (as shown in Figure 2). This distribution of arrival times has been extensively employed in many theories concerning vehicular traffic, following Greenshields.

The Poisson theory is based on the random placement of discrete points on a line. A vehicle, however, occupies a finite length of roadway. Thus, the actual characteristics and behavior of vehicles vary considerably from the theoretical at higher traffic volumes. At capacity, arrivals may approach a uniform spacing and the theory is not suitable.

The exponential distribution is continuous as is the physical phenomenon it represents. If vehicles are constrained so that they cannot pass, there will be some minimum nonzero gaps (headways) which can exist between successive vehicles. Gerlough (32) has proposed the shifted exponential distribution (as shown in Figure 3). The idea of this distribution is to translate a small distance, τ, away from the origin along the time axis.

At high traffic volumes, vehicles are not free to select their own positions in the roadways. The distribution existing under this constraint is called platoon behavior. Platoon behavior also occurs at an intersection within a network of a sequence of intersections. Vehicle arrivals are no longer random, but are dependent on the departure pattern from the adjacent intersection. It is observed in practice (33) that vehicles leave a signalized intersection in the form of platoons which

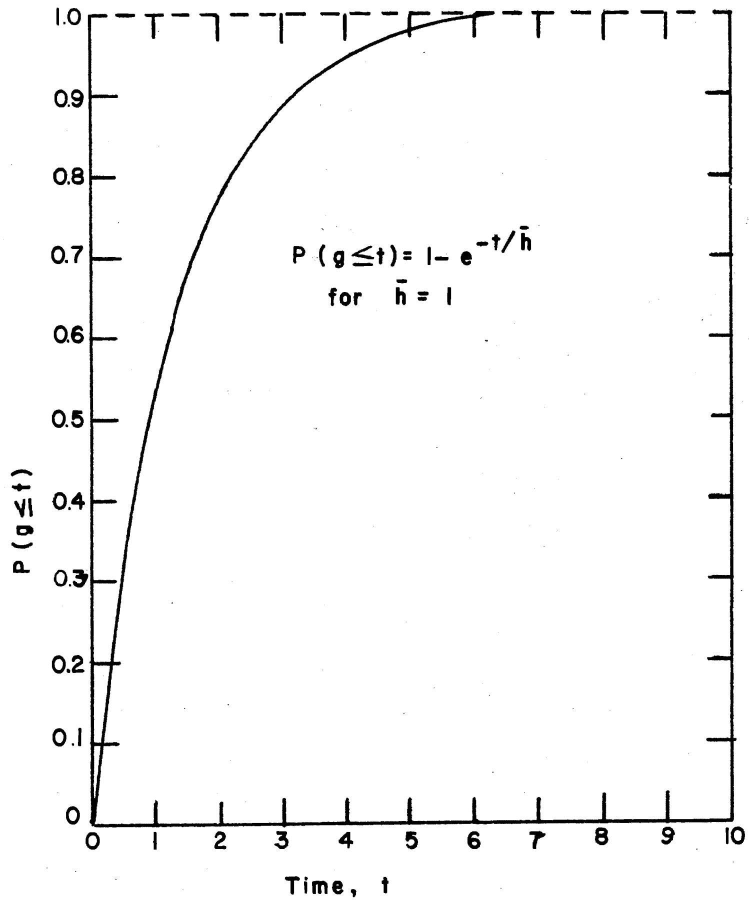


Figure 2. Exponential Distribution of Gaps (Inter-Arrival Times, Where $\overline{h}$ is the Mean of the Gaps) (32)

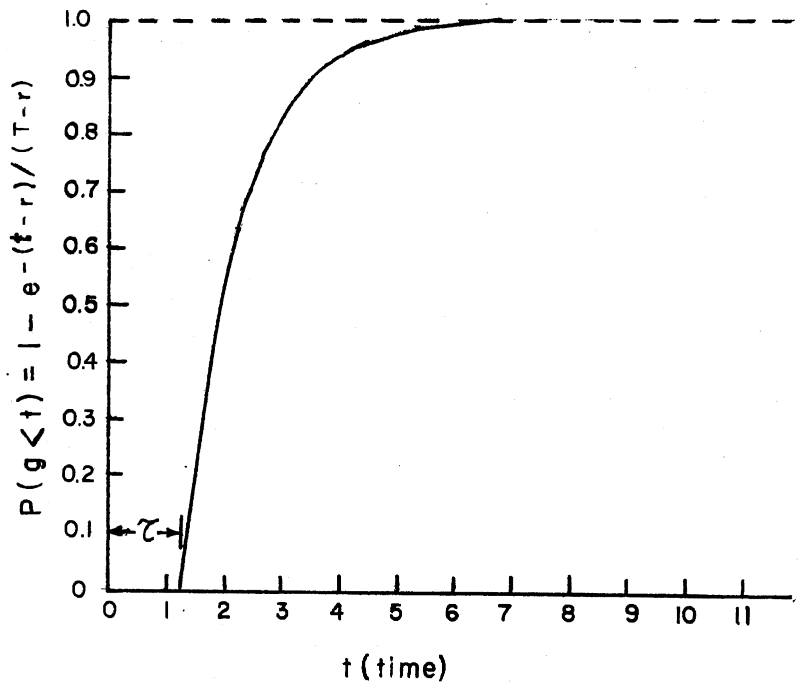


Figure 3. Exponential Distribution of Gaps Shifted From Origin (2)

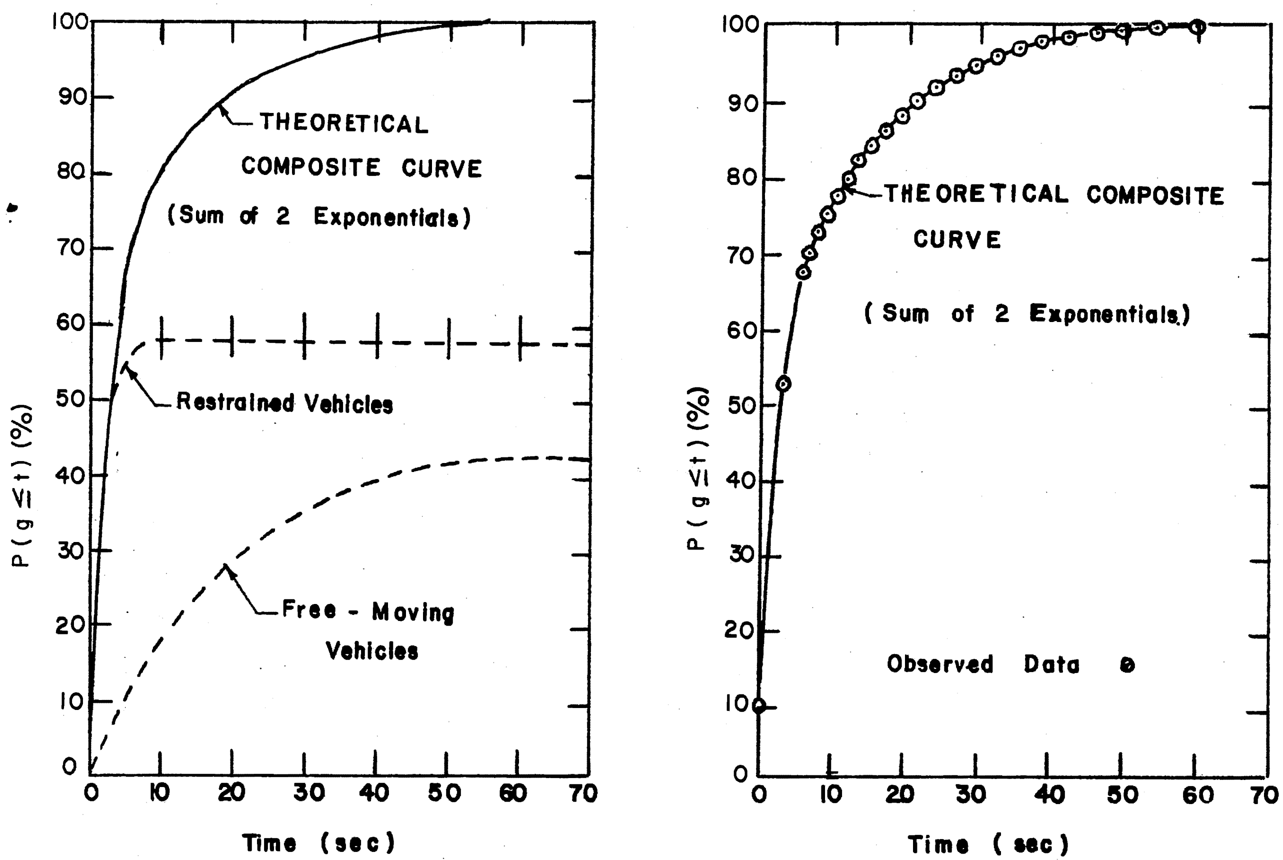
spread out as the vehicles travel toward the neighboring intersection.

Work by Damson and Chimini (34) in fitting the hyperlang probability distribution (a linear combination of the translated [shifted] exponential and the translated erlang distributions) to intervehicular headway, indicates a minimum headway, τ, of 0.75 seconds for unconstrained vehicles. For the constrained vehicles, this parameter varies down to 0.55 seconds, which is indicative of an intervehicular spacing of only 7 or 8 feet. This value is the absolute minimum spacing that can occur in the traffic stream.

Schuhl (35) pointed out that a traffic stream may be regarded as a mixture of free-moving and constrained vehicles, each of which conforms to a Poisson-like behavior. Free-moving vehicles can be represented by an exponential through the origin; constrained vehicles, by a shifted exponential. The composite exponential (32) is the sum of these two exponentials (as shown in Figure 4 (a)). This exponential has compared favorably with data from field observations (Figure 4(b)).

Car-Following Behavior

In the absence of other interfering vehicles, a driver will attempt to keep the speed of his vehicle fairly constant at his desired speed in order to minimize trip duration and maximize safety. When following other vehicles whose speed is within the range of his speed, the driver introduces a new consideration, the intervehicular spacing, the magnitude of which depends on his speed. According to the general rule, the safe spacing is the length of a vehicle (about 15 feet) for every 10 miles per hour of traveling speed.



(a) Theoretical

(b) Comparison of Theoretical and Actual

Figure 4. Composite Exponential Curve for Inter-Arrival Times (32)

The car-following theory developed by Herman et al. (8, 9) is based on the role and interaction of the three components of the traffic stream: road topology (number of lanes, nature of intersection, signals, warning signs, etc.), vehicle characteristics (speed, acceleration and deceleration, vehicle signaling, vision, etc.), and driver behavior (range of perception, lags between perception and response, etc.).

Since the full spectrum of behavior at an intersection involves a tracking or following process, the car-following theory may be used to describe certain patterns of intersection performance. This relationship was applied directly to the problem of processing vehicles in a simulation model developed by Davies et al. (11). The car-following theory, as simplified for use in Davies' model, may be expressed in the general form:

response = sensitivity x stimulus.

This can be expressed in the form,

$$A_{t+T} = a_o \left[\frac{V'_t - V_t}{X'_t - X_t}\right] \tag{2.1}$$

where

A_{t+T} = acceleration of the follower initiated at time $(t + T)$;

T = time lag of the driver-vehicle system;

V'_t, V_t = velocities of the leader and follower, initiated at time t;

X'_t, X_t = positions of the leader and follower initiated at time t;

a_o = characteristic speed.

Drew (10) has developed Equation (2.1) and expressed it in the form:

$$A_{t+T} = a_o \left[\frac{V'_t - V_t}{(X'_t - X_t)^m}\right] \tag{2.2}$$

From his analysis Drew suggested various values of m; for general cases where m = 1, Equation (2.2) converts back to Equation (2.1), which indicated that $a_o = U_m$, the optimum car speed. If m = 2, then $a_o = U_f/k_j$, where U_f is the free speed and k_j is the jam concentration (speed is down to zero in the latter case.

The behavior of the leading vehicle of a queue being discharged from a signal differs from that of a follower. This case was described by Davies et al. (11) as free behavior and may be expressed as:

$$A'_{t+T} = K[TV' - V'_t] \tag{2.3}$$

where

A'_{t+T} = acceleration of vehicle initiated at time t + T;

K = proportionality coefficient;

TV' = target velocity of vehicle;

V'_t = velocity of vehicle at time t.

Derivation of the Spacing Equation

In his simulation model Lewis (4) considered the above discussion as concerned with the capacity or near-capacity situation. He believed that to get relatively realistic results, as far as the traffic simulation is concerned, a car-following model should also be applicable for a wide range of traffic volumes including those that are well below a capacity situation. His derivation of this relationship is based on the premise that vehicles do not collide and are operated in a safe manner. He then developed the equations to calculate space limitations or margin for safety.

The spacing, including the vehicle length and a clear space (between successive vehicles) of vehicles stopped in a queue averages about 22 to 25 feet (12, 13). When vehicles are moving at the same speed the spacing is greater to ensure safety, and may be expressed as (4):

$$S \geq P + 1.09V \tag{2.4}$$

where

S = minimum desired spacing in feet, measured from front to front of adjacent vehicles;

P = minimum stopped vehicle spacing;

V = velocity in ft/sec.

This equation includes the product of brake reaction time and velocity, plus a few feet for safety clearance. It is substantiated by the practical consideration of braking behavior and is approximated by:

$$S \geq P + V \tag{2.5}$$

In general traffic flow, vehicles will not be likely to travel at the same speed. If the following vehicle is traveling at a higher speed than the lead vehicle, then the former has to reduce its speed to provide minimum spacing. The relationship, analyzed by Lewis (4), is shown diagrammatically in Figure 5.

If the lead vehicle maintains a constant speed throughout the maneuver, ($V_1' = V_2'$), then:

$$S_1 = P + V_1' + \frac{1}{2D}(V_1 - V_1')^2 \tag{2.6}$$

where

S_1 = minimum spacing;

V_1' = velocity of lead vehicle;

V_1 = velocity of following vehicle;

D = rate of deceleration.

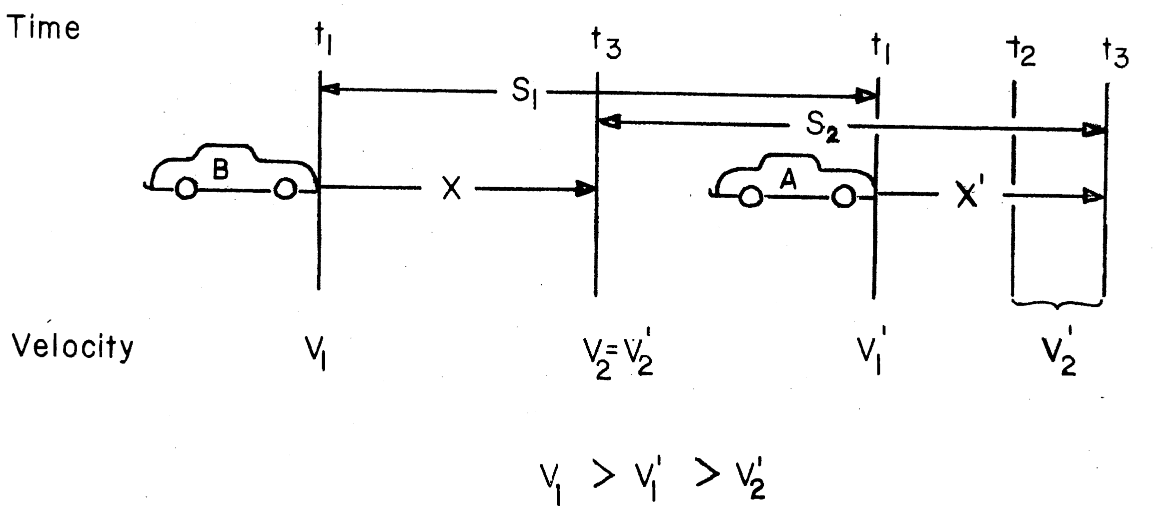


Figure 5. Factors Involved in the Spacing Relationship (4)

If the situation occurs that the lead vehicle decelerates to stop ($V_2' = 0$), then the relation is:

$$S_1 = P + \frac{1}{2D}(V_1^2 - V_1'^2). \qquad (2.7)$$

Starting Headways for Vehicles in a Stopped Queue

This is a short delay between the onset of a green signal phase and the actual movement of a vehicle into an intersection, involving both perception and response. The delay time from the beginning of green phase until the first vehicle in a queue moves into the intersection (rear wheels beyond cross-street curb line) may be assigned the term "starting headway." The study of Greenshields et al. (5), which is still considered applicable by Pignataro (19), showed that the average starting headway is 3.8 seconds. The average headway between successive pairs of vehicles is shown in Table I.

TABLE I

TIME INTERVALS BETWEEN VEHICLES ENTERING A SIGNALIZED INTERSECTION (5)

Vehicle Number	Interval Between Vehicles (sec)	Entrance Time (sec)
1	--	3.8
2	3.1	6.9
3	2.7	9.6
4	2.4	12.0
5	2.2	14.2
6	2.1	16.3
7	2.1	18.4
8	2.1	20.5
9	2.1	22.6
10	2.1	24.7

Bartle et al. (20) studied the starting delay of vehicles at 13 signalized intersections. They observed that mean starting delays range from 2.91 seconds to 4.40 seconds with an average of 3.83 seconds. They considered the remaining vehicles in the queue as a single platoon. The average time spacing, dividing the time for platoon movement by one less than the number of vehicles entering during that time, ranged from 0.95 to 1.63 seconds.

Capelle and Pinnell (21) studied the headways for queues entering signalized intersections at diamond interchanges. They found that the headways decrease rapidly for the first two vehicles in line with a lesser decrease for each succeeding vehicle. They then concluded that instead of using the ordinarily accepted general definition, 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 (as illustrated in Figure 6). Their observations yield a starting delay (for the first two vehicles) of 5.9 seconds and an average headway of 2.2 seconds.

Berry and Gandhi (17) studied the headways of compact platoon vehicles during peak hours. Their observation indicated significantly decreased values for both starting delay and headway during peak hours. The field data showed that starting delay ranges from 2.37 to 2.76 seconds and headway of the platoon ranges from 1.07 to 1.31 seconds. Analysis also indicated that the adverse weather significantly increased headways.

Consideration of the Yellow Phase of a Traffic Signal

When the vehicle is approaching the signalized intersection, in some cases the driver may have difficulty in making a decision. If the green phase is displayed, he will proceed at about the same speed; if it is red, then he will have to decelerate and prepare to stop. However, if the signal indication is turning yellow, then he must decide whether to continue through the intersection or to stop. Gazis, Herman and Maradudin (18) described this zone on an approach to the intersection as a "dilemma zone." It is the zone in which the driver either has to speed up to clear the intersection on the yellow phase or decelerate at an uncomfortable rate to stop at the stop line. Field observation by Dart (1), as summarized in Table II, indicates that the driver's

TABLE II

A STUDY OF DRIVER RESPONSE TO YELLOW PHASE OF SIGNAL (1)

Distance From Intersection When Yellow Phase Starts	Approaching Vehicles			Indicated Probability of Stopping
	Total Number	Number That Stop	Percent Stopping	
0 - 40'	18	1	5.6%	0.056
50 - 80'	17	5	29.4	0.294
90 - 120'	21	17	80.9	0.809
130 - 160'	21	19	90.5	0.905
180 +	41	41	100.0	1.000
Totals	118	83		

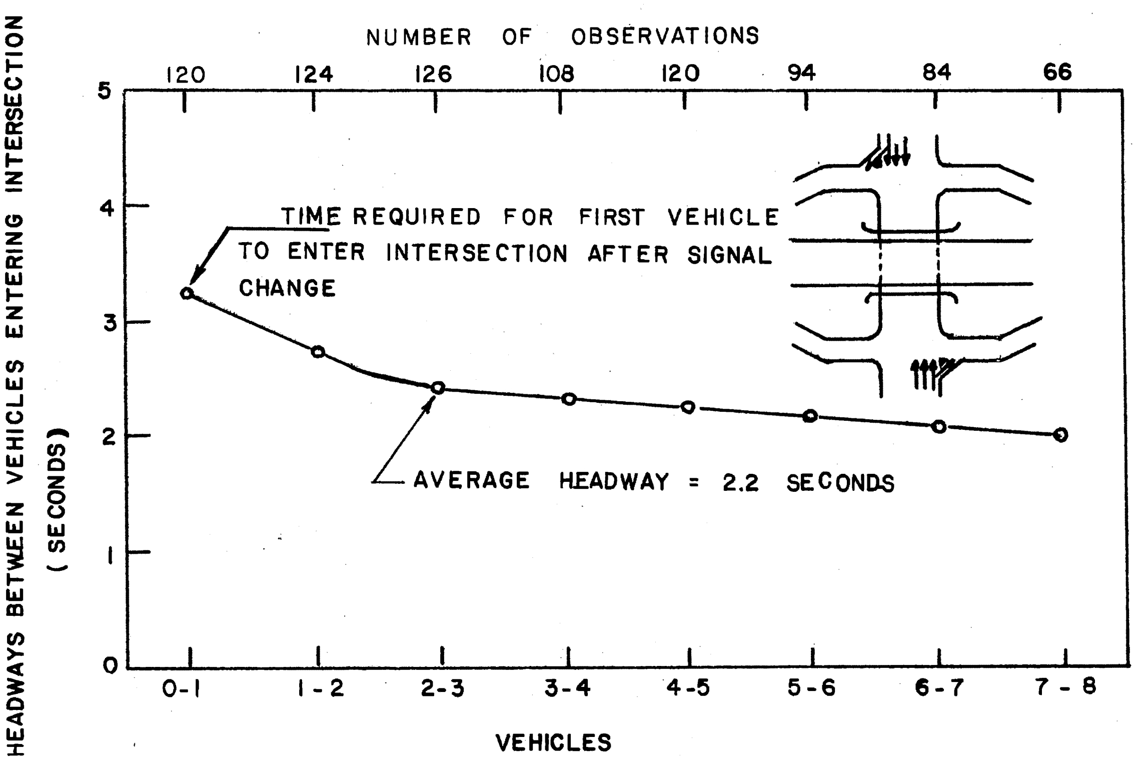


Figure 6. Time-Headways Between Successive Passenger Vehicles (21)

decision depends on the distance between his car and the intersection proper when the yellow phase starts.

During peak hours, Berry and Gandhi (17) observed that about half of the yellow phase length is utilized by traffic. They termed this portion of the yellow phase as the "effective yellow."

Acceleration and Deceleration Characteristics

The laws of motion can be applied to evaluate the relationship of vehicular operating characteristics including speeds, spacings, acceleration, deceleration, and stopping distances (3). For the vehicles moving straight, the formula of straight line motion is applied as follows:

Uniform motion, velocity constant:

$$S = Vt \tag{2.8}$$

Uniform accelerated motion:

$$V = V_o + at \tag{2.9}$$

$$S = V_o t + \frac{1}{2} at^2 \tag{2.10}$$

$$V = V_o + \sqrt{2\ as} \tag{2.11}$$

where

V_o = initial velocity, ft/sec;

V = final velocity, ft/sec;

S = distance in feet;

t = time in seconds;

a = acceleration, ft/sec^2.

A uniform rate of speed change is generally assumed under free flowing conditions. Although observed rates of acceleration are not quite uniform, the uniform case was used in Lewis' simulation model (4) and was considered to supply an adequate approximation of the real case.

Greenshields (5) classified the different forms of speed change from his study of intersection performance. Lewis (4, p. 12) has simplified and summarized this study as follows:

> 1. A chronotropic acceleration is one where delay is independent of the time lost in speed change, such as a vehicle stopping for a red signal. Regardless of the time lost in stopping, the vehicle is still delayed until the signal turns green.
>
> 2. Functional speed change occurs when the loss of time is dependent on the rate of speed change, such as a bus stopping to discharge passengers.

Lewis (4) assumed an acceleration rate of 3 ft/sec^2, while an average velocity of 30 miles per hour or 44 ft/sec was selected in his simulation model. This value is functional and used for the free-flow acceleration. When vehicles are under the pressure of traffic flows, higher rates of acceleration are used. For the case of vehicles accelerating from a stopped or near-stopped condition at stop signs or signals, and also the case of left turn maneuvers, these vehicles must accelerate rapidly to take advantage of available gaps in the traffic stream. For these cases accelerations of 6, 5 and 4 ft/sec^2 were assumed for the first three queued vehicles, respectively. The free-flow acceleration of 3 ft/sec^2 was applied thereafter.

One study of vehicle acceleration (60) indicated that maximum accelerations up to 14.67 ft/sec^2 (10.0 mi/hr/sec) were obtained from the field. It is believed that about 10 ft/sec^2 is the limiting value for comfort if maintained for any length of time, and that at least one-half second should be used to change from zero acceleration to 10 ft/sec^2 (6).

For vehicles decelerating to a stop, previous studies have shown that the maximum deceleration rate of vehicles varies from 20.2 to 28.9 ft/sec^2 (3). These figures are much higher than any observed under ordinary conditions and those used in most simulation models.

The deceleration rates should not be determined by the vehicle itself, but by passenger reaction. Deceleration rates greater than those now practically applied are very probably not desirable because of human limitations and response. The limitation is the length of time required by an individual to adjust himself to externally applied forces. The National Safety Council has adopted a deceleration rate of 17 ft/sec^2 as the maximum for comfort (7). It is found that practical values of deceleration used in every day traffic conditions range from 4.84 to 7.77 ft/sec^2 (60). Wilson's studies (6) showed an average deceleration rate of passenger vehicles with relatively unimpaired comfort for their passengers to be 8.55 ft/sec^2. Additional values of deceleration rates cited by Baerwald (3, p. 26-27) are:

1. 11 ft/sec^2--considered undesirable but not alarming to passengers.
2. 14 ft/sec^2--packages may slide off the seat, and the occupants of the vehicles find this rate uncomfortable.
3. above 20 ft/sec^2--the occupants must brace themselves firmly to avoid being thrown off the seat, will be used only in emergency situations.

Stopping Performance

There are two types of stops that occur at the intersection, stoppint first in line at the intersection and stopping behind another stopped vehicle. It is accepted in practice that the use of a constant deceleration stopping model is realistic. When they have the choice, drivers tend to decelerate at an approximately constant rate throughout the duration of their stops (11). Based on the motion equations with uniform deceleration, Equation (2.11) can be expressed for each time increment as (4):

$$V_t^2 = 2D(X - ZD) \tag{2.12}$$

where

V_t = velocity at time t;

D = deceleration rate;

X = the distance between the vehicle at time t - 1 and the stopping point;

ZD = the distance traveled during one time increment.

Equation (2.12) has been further analyzed by Lewis (4) to obtain the stopping restriction related to the velocity of vehicle at time t-1, when the vehicle starts to decelerate. Finally, the relationship becomes:

$$ZD = \frac{1}{2} V_{t-1} - \frac{D}{4} + \left[\frac{D^2}{16} - \frac{D}{4} V_{t-1} + \frac{D}{2} X\right]^{1/2} \tag{2.13}$$

where

V_{t-1} = velocity at time t-1.

Turning Performance

Vehicles that desire to turn left or right at an intersection at some point must abandon free-flow operating and accept the turning schedule. They should not be operated in excess of a maximum safe speed during the turn. Maximum turning velocity is related to turning radius and side friction by the equation:

$$V_{max} = \sqrt{fgr} \tag{2.14}$$

where

V_{max} = maximum turning velocity, ft/sec;

f = coefficient of friction;

r = turning radius;

g = acceleration of gravity.

During this maneuver it is assumed that a free-flowing vehicle will decelerate uniformly up to a point during the turn which is called "turn point" (4). Once past the turn point the vehicle is free to accelerate normally. Vehicles having a high initial speed start decelerating at some point prior to turning and start to accelerate at some point during the turn, while the vehicles with low speed may accelerate throughout the entire turning maneuver.

It has been observed from previous field observations (14, 15, 16) that the maximum velocity at the turning point is about 15 ft/sec. Since turning speeds depend on the turning radius, there is a tendency to use a slower turning speed for right turns than for left turns due to the shorter turning radius available. In the case of lack of interference for right turns, the opposite may occasionally be true.

Gap Acceptance for Left Turn Maneuver

When approaching a signalized intersection without a separate left-turn phase, the left-turning driver has to evaluate the gap sizes in the opposing traffic stream and select an opening that is large enough to cross through safely. Acceptance of a gap suitable for attempting the left-turn maneuver depends on the characteristics of driver, intersection, and traffic situation (22), so that accepted gaps will not be the same size.

The waiting driver considers each gap, h, in the opposing traffic stream. He will either cross (accepts the gap if $h \geq \tau$) or wait (rejects the gap if $h < \tau$). The value of τ, the critical gap, was assumed to be a single constant value by early theorists (23).

Kaiser (24), from a study at an unsignalized intersection, found that the smallest gap accepted was 3.75 seconds and the largest gap rejected was 4.75 seconds. Noblitt (25) showed that acceptable gaps for left-turn truck combinations were 1.4 to 1.8 times as large as the required gap for passenger cars, and 1.2 to 1.5 times as large as the required gap for single-unit trucks. Kell (26) summarized the data from 500 field observations on two-land two-way streets. His left-turn gap acceptance distribution is shown in Table III.

TABLE III

GAP ACCEPTANCES BY LEFT-TURN VEHICLES, TWO-LANE, TWO-WAY STREETS AFTER KELL (26)

Gap Size	Cumulative Percent Accepting	Gap Size	Cumulative Percent Accepting
$<$ 1.0 sec	0	$\leq$ 5.0 sec	94.7
$\leq$ 1.5	1.4	$\leq$ 5.5	96.4
$\leq$ 2.0	10.2	$\leq$ 6.0	97.9
$\leq$ 2.5	18.3	$\leq$ 6.5	98.2
$\leq$ 3.0	31.3	$\leq$ 7.0	98.5
$\leq$ 3.5	50.0	$\leq$ 7.5	99.3
$\leq$ 4.0	64.6	$\leq$ 8.0	99.4
$\leq$ 4.5	85.3	$>$ 8.0	100.0

Solberg and Oppenlander (27) studied the lag and gap acceptances for drivers entering and crossing a major roadway from a stopped position. They observed that the overall median acceptance time for left-turn movements was 7.82 seconds. The distribution of this study is illustrated in Figure 7.

At signalized intersections, the left-turn critical gaps, as shown by Behnam (22), are depicted in Figure 8. The point of intersection of the two curves in this picture represents the average critical gap, which is 4.5 seconds.

The speed of vehicles is an important factor in considering the gaps to be acceptable for a driver. In all cases, the driver of a stopped vehicle will require a larger size of gap than the driver of a moving vehicle. Figure 9 (23) illustrates the results of data gathered by Texas Transportation Institute on several Texas freeways to analyze gap acceptances for moving and stopped vehicles merging at the freeway entrance ramp.

Weiss and Maradudin (28) developed a method of treating gap acceptance delay which accounts for driver impatience. They believed that the size of acceptable gap is reduced as delay increases. Instead of a constant size of acceptable gap, τ, originally desired by the drivers, they will probably accept a shorter gap. The probability of a driver accepting a gap of size H after the ith vehicle has passed is $F_i(H)$, or

$$F_0(H) \leq F_1(H) \leq . . . \leq F_i(H).$$

Wagner (29) studied the gaps and lags that were accepted by the driver of a side-street vehicle stopped at the stop sign, waiting to enter the intersection. He compared the gap acceptance distribution

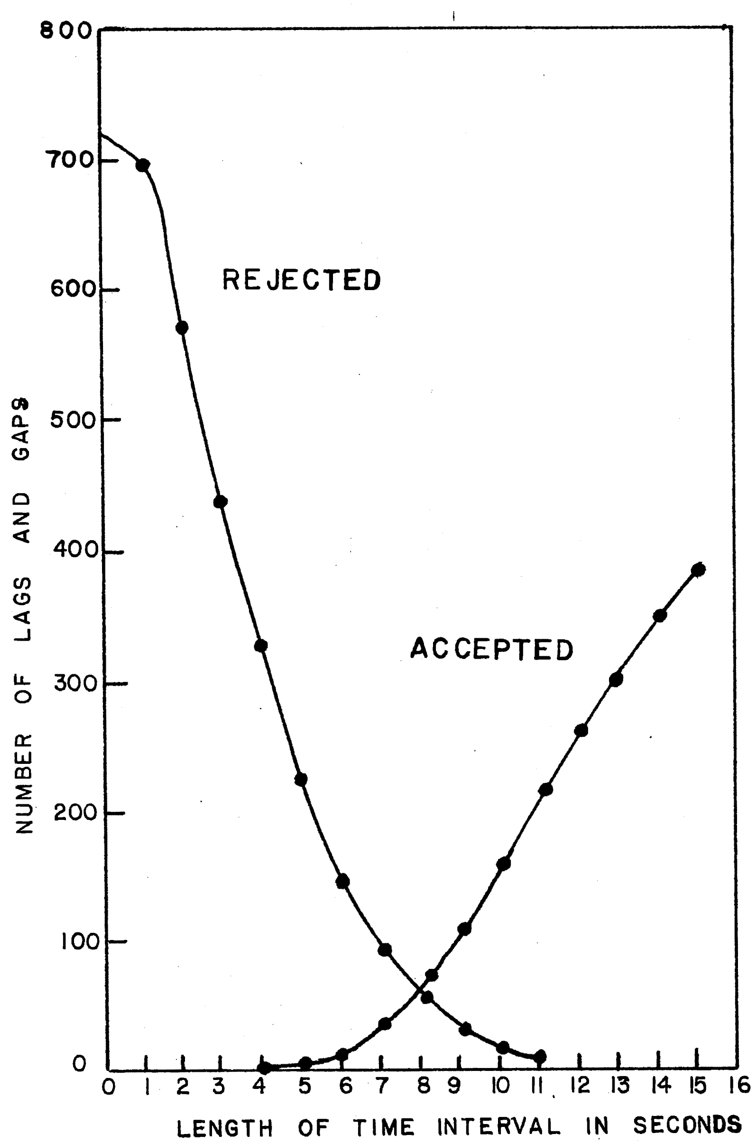


Figure 7. Distribution of Accepted and Rejected Lags and Gaps at Intersection Left Turns (27)

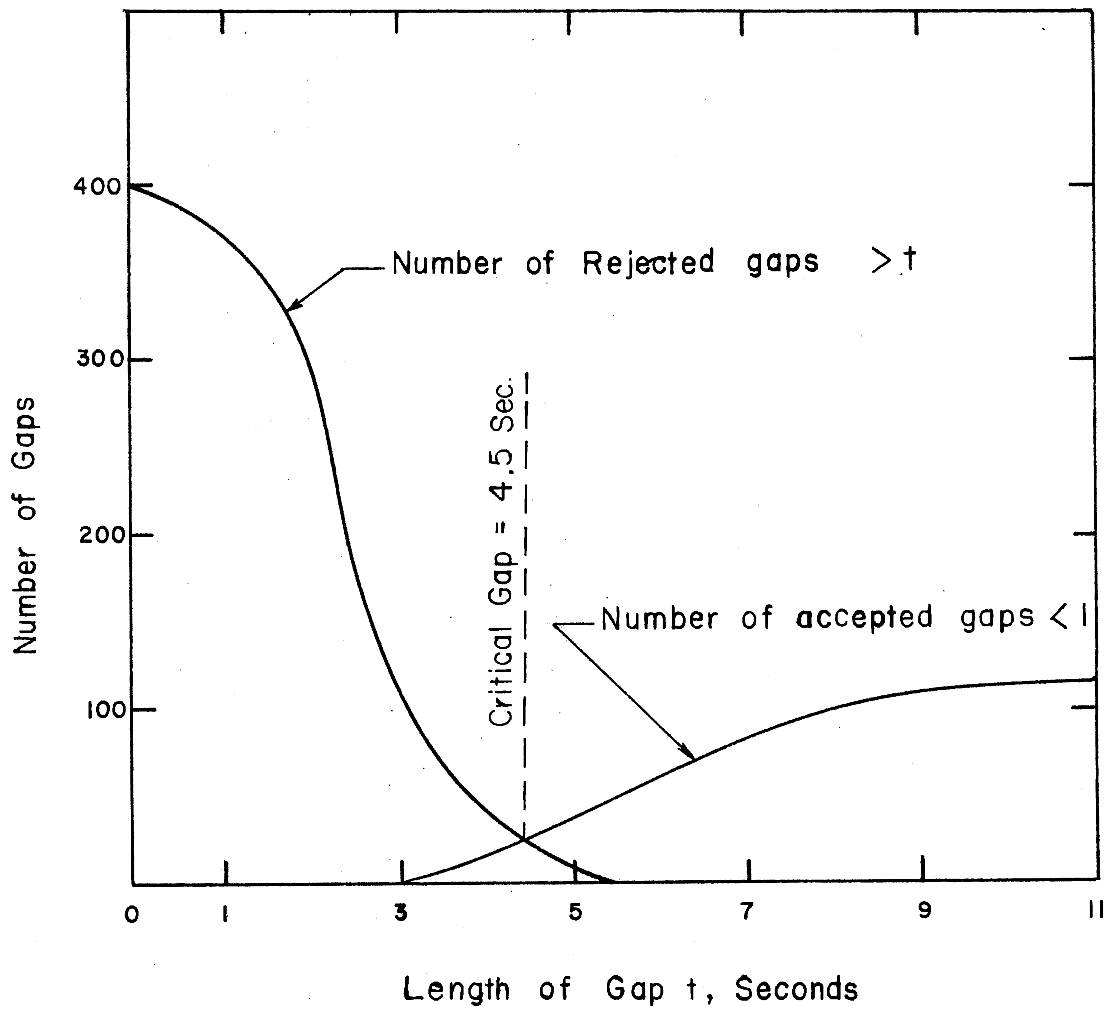


Figure 8. Cumulative Distribution of Accepted and Rejected Gaps at Signalized Intersections (22)

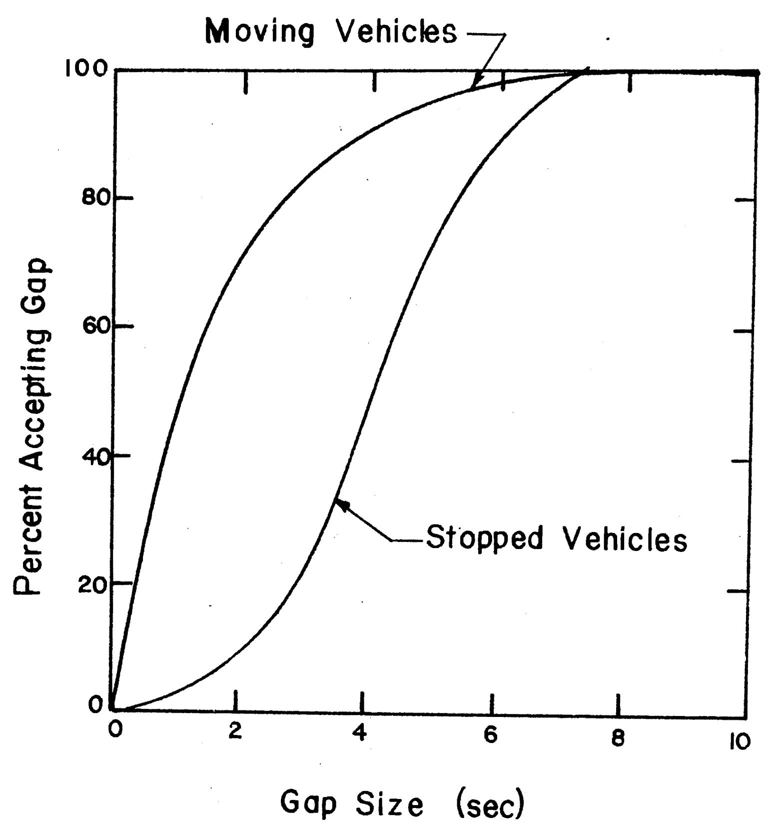


Figure 9. Gap Acceptance of Merging Vehicles (23)

for right-turn, straight, and left-turn vehicles, as shown in Figure 10. This analysis indicated no significantly different results among these maneuvers.

Dart (30) collected his field data about various types of gap acceptances, as shown in Figure 11, on four-lane approach signalized intersections. He indicated the relationships as shown in Figure 12. This research also indicated that there is the probability of 0.145 or 14.5% that the first vehicles of the left turn channelization queue will make the turn before the opposing traffic enters the intersection when the signal turns green. This action is termed "jump-the-gun."

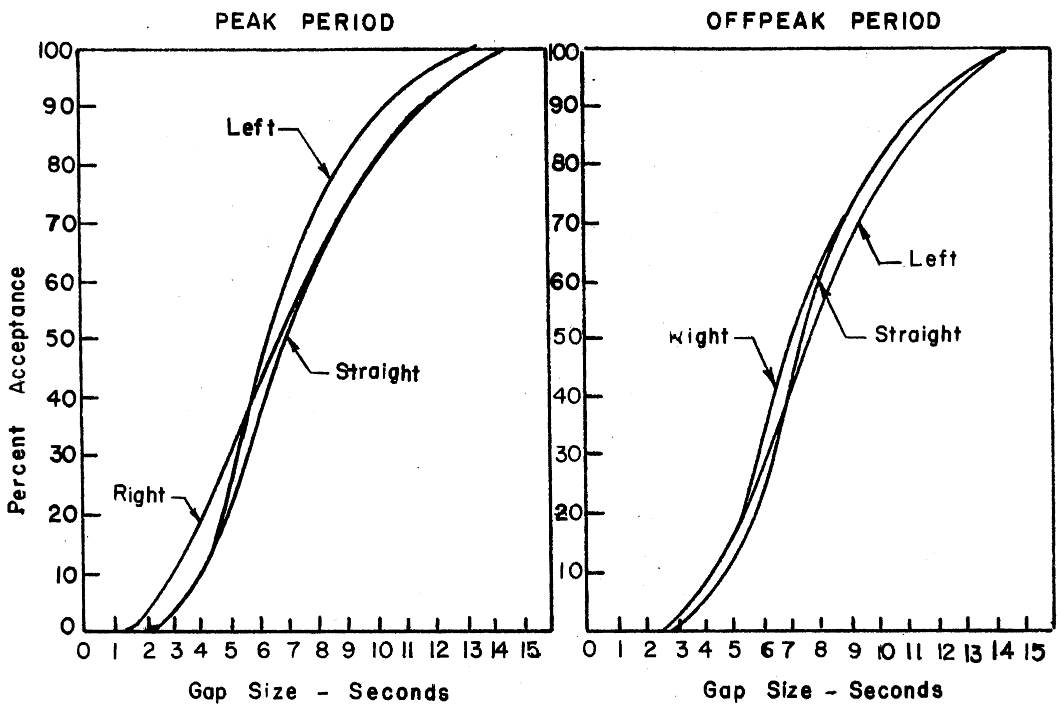


Figure 10. Effect of Direction of Side Street Vehicle Movement on Gap Acceptance Distribution (29)

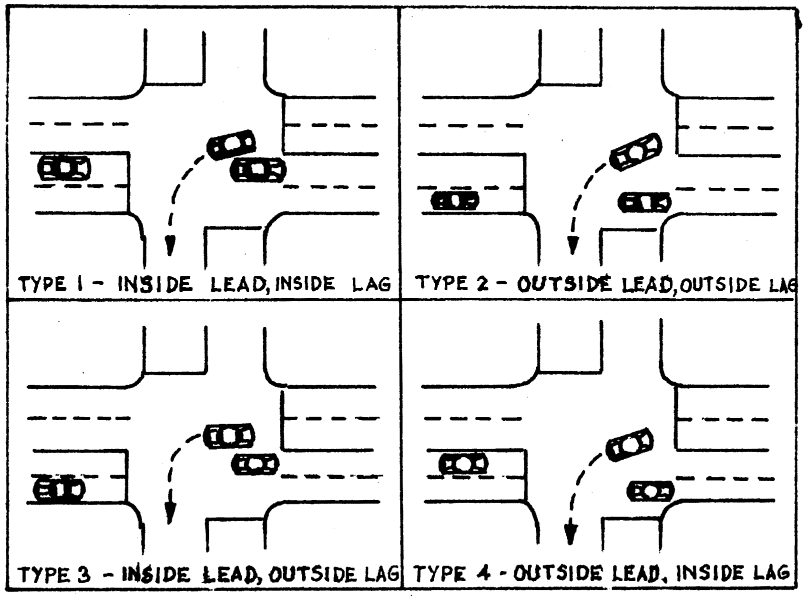


Figure 11. Opposing Traffic Stream Gap Types Confronting Left-Turn Vehicles (30)

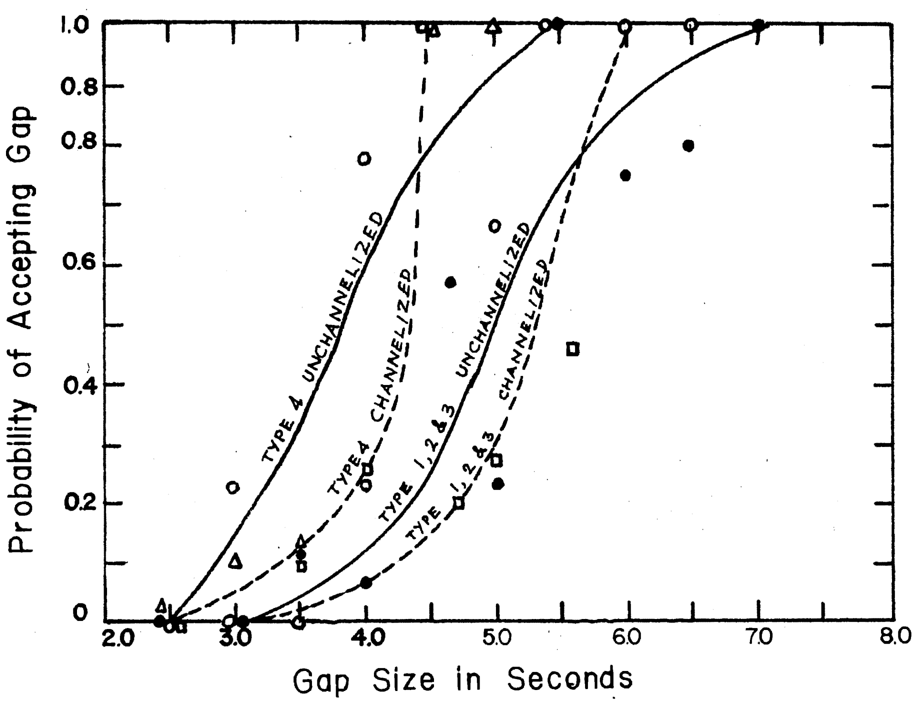


Figure 12. Probability of Left-Turn Vehicle Accepting Gap From Stopped Position at Signalized Four-Way Intersections (30)

CHAPTER III

INTERSECTION FIELD STUDIES

Objectives of the Studies

The prime purpose of this simulation program is to represent the real traffic condition for any street intersection having one or two lane approaches at right angles to each other. Although the computer program may be operated with artificial input traffic data, actual field data, if available, is more authentic and leads to more realistic results. Previous field observation studies have indicated that it is not unusual to find that the data from field observations may be far different than the theoretical traffic distributions appearing in the traffic literature. In any simulation model, the first thing one must do to get correct and accurate results is to supply the right input data. It is therefore important for this model that all traffic characteristics of the intersection under consideration agree with the actual conditions there. In addition to the data collected for input of the program, field observations also supply statistical data such as queue length and delay time distributions, used to check output results.

Most former researchers who have employed traffic intersection simulation (38, 42, 46) have obtained the vehicle delay as their principal "figure-of-merit." Only two have actually compared simulation output with actual field studies. Lewis (4) observed data from actual intersections and tried to compare his simulation results with the actual field delays, but found that it was impossible to correlate conditions. Dart (1) measured delays in field studies at several intersections in Texas. These delays were then compared with delays obtained from simulation studies. He reported that correlation was obtained in roughly one-half of the intersections.

Field observations yield the following data:

1. Beginning of the green signal phase.

2. Length of the cycle.

3. Time each vehicle enters the system.

4. Type of vehicle: passenger car, single unit truck, etc.

5. Speed of those vehicles, passing through the intersection without delay.

6. Whether the vehicle was stopped by the signal.

7. Starting delay after the start of green signal phase, and headway of the following vehicles.

8. Number of stopped vehicles in the queue for each signal cycle.

9. Percent of turning vehicles.

10. Total time each vehicle appears in the system: including the stopped delay, delays during acceleration and deceleration, and the traveling time.

11. Total vehicles in each lane during period being studied.

Study Method Selection

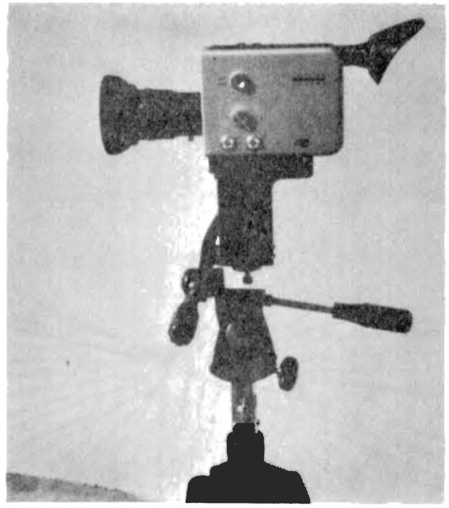
Based on experience from field studies, Dart (1) believed that the most satisfactory and economical study procedure, from both field study and data analysis time standpoints, was the time-lapse photography technique. Cribbins and associates (59), who conducted urban traffic control studies in North Carolina, also found that time-lapse equipment provided the ability to record on film a very precise, quantitative account of specific traffic variables, such as speed, volume, density, headway, merging, and weaving. With limited amount of funds and time for the research, the time-lapse photography technique is considered the most suitable to collect all necessary data. Most of the field data therefore were collected via this procedure.

In addition to the above method, stop watches also can be used to get data from which more precise and accurate results are needed, such as starting delay and traveling time across the intersection proper. While the time-lapse photography method with frames exposed at 1 second intervals provides an accuracy of ± 0.5 second, a stop watch, on the other hand, can be read to 0.01 second.

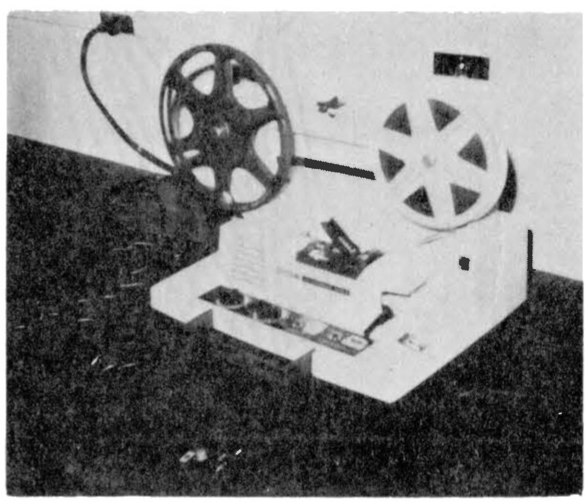
Equipment and Time-Lapse Procedure

A Nizo S-80 Schneider Verigon time-lapse camera was used to collect most data. This camera uses Super-8 film cartridges and is powered by six 1.5 volt batteries. The camera can take automatically exposed pictures at rates of one frame per minute to 54 frames per second. The appearance of the camera is illustrated in Figure 13 (a).

The time-lapse controller dial that is used for setting the filming rate is not accurately calibrated from the factory, so it is left to the operator to check with a stop watch prior to the operation. This is a disadvantage of using the camera in this research. Since it is very difficult to set the film rate exactly as desired, calibration required



(a) Nizo S80 Camera



(b) Kodak Stop-Action Projector

Figure 13. Time-Lapse Photography Equipment

extra time for summarizing the data. Once the framing rate is set, filming will continue without variation.

Kodak Kodachrome II film in Super-8 cartridges was used in all filming. Color film enables the viewers to more readily identify the phase and cycle changes of traffic signals and to recognize the moving vehicles more easily.

Data from the color films were projected manually by means of a Kodak Ektagraphic MFS-8 projector, as illustrated in Figure 13 (b). This projector can be operated at various speeds or in the still mode for a single frame analysis. The latter is the one used in this research. The projector lamp, with average life of 12 hours, is air cooled and film can be viewed indefinitely in the still mode without burning.

During the filming process, the camera was set up on a tower of a church within 500 feet of the intersection being studied. The single frame button on the camera was set at about 1 second intervals. A clock was included in the field of view to establish time of the day and provide a check on the frame interval obtained.

This technique provides time measurements to an accuracy of ± 0.5 second after calibration. One advantage of the method is that the traffic data from the intersection is permanently available and complete analysis of a situation can be obtained by running and rerunning the film through a projector.

Use of Stop-Watches

While observing the field data by means of stop-watches, three observers in a parked car at a corner of the intersection near the traffic lane being studied were used to observe data from individual vehicles. Each observer simultaneously studied different vehicles for the same variable, such as the starting delay of the first vehicles in queue, etc. When the signal phase turned green, a stop-watch was started counting and was stopped when the rear wheel of the first vehicle in that queue crossed the stop line and entered the intersection proper. The time recorded was the starting delay. Similarly, the headway of the following vehicles, according to their positions in the queue, was considered as the interval of time between successive vehicle crossings of the stop line.

The stop lines on each lane of the pavement are also used as reference points in observing the traveling time across the intersection for through, right-turn, and left-turn vehicles. Despite the different lengths of vehicles, the position of the rear wheels was always used to represent the position of that vehicle.

CHAPTER IV

RESULTS OF FIELD STUDIES

The intersection at W. Sixth Avenue and S. Duck Street in Stillwater, Oklahoma was the selected site for field observations. This intersection is a typical one, similar to that shown in Figure 1, with two lanes in each approach and additional left-turn lanes. The field data were observed between 4:30 p.m. and 5:30 p.m. in April, 1975. There is a small percentage of trucks that may increase the delay during this period and were ignored in this study. It was also observed that there is no pedestrian interruption affecting the right-turn vehicles.

The approximate width of each approaching lane is about 11 feet, while the length of the separate left-turn lane is about 100 feet. Parking is prohibited on all approaches. The stop-lines for vehicles are about 5 feet back from the pedestrian crosswalk, but the obscure markings tended to lead first vehicles in the queues to stop at the crosswalk lines.

Because of limited observation sites for taking the time-lapse movies, traffic in each direction was taken on different days. The sites for the camera setting were two church towers about 300 feet away from the intersection. The reference lines, for the entry of the vehicles, were set at different distances from the intersection to best fit the capability of the camera angle. Since there is more traffic on W. Sixth Avenue, these reference lines were set at 300 feet from the stop lines on each approach. They are 100 feet and 175 feet on S. Duck Street for southbound and northbound traffic, respectively.

The fixed-cycle traffic control system has traffic-actuated left turn phases. Alternative cycle lengths are 60 and 80 seconds, with equal time for each street. There is a separate left-turn phase after the through traffic signal, but it is automatically omitted if there is no vehicle on left-turn lane by the time the left-turn phase should turn green.

Starting Delay and Headways

According to the summarized data in Appendix A, the average starting delay of the first vehicles in a queue is observed to be 3.26 seconds for the straight-through vehicles, and 3.58 seconds for the left-turn vehicles. These figures are assumed to be applicable to all approaches since their geometrics are identical.

The headways of the following vehicles are listed:

Position of Vehicles in Queue	Through and Right-Turn Vehicles (Seconds)	Left-Turn Vehicles (Seconds)
2nd	2.55	2.49
3rd	2.39	2.20
4th	2.19	2.07
5th	2.23	1.92

As compared to the critical headway of 1 second, the above figures show that the drivers here are not in a hurry, probably because there are not too many vehicles in queues. The headways are decreased successively because of increasing speed, and believed to be constant after the 5th vehicle. Decreasing headway in left-turn lane is sharper because of the short time phase of signal for these vehicles.

Traveling Time Across the Intersection

One purpose of the study is to observe the actual time required for the vehicles to pass the 55-60 feet width of the intersection proper. The details of results from each approach are shown in Appendix B, from which the average values are shown in Table IV.

TABLE IV

AVERAGE TRAVELING TIME FOR VEHICLES TO CROSS THE INTERSECTION PROPER

Types of Vehicles	Position of Vehicles in Queue					
	Non-Stop	1st	2nd	3rd	4th	5th
Right-turn	1.70	1.92	1.98	1.32	1.65	1.25
Through	2.13	2.80	3.21	2.99	2.86	3.03
Left-turn	2.80	2.80	3.05	3.19	3.05	--

It is apparent from Table IV that there is not much difference for right-turn vehicles in any position in the queue since they have to slow down before making the turn, as is also the case for left-turn vehicles. After starting delay of about 3.26 seconds, the first straight-through vehicle in the queue can pass the intersection more quickly than the following cars because there are not any obstacles in sight. The driver can accelerate his vehicle at will up to the desired speed, while the speeds of the following vehicles are constrained by preceding ones, and may have to decrease speed if the preceding car makes a right turn.

The non-stop vehicle is one that arrives at the intersection during a green signal phase and can go on without stop.

Turning Traffic

Table V summarizes results of field observations of turning traffic in each approaching lane. There are high percentages of turning vehicles in most approaching lanes.

TABLE V

PERCENTAGE OF TURNING VEHICLES

Direction of Traffic	Lanes	Total Vehicles	Right-Turn Vehicles	Left-Turn Vehicles	Percentage
Southbound	outside	175	65	--	37.1
	inside	321	--	119	37.1
Westbound	outside	330	121	--	36.7
	inside	275	--	28	10.2
Northbound	outside	234	28	--	12.0
	inside	334	--	133	39.8
Eastbound	outside	443	116	--	26.2
	inside	413	--	124	30.0

Traveling Time of Vehicles From Reference Lines Until Leaving the System

The traveling time of each individual vehicle, starting when it passes the reference line until it moves past the intersection proper, was observed by analyzing the recorded movies frame by frame. These results are calculated separately for vehicles moving through, making a left turn, or making a right turn. The purpose is to provide field data to compare with the results of the computer simulation program. Differences in traveling time are affected by the characteristics of the vehicles, starting delay, stopping delay, and also the position of vehicle in the stopped queue when the signal phase turns green (Appendix D).

Vehicle Arrival Time*

Arrival time at the reference line for each individual vehicle is recorded by using the single frame movie photographic method of about 1 second per frame. The distributions of the arrival time fit very well with the shifted exponential distribution. This agrees with previous work (2) using this method, with the physical length of cars and minimum following distance by drivers being considered.

As previously mentioned, many traffic engineers in the past have concluded that general traffic flow patterns are likely to follow some type of Poisson distribution which may be generally expressed as:

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*The theory and equations cited in this section are from Reference (2).

$$P(x) = \frac{m^x e^{-m}}{x!} \tag{4.1}$$

where

m = mean of observed data;

x = number of successes;

P(x) = the probability of exactly x successes.

Since the arrival rate of vehicles deals with counting distributions for discrete events (arrivals of vehicles) within a given time interval, the distribution of gaps (time spacing) between vehicles is a continuous variable and is treated by means of interval distributions. Of possible distributions, the best known is the (negative) exponential distribution (2). The value of m in the Poisson distribution is replaced here as follows:

$$m = \frac{Vt}{3600} \tag{4.2}$$

where

V = hourly volume;

t = length of each observation in seconds.

Thus,

$$P(x) = \left(\frac{Vt}{3600}\right)^x e^{-\frac{Vt}{3600}}$$

$$P(o) = e^{-\frac{Vt}{3600}} \tag{4.3}$$

If there are no vehicles in a particular interval of length t, then there will be a gap of at least t seconds between the last previous vehicle and the next vehicle. This means that P(o) is also the probability of a gap equal to or greater than t seconds, which may be expressed as:

$$P(g \geq t) = e^{-\frac{Vt}{3600}} \tag{4.4}$$

where

$$m = \frac{Vt}{3600}$$

is the mean of the arrival (counting) probability distribution.

It may be seen, from this relationship, that the number of gaps greater than any given value will be distributed according to an exponential curve.

Now, if we set

$$m = \frac{t}{T} \tag{4.5}$$

then

T = the mean of the interval (gap) probability distribution.

Thus, the probability of a gap equal to or greater than t may be written:

$$P(g \geq t) = e^{-t/T} \tag{4.6}$$

or in reverse probability as:

$$P(g < t) = 1 - e^{-t/T} \tag{4.7}$$

If we include gaps of size smaller than some value, say τ, this situation can be represented by shifting the exponential distribution by an amount of τ. The equation now becomes

$$P(g < t) = 1 - e^{-(t-\tau)/(T-\tau)} \tag{4.8}$$

The minimum arrival time observed in the field is about 1 second, and thus $\tau = 1$ is used in the above equation to evaluate the interval probability distribution in this study. All results are summarized in Appendix C.

Vehicle Speed

The speeds of the free flowing vehicles, without any interruptions while moving in the system, were observed by playing and replaying the individual frames of the time-lapse movie. The total time of each vehicle, as determined by the number of frames in which it appears in the movie, divided by the distance, results in vehicle speed. Cumulative speed distribution for each approaching lane is shown in Appendix E. Most of the distributions, except the observed speeds of the southbound vehicles, have a median value between 32.5 and 40 ft/sec and have distribution shapes similar to that used in Dart's model (1). The southbound vehicles showed considerably slower speed with the median of about 25 ft/sec, while the design speed for all approaches is 41 ft/sec (28 mi/hr). There is no obvious reason to explain slower speed in this direction.

CHAPTER V

THE SIMULATION MODEL

The purpose of the traffic simulation model is to produce realistic results economically. The movement of traffic in and near a signalized intersection is a very complex operation, which tends to be oversimplified if this operation is represented mathematically. The use of simulation techniques, however, permits the analyst to build any degree of realism that he wishes into the intersection model. However, in developing a simulation model, it is unwise to build a perfect representation of the real intersection. Not only is such perfection difficult and time-consuming to obtain but the resulting model will likely be very inefficient (4). Therefore, a simulation model should be employed which will adequately represent the most important operating characteristics and ignore the unusual or insignificant events.

Since the primary objective of this research is to study the delays caused by traffic congestion and signalization, the simulation model will not represent realistically those characteristics of vehicles and drivers which do not significantly affect the delays. GPSS language is chosen because, in the writer's opinion, its program statements are powerful. In other words, one statement in GPSS can represent many things at the same time. Furthermore, this language provides some built-in statistical parameters which adequately fit the traffic data requirement. It therefore results in simpler and more efficient programs in terms of computer time to real time ratio.

Mode of Representation

There are certain specific block types in GPSS language that are analogous to the pattern of traffic flow in the roadway. The general representations can be briefly described as follows:

The "generate" blocks are used to represent the arrival of vehicles at the entrance to the system. Vehicles are "created" and sent into the system at the precalculated intervals. Many "function" blocks are used to meter the vehicle arrivals, usually in the form of statistical distributions.

Each vehicle is represented by a "transaction" created from the generate blocks. Once leaving the generate block, a transaction will be assigned its own behavior, such as speed, turning maneuver, marked time when entering the system, etc. This transaction will try to move as far as possible without violating physical constraints, similar to a real vehicle on the roadway, until it passes through the intersection and leaves the system. Along each approaching lane to the intersection, a transaction may be interrupted by preceding slower transaction, by the traffic signal, represented by "logic" blocks, and by the queue of stopped transaction which are represented by a "user chain." The traveling times of vehicles having assigned speeds, acceleration and deceleration, turning speeds, etc., are supplied by "variable" blocks according to the immediate condition a vehicle is dealing with. These traveling times are controlled by the use of "advance" blocks. "Group" blocks are used along with the user chains to determine the queue length in each cycle of the signal.

By this means, each transaction in the model will act analogously to the average driver-operated vehicle. It may be noted that in actuality, not all drivers are average, but will make driving decisions based on many individual characteristics and state of mind. However, the computer transactions always act in an identical manner.

The Basic Form of the Model

The simulation model used in this research represents the traffic operation at a single signalized intersection. The model admits one or two approach lanes plus one additional left-turn lane, where desired. The computer program is separated into two parts: the traffic control program and the traffic flow program.

1. Traffic control program. This program simulates the traffic signal in all four directions. For easier handling and adjustment when signal complexity increases, each signal direction is separately represented. This permits the user to trace the signal simulation in one direction in the same way the drivers observe the signal. The control system can employ two-phase or three-phase signals, or may be adjusted to any type of signal that is possible to be installed at the studied intersection.

2. Traffic flow program. This program simulates the vehicular traffic from all approaching lanes (maximum of eight lanes). Traffic is represented for a distance back from each approach stop line. This distance is the length along the approach lane sufficient to permit an entering vehicle to stabilize its behavior before reaching any of the

critical points in the lane. In addition, this length is necessary to provide storage room when demand volumes increase and the queuing has been built up. This section of roadway may be called "zone of influence," since vehicles in the zone are influenced by any intersection congestion. It is bounded by the "reference line" and the "stop line" (see Figure 14).

A vehicle enters the system when it crosses the reference line (or entrance), triggering the travel timer for this vehicle. If this vehicle is not in a platoon, affected by the previous signalized intersection, it may move at its desired constant speed. When there is a preceding slower vehicle, the following one must decelerate and join the platoon-like stream. The arriving vehicle will slow down to join the line when there is a queue of vehicles waiting for the green signal. If there is no queue, the arriving vehicle will check the signal and will enter the intersection at the same speed during a green signal, or decelerate to stop at the stop line during the red phase of the signal, to be the first vehicle in the queue. If the vehicle is making a right turn it will check the acceptable gap for a right turn on red, or it may wait for the next green phase. Once a vehicle has entered the intersection and reached the second stop line, on the leaving lane, it is considered out of the system and no longer has any effect on other vehicles still in the system.

Assigned Intersection System Geometry

The principal geometrics of the intersection system, where separate left-turn lanes are provided, are shown in Figure 14. Key points for

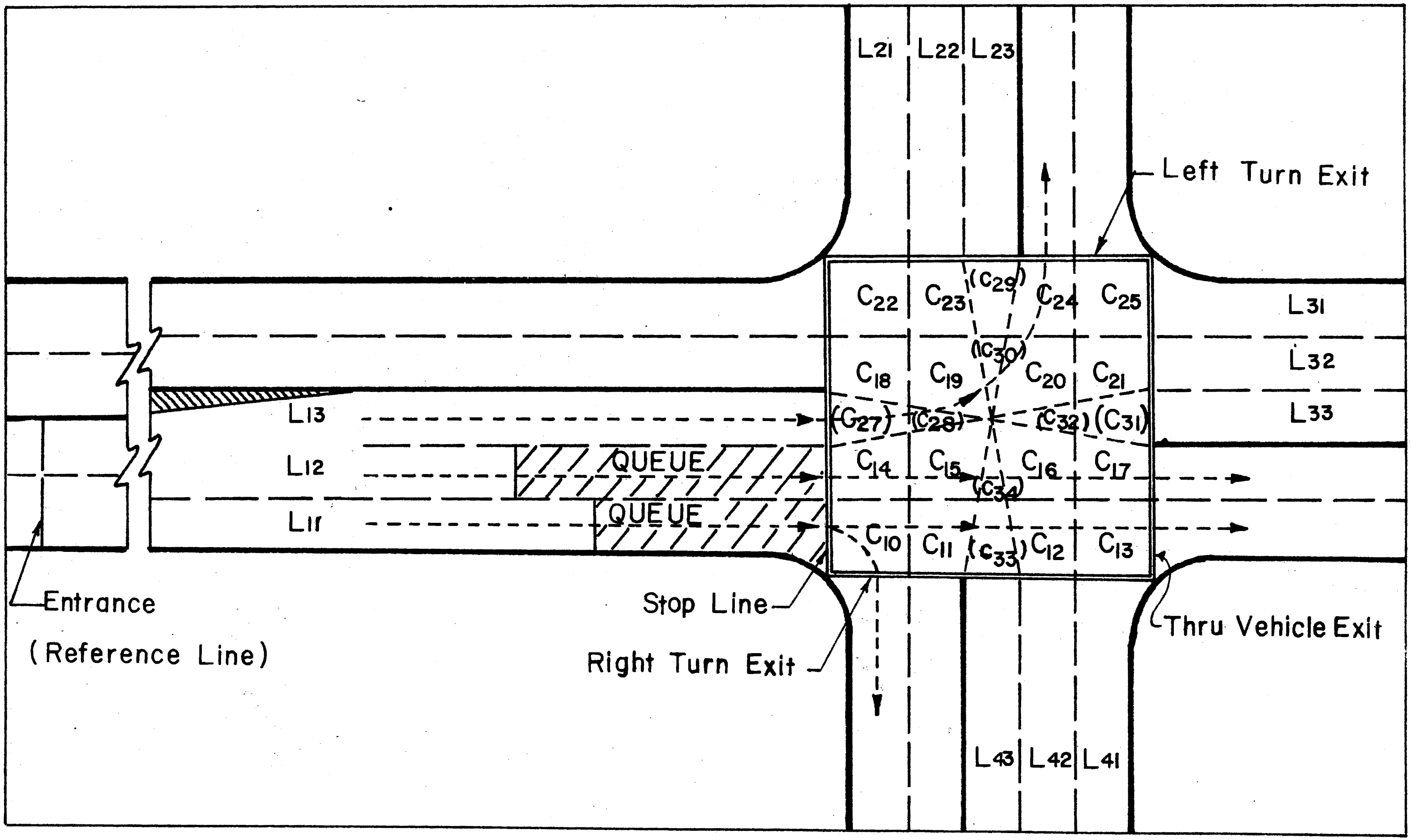

Figure 14. An Intersection Module and Vehicle Paths

approaching vehicles are located by the reference lines and stop lines. The system consists of the two elements:

1. Intersection cells. The intersection is divided into a "checkerboard" arrangement of cells of similar size, with the exception of separate left-turn lanes which are considered as special and extra cells. The boundaries of each of these cells are determined by the region formed by pairs of intersecting lanes in the intersection. In the special case where there is a separate left-turn lane, two additional cells, for example C_{27} and C_{28} in Figure 14, are included and utilized by left-turn vehicles only. For other than left-turning vehicles these two cells are considered as parts of cells in the inner lane: cells C_{14} and C_{15}.

2. Lanes. Each approaching lane is assigned a different number to go with the symbol of the lane, L_{11}, L_{12}, etc., denoting the different lanes. For the case where only one lane exists in any direction, the inner lane is used and vehicles can make the right turn from this lane.

Simulation of Signal Control

Traffic in the system is controlled primarily by the settings of the signal control box dials. Usually, variation in signal settings include:

Signal Cycle Length--total time for a single sequence of red, yellow and green lights.

Splits--percentage of signal cycle length for the red, yellow and green periods.

Offsets--percentage of cycle length for initial synchronization of consecutive traffic signals to maintain an uninterrupted flow.

In the simulator signal, control for each direction of traffic is accomplished by a signal control loop (as shown in Figure 15). An independent program generates a signal "regulator" for each traffic phase in each direction. The regulators circulate in the signal control loop, turning the signals on and off at fixed or calculated intervals. Vehicles arriving or waiting at the intersection must test a red signal associated with the phase; they cannot enter the cells until the signal has turned green. There is one exception: the right turn is permitted on red phase with an acceptable gap. Yellow signal time is included in the total green time.

Simulation of Vehicles

Inter-arrival times for vehicles entering the system are generated by a shifted exponential distribution (based on Poisson arrivals). The assumption of Poisson arrivals has been found to be reasonably representative of actual traffic conditions when traffic is light and when the effect of a previous signal is negligible. In heavy traffic and/or where a prior signal is of considerable consequence (vehicles usually arrive in platoons at an almost uniform rate), observed arrival distributions may be substituted for the assumed distribution. Different "function" blocks were formed to represent the inter-arrival times for each lane of traffic.

For realistic simulation of the traffic stream, traffic in each lane was programmed independently. Vehicles in different lanes have different paths and appear in different places in the program. This allows the standard statistical outputs to be examined and traced individually for each lane. Characteristics of vehicles on inside lane and

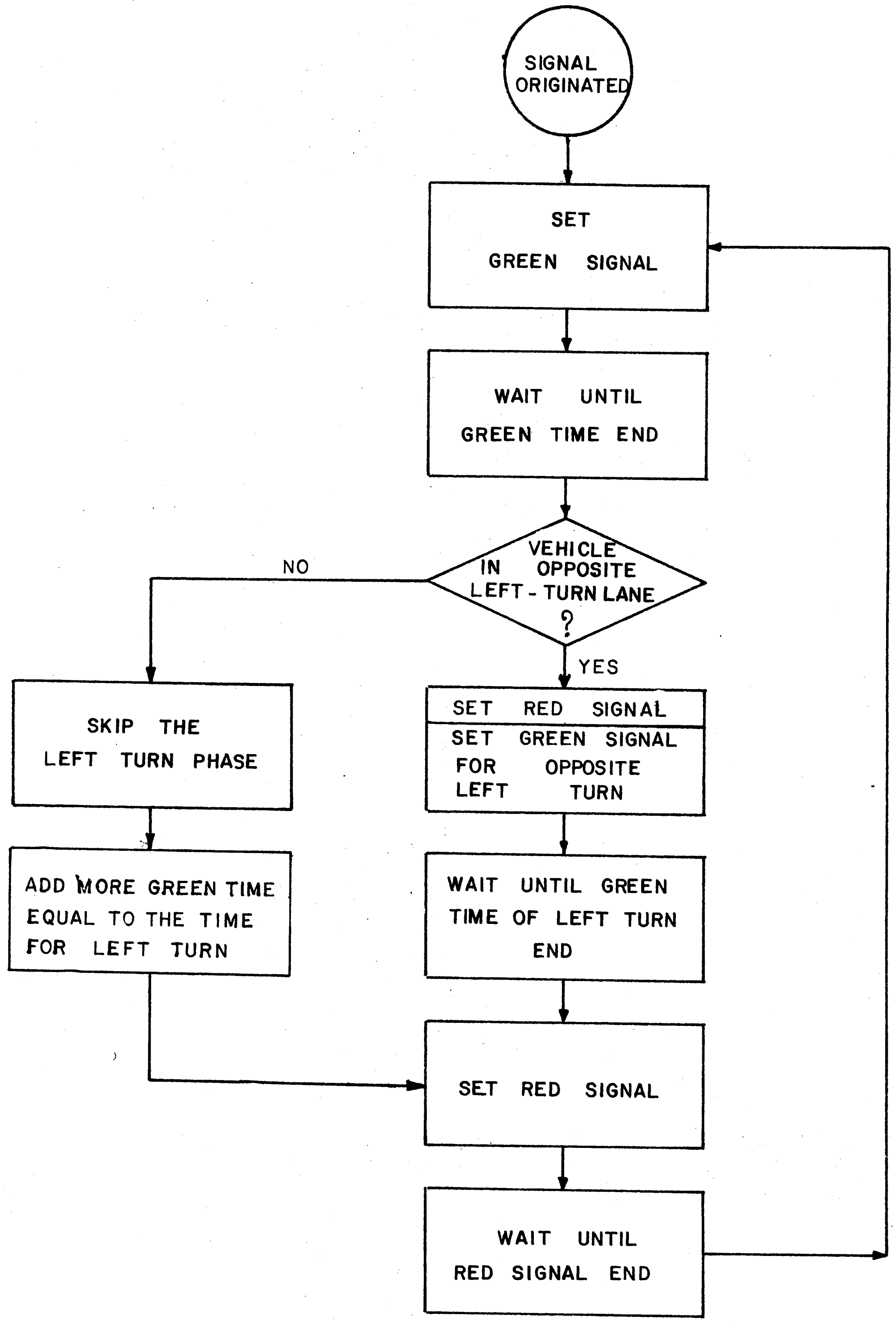

Figure 15. Signal Control Loop (3 Phase Cycle)

outside lane were programmed a little differently, as shown by flow charts in Figures 16 and 17. The difference concerns the turning maneuver and checking for the left-turn lane.

Upon entering the network, a number of operating characteristics are assigned to individual vehicles in the system, velocity from observed distribution is assigned to the vehicle until the vehicle's free flow is inhibited by a preceding vehicle or signal light. If the vehicle overtakes a preceding one, it assumes the slower vehicle's speed. Changing lanes near the intersection rarely occurs, except when blocked by a vehicle waiting for a left-turn opportunity. This will be ignored in the program. The decision of the driver to go straight through or make a turn is also assigned to the vehicles.

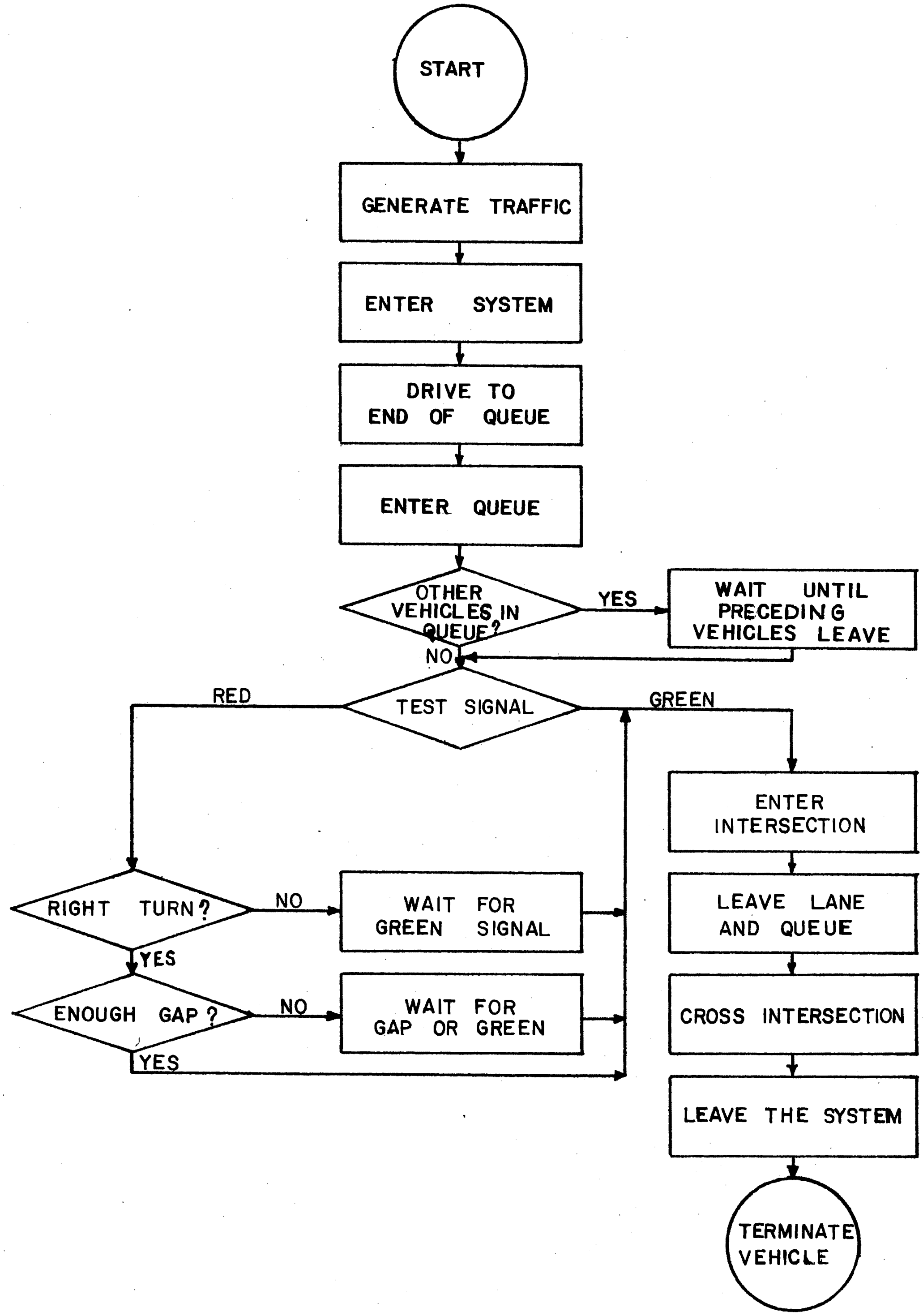

Figure 16. Outside Lane Vehicle Flow

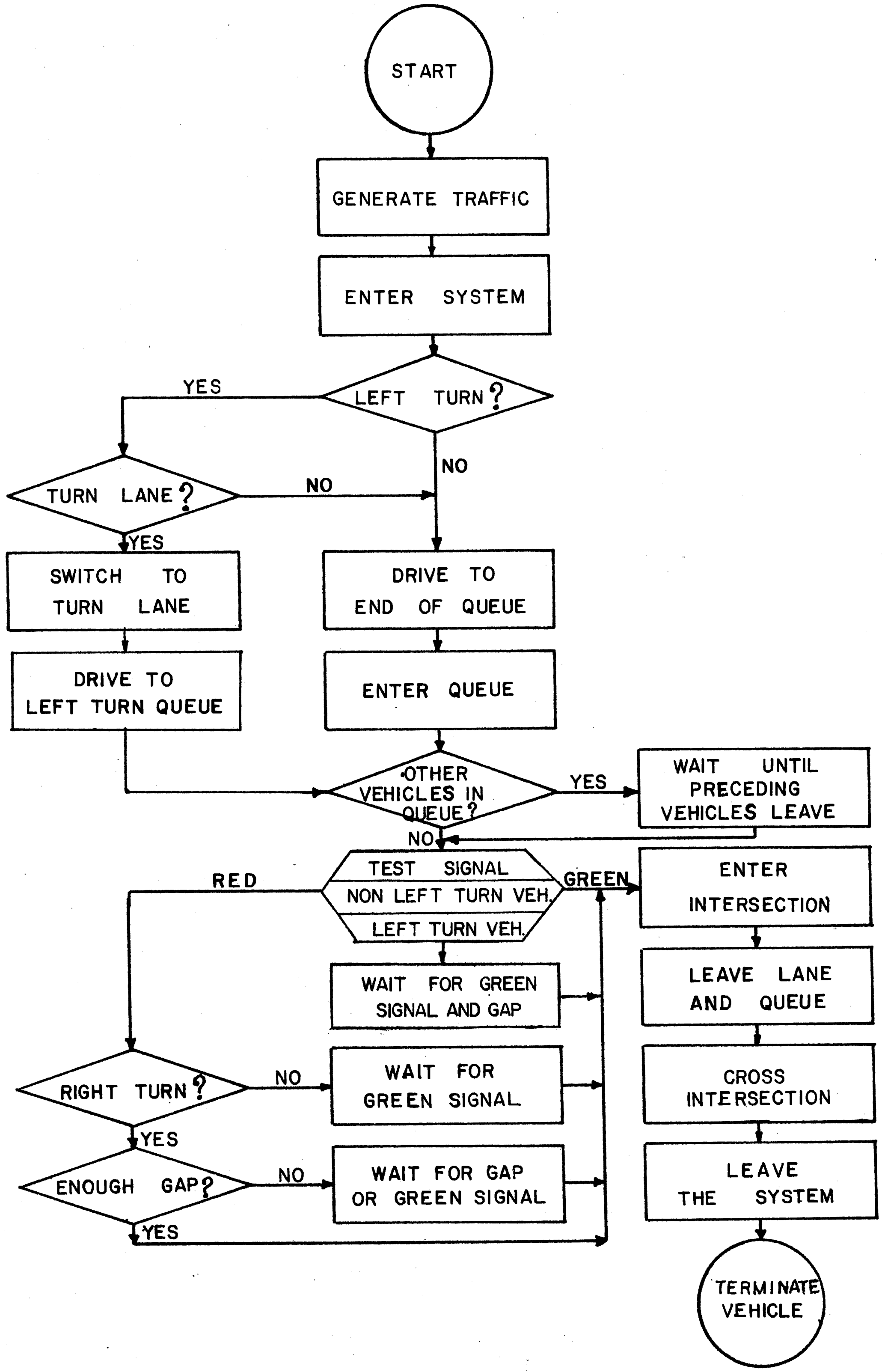

Figure 17. Inside or Single Lane Traffic Flow

CHAPTER VI

SIMULATION RESULTS

The principal purpose of the simulation model is to provide output consisting of mean queue lengths and mean traveling times for traffic from each approaching lane of the studied intersection, using any particular set of input data. It requires several sets of results for comparison and evaluation in order to determine the best possible choice of signal cycle phase lengths. The first run usually employs actual signal times currently in use at the intersection. The output thus shows the efficiency of present traffic situations and control signals. Succeeding sets of data, in the direction of possibly decreasing the mean traveling times and/or mean queue lengths, may be programmed to run separately or, by the use of reset procedures, may provide a series of solutions.

The simulation model also tabulates data, for checking purposes, on the arrival time and speed distributions of traffic from each approaching lane. Likewise, the accuracy of the program in representing turning data may be traced from the block statistics, supplied as the standard output for GPSS (as demonstrated in Appendix F).

Typical Simulation Output

All the computer outputs are in the form of statistical distributions which include the mean values and standard deviations, separately determined for each lane of traffic. The results include the distributions of queue length, traveling time, speed, and arrival times of vehicles. Appendices G through L contain sample computer output sheets. Also included are examples of the built-in standard statistical output associated with clock and block entities as shown in Appendix F. These are useful for the purposes of tracing each step of the program and also checking certain parameters, such as the percentage of turning vehicles.

Validation of Simulation Model

In writing this computer program, the writer tried to formulate a model which would reflect all of the important traffic parameters of a signalized intersection, as well as take cognizance of those minor parameters which may not significantly affect the final results. Previous investigators using nominal input (e.g., 10 percent turning volume) for their models (1, 4), have found that their results were hardly comparable to the field observations. This was because their models permitted only a few input data values, and depended on assumptions to fill in the remaining parameters. The intersection traffic was therefore not adequately represented. The writer believes that any specific intersection will have unique conditions and characteristics of its own. The model used to evaluate a specific intersection should be able to represent these characteristics.

For reality in evaluating the selected intersection, all characteristics and statistics collected from the field were used as the input of the program. These values appeared in the INITIAL, VARIABLE, and FUNCTION cards in the first part of the source program.

Check of Random Distributions

The first part of the checking procedure deals with the goodness and accuracy of random numbers generated by the GENERATE blocks in the model. Since these numbers are important to represent the correct statistical input, different sets of random numbers may have to be used for groups of data.

Appendix C contains comparisons between computer generated and observed arrival-time distributions. Results show that the distributions from field observations, theoretical shifted-exponential distribution, and the values supplied by the model all agree very well.

In Appendix E, the distributions of vehicle speeds, observed from the field, are compared with those generated in the model. Even though in most lanes the generated values in the model are somewhat higher than those observed, they may be considered as being in good agreement.

Percentage of turning vehicles is the last test of the accuracy that the model GENERATE blocks can supply in accordance with a set of input data. Table VI shows the comparison of percentage of turning traffic between the observed values which are used as input data, and the values generated in the model. The greatest deviation is +4.8% for the southbound, right-turn traffic, while the smallest deviation is -0.4% for the westbound, right-turn traffic. The comparisons here also indicate reasonable agreement.

Tests of Output Results

The queue length and traveling time are the most important outputs that derive from the model. These values were thus compared with field

TABLE VI

COMPARISON OF TURNING VEHICLES BETWEEN FIELD DATA AND THE SIMULATED VALUES

Direction	Type of Turn	Percent Turning Vehicles Observed	Percent Turning Vehicles Simulated	Differences
Southbound	Right-turn	37	41.8	+4.8
	Left-turn	37	35.0	-2.0
Westbound	Right-turn	36	35.6	-0.4
	Left-turn	10	9.0	-1.0
Northbound	Right-turn	12	15.5	+3.5
	Left-turn	40	37.6	-2.4
Eastbound	Right-turn	26	27.3	+1.3
	Left-turn	30	27.5	-2.5

observations under similar conditions. For the northbound traffic, the comparison was made for a signal cycle of 80 seconds, while other approaches were compared when the signal cycle was 60 seconds.

Table VII gives the values of average queue length and also the standard deviation for each traffic approach. It is noted that, for each approach, the simulated queue length is a little smaller than that observed in the field. Because of light traffic at the selected intersection and the resulting short queues, the percentage differences are magnified while the absolute differences are very small.

Like the queue lengths, the mean traveling times as shown in Table VIII generally indicated smaller simulation values than the observed values, the differences averaging 10 percent. Standard deviations were in somewhat closer agreement. This amount of difference between actual and simulated values is not unexpected when dealing with phenomena as variable as traffic.

TABLE VII

COMPARISON OF MEAN QUEUE LENGTHS

Direction	Lanes	Mean Queue Length		Differences	Standard Deviation		Differences
		Observed	Simulated		Observed	Simulated	
Southbound[1]	Outside Inside Left-turn	1.84 2.40 2.34	1.81 2.00 1.80	-0.03 -0.40 -0.54	1.23 1.24 1.43	1.29 1.59 1.45	+0.06 +0.35 +0.02
Westbound[1]	Outside Inside Left-turn	4.37 2.72 0.46	3.41 2.44 0.40	-0.96 -0.28 -0.06	2.13 1.61 0.71	2.00 1.38 0.64	-0.13 -0.23 -0.07
Northbound[2]	Outside Inside Left-turn	2.94 2.76 2.16	2.70 2.41 1.98	-0.24 -0.35 -0.18	1.63 1.40 1.43	1.41 1.51 1.68	-0.21 +0.11 +0.25
Eastbound[1]	Outside Inside Left-turn	3.14 1.79 1.83	2.81 2.12 1.37	-0.33 +0.33 -0.46	2.20 1.33 1.27	1.64 1.50 1.08	-0.56 +0.17 -0.19

[1] Signal cycle = 60 seconds

[2] Signal cycle = 80 seconds

TABLE VIII

COMPARISON OF TRAVELING TIME

Direction	Lane	Mean Travelling Time (seconds)		Differences	Standard Deviation		Differences
		Observed	Simulated		Observed	Simulated	
Southbound[1]	Outside (Th)[3]	22.50	19.96	-2.54	16.17	12.85	-3.32
	Outside (R)[4]	22.32	20.01	-2.31	17.00	13.09	-3.91
	Inside	22.59	22.21	-0.28	15.73	14.56	-1.17
	Left-turn	40.70	37.50	-3.20	25.44	17.31	-8.13
Westbound[1]	Outside (Th)	34.69	27.25	-7.44	20.67	13.73	-6.94
	Outside (R)	30.44	28.60	-1.84	18.28	13.28	-5.00
	Inside	24.51	26.80	+2.29	14.53	13.69	-0.84
	Left-turn	37.17	39.56	+2.39	15.94	21.61	+5.67
Northbound[2]	Outside (Th)	31.95	31.00	-0.95	21.07	19.23	-1.84
	Outside (R)	35.00	28.41	-6.59	21.87	21.16	-0.71
	Inside	33.48	28.61	-4.87	22.02	19.03	-2.99
	Left-turn	37.94	42.74	+4.80	22.82	22.02	-0.80
Eastbound[1]	Outside (Th)	26.40	24.02	-2.38	15.35	12.49	-2.86
	Outside (R)	26.31	22.66	-3.65	14.24	11.76	-2.48
	Inside	22.06	24.76	+2.70	14.66	13.39	+1.27
	Left-turn	40.72	38.70	-2.02	21.62	16.74	-4.88

[1] Signal cycle = 60 seconds
[2] Signal cycle = 80 seconds
[3] Straight through vehicles
[4] Right-turning vehicles

CHAPTER VII

INTERSECTION ANALYSIS

The purpose of this chapter is to consider the selection of the most effective traffic control system providing minimum delay for vehicles entering the intersection. Measures to be considered may include changing of traffic signal phasing as well as proposed additional lanes for left-turn and right-turn traffic, wherever it seems to be desirable and applicable. The studied intersection already has a separate left-turn lane on all four approaches; this type of geometric development is therefore not a possibility. The primary consideration here is the intersection control signal.

As previously mentioned, the first trial computer run is usually based on the constraints of the present signal control system. The results may be used as an indicator of how effective the signal phasing is: whether the proportion of total green time allotted to an approach conforms to its ratio of volume, whether the turning vehicles can continue without too much delay, etc. If the results indicate that the signal phasing should be rearranged to be more appropriate, then changes can be made and the results compared.

In the trial computer runs to observe the effects of signal cycle changes, the entering traffic behavior and characteristics are maintained exactly the same for each trial run. Only the signal phasing and cycle lengths are changed.

It should be noted that the yellow signal phase is combined with the green, resulting in what has been termed an "effective green" time. This procedure not only simplifies calculations somewhat, but also corresponds to observed driver behavior, and has been used similarly by other investigators.

The parameters used to evaluate and to compare intersection efficiencies are the mean queue length and mean traveling time. Traveling time is chosen instead of delay time because, in the opinion of the writer, it has broader application when comparisons are made involving vehicles traveling at different speeds.

Comparison of Traffic Signal Phasing

There are six alternatives of signal phasing considered in this analysis. They may be classified as follows:*

1. A 60-second cycle with 20 seconds of green for through traffic and 10 seconds (optional**) for left-turn phasing on each approach (20-10, 20-10). This is the present signal phasing at this intersection, except for the period from 4:00 p.m. to 6:00 p.m., for which the signal changes to an 80-second cycle.

2. An 80-second cycle with phasing of 24 seconds green for through traffic and 18 seconds (optional) for left-turn on northbound and southbound, while phasing for eastbound and westbound traffic is 22 seconds

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*Yellow phase time is considered an extension of green in the following.

**When there is no left-turn demand, opposing through traffic retains the green signal for the additional indicated amount of time.

and 16 seconds for through and left-turn traffic, respectively (24-18, 22-16). This is the typical signal phasing being operated during the period from 4:00 p.m. to 6:00 p.m. on weekdays.

3. A 70-second cycle with phasing of 25 seconds and 10 seconds for all approaches (25-10, 25-10).

4. A 50-second cycle with phasing of 15 seconds and 10 seconds for all approaches (15-10, 15-10).

5. An 80-second cycle with phasing of 25 seconds and 15 seconds for all approaches (25-15, 25-10).

6. A 60-second cycle with phasing times proportioned approximately to the volumes of traffic on each approach. The green times are 17 seconds and 10 seconds for northbound and southbound, 25 seconds and 8 seconds for eastbound and westbound traffic.

Table IX shows the comparison of mean queue values for the above alternatives. The results indicate that the 50-second cycle (No. 4) provides the smallest queue for northbound and southbound traffic, while the 60-second cycle (No. 6) is the best for eastbound and westbound traffic (except for the left-turn traffic eastbound).

The results indicate the same conclusion when considering the mean traveling times, in Table X. Therefore, either the 50-second cycle (No. 4) or 60-second cycle (No. 6) should be the choice for this intersection.

It should be noted, however, that the left-turn phase of 8 seconds for eastbound and westbound traffic (60-second cycle, No. 6) provides insufficient time for the left-turners of eastbound traffic (30% left-turn). Both east and westbound left-turners show increased travel time over similar traffic in trial No. 4, because a phasing of 8 seconds

TABLE IX

MEAN QUEUE LENGTH ANALYSIS

Direction	Lanes	Mean Queue length (Vehicles/cycle)					
		60 sec.[1]	80 sec.[2]	70 sec.[3]	50 sec.[4]	80 sec.[5]	60 sec.[6]
Southbound	Outside	1.81	2.79	2.33	1.50	2.82	2.03
	Inside	2.00	2.90	2.51	1.77	2.91	2.15
	Left-turn	1.80	1.86	2.06	1.39	2.02	1.73
Westbound	Outside	3.41	5.59	3.62	3.14	4.95	2.86
	Inside	2.44	3.63	2.58	2.12	3.39	2.17
	Left-turn	0.40	0.51	0.47	0.33	0.51	0.31
Northbound	Outside	1.91	2.70	2.02	1.35	2.68	2.05
	Inside	1.62	2.41	1.98	1.42	2.43	1.78
	Left-turn	1.53	1.98	1.94	1.33	2.00	1.58
Eastbound	Outside	2.81	4.75	3.56	2.56	4.36	1.72
	Inside	2.12	3.16	2.66	1.73	2.91	1.18
	Left-turn	1.37	1.80	1.65	1.15	1.78	1.42

[1] 60 sec. - (20-10*, 20-10**).
[2] 80 sec. - (24-18, 22-16).
[3] 70 sec. - (25-10, 25-10).
[4] 50 sec. - (15-10, 15-10).
[5] 80 sec. - (25-15, 25-15).
[6] 60 sec. - (17-10, 25-8).

* Signal phases for northbound and southbound traffic.
** Signal phases for eastbound and westbound traffic.

TABLE X

MEAN TRAVELING TIME ANALYSIS

Direction	Lanes	Mean Travelling Times (sec.)					
		60 sec.[1]	80 sec.[2]	70 sec.[3]	50 sec.[4]	80 sec.[5]	60 sec.[6]
Southbound (98 ft.)***	Outside(Th)	19.96	26.84	25.31	18.19	26.98	21.69
	Outside(R)	20.01	25.36	22.00	18.37	25.14	22.20
	Inside	22.21	27.59	22.87	20.22	27.87	24.37
	Left-turn	37.50	36.87	40.99	28.31	40.50	34.94
Westbound (300 ft.)	Outside(Th)	27.25	38.71	27.24	26.98	33.48	22.70
	Outside(R)	28.60	42.01	26.73	27.98	34.46	22.88
	Inside	26.80	35.09	27.31	25.84	31.89	22.80
	Left-turn	39.56	45.52	45.56	38.68	45.16	36.80
Northbound (198 ft.)	Outside(Th)	22.10	31.00	23.01	19.50	29.47	24.12
	Outside(R)	23.93	28.41	20.00	19.93	31.64	27.28
	Inside	21.45	28.61	22.91	20.55	28.02	23.81
	Left-turn	35.96	42.74	45.16	33.90	43.85	36.92
Eastbound (300 ft.)	Outside(Th)	24.02	31.92	25.15	23.11	28.63	18.24
	Outside(R)	22.66	31.93	26.53	22.52	29.68	15.61
	Inside	24.76	31.26	26.56	22.23	28.41	16.80
	Left-turn	38.70	43.23	43.83	32.81	43.68	40.30

[1] 60 sec. - (20-10*, 20-10**).
[2] 80 sec. - (24-18, 22-16).
[3] 70 sec. - (25-10, 25-10).
[4] 50 sec. - (15-10, 15-10).
[5] 80 sec. - (25-15, 25-15).
[6] 60 sec. - (17-10, 25-8).

* Signal phases for northbound and southbound traffic.

** Signal phases for eastbound and westbound traffic.

*** Distances between reference line and stop line on each approach.

allows only two stopped vehicles to pass through, in comparison with three vehicles in a 10-second phase. The third vehicle in the 8-second phase must wait for the next signal cycle.

Some traffic engineers may prefer to have the same amount of delay for vehicles in each approach. Delay times may be computed by subtracting from traveling times the amount of travel time that would be required for an undelayed vehicle. By this measure the 50-second cycle (No. 4) would be the best of the six alternatives.

Other Considerations in Intersection Analysis

There are several parameters which may be used to evaluate the intersection efficiency in simulation. The term "load factor"* has lost its popularity and now is rarely mentioned. The new terms introduced to this field are "queue length," "mean delay," and "total delays," etc. These parameters are perhaps not as significant for a single approach as they are when there are several approaches to be compared, especially when they have different traffic volumes.

Delay may be the most preferable and understandable parameter. Mean delay is generally used. This parameter carries considerable meaning as far as delay of an individual vehicle is concerned. However, in some cases it may not be suitable when evaluating the entire intersection system, especially if traffic volume from the cross street is much different than the main street. In this case a moderate mean delay may

*The ratio of the number of green phases fully utilized by traffic to the total number of cycles.

result in a large total delay for the street with heavy traffic. For this situation the total delay, either the total delay of each street or total delay of the intersection, may be more appropriate. Furthermore, equal values of delay time will be more meaningful for a faster vehicle than a slower one. The true efficiency of an intersection may thus be obscured if the measure of efficiency is not carefully chosen.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This research has been concerned with the utilization of the GPSS language in traffic simulation. It has shown that, by proper programming, GPSS may be one of the most advantageous languages in simulation of dynamic traffic characteristics, since it provides easier and shorter programming efforts, as well as more understandable concepts which can be followed without much difficulty. The model itself provides built-in useful statistical supplementary data which results in shorter programs, less memory storage required, and less computer time.

Based on this simulation, the following conclusions should be briefly mentioned.

1. Field observation studies of traffic characteristics and statistics are usually difficult and time consuming. A time-lapse photography technique with a one-second exposure interval seems to be a most satisfactory and economical method for observing the inter-arrival times, speeds, and turning percentages of vehicles approaching an intersection. The stop watch, on the other hand, is still useful for more precise parameters. However, the new electrical timer, such as the one built into the Hewlett-Packard calculator HP-55, should be introduced to this

field, since it has the capability of 10 stop-watches operating simultaneously.

2. The simulation results of the selected intersection agree well with the field observations. Even though they are based on only one intersection, the different signal cycles observed provide numerous checks of model validity. By comparing results from different sets of data, a traffic engineer can choose the most effective alternate for a particular intersection.

Recommendations

The simulation model, in its present form, is capable of evaluating other types of intersections than the four-lane street crossing a four-lane street selected for this study. The following are some of commonly found intersections which would be worthy candidates for investigation in order to extend the scope of application of this model:

1. An intersection of a four-lane street crossing a four-lane street without separate turning lanes.

2. An intersection of a four-lane major street crossing a two-lane minor street.

3. An intersection of a two-lane street crossing a two-lane street.

In addition to the above traffic signal controlled intersections, the computer program could also be modified to evaluate intersections having two-way or four-way stop-sign control.

As far as the concepts of developing a computer simulation is concerned, the most important single parameter affecting the delay time is the inter-arrival time between each approaching vehicle. This is unfortunate, because each individual intersection has a different pattern of inter-arrival times. Some of them exemplify platoon behavior, especially when strongly influenced by a preceding signalized intersection. Some more nearly represent random behavior, for which the shifted-exponential distribution is appropriate. Most consist of some combination of the two, making the simulation accuracy dependent on collected inter-arrival field data. There is a great need for better understanding of dynamic traffic elements to achieve more adequate mathematical models of intersections.

A SELECTED BIBLIOGRAPHY

(1) Dart, Olin Kenneth, Jr. "Development of Factual Warrants for Left-Turn Channelization Through Digital Computer Simulation." (Unpub. Ph.D. dissertation, Texas A&M University, 1966.)

(2) Gerlough, Daniel L. and Frank C. Barnes. Poisson and Other Distributions in Traffic. Saugatuck, Conn.: Eno Foundation for Transportation, 1971.

(3) Institute of Traffic Engineers. Traffic Engineering Handbook (3rd ed.). Englewood Cliffs, N.J.: Prentice-Hall, Inc., 1965.

(4) Lewis, Russell M. "The Simulation of Traffic Flow to Obtain Volume Warrants for Intersection Control." Joint Highway Research Project No. 23. Lafayette, Ind.: Purdue University, September, 1962.

(5) Greenshields, B. D., D. Schapiro, and E. L. Erickson. Traffic Performance at Urban Street Intersection. Technical Report No. 1. New Haven, Conn.: Bureau of Highway Traffic, Yale University, 1947.

(6) Wilson, Ernest E. "Deceleration Distances for High Speed Vehicles." Highway Research Board, Proceedings, Vol. 20 (1940), pp. 393-398.

(7) Hammond, Harold F. "Report of Committee on Safe Approach Speeds at Intersections." Highway Research Board, Proceedings, Vol. 20 (1940), p. 657.

(8) Chandler, R. E., R. Herman, and E. W. Montroll. "Traffic Dynamics: Studies in Car Following." Operations Research, Vol. 6, No. 2 (1958), pp. 165-184.

(9) Herman, R., D. Gazis, and R. B. Potts. "Car-Following Theory of Steady State Traffic Flow." Operations Research , Vol. 7 No. 4 (1959), pp. 499-505.

(10) Drew, Donald R. "A Study of Freeway Traffic Congestion." (Unpub. Ph.D. dissertation, Texas A&M University, 1964.)

(11) Davies, G. W. et al. "A Generalized Street Network Simulation Model." Transportation Research Record, No. 509 (1974), pp. 16-28.

(12) Bellis, W. R. "Capacity of Traffic Signals and Traffic Signal Timing." Bulletin 271. Washington, D.C.: Highway Research Board, 1960, pp. 45-67.

(13) Strickland, R. I. "Traffic Operation at Vehicle Tunnels." Highway Research Board, Proceedings, Vol. 33 (1954), p. 398.

(14) Cysewski, G. R. "Urban Intersectional Right Turning Movements." Traffic Engineering, Vol. 20, No. 1 (Oct., 1949), pp. 29-31, 34-37.

(15) Rice, P. W. "Effectiveness of Lane Markings on Urban Turning Movements." Traffic Engineering, Vol. 20, No. 10 (July, 1950), pp. 394-397.

(16) George, L. E. "Characteristics of Left-Turning Passenger Vehicles." Highway Research Board, Proceedings, Vol. 31 (1952), pp. 374-385.

(17) Berry, Donald S. and P. K. Gandhi. "Headway Approach to Intersection Capacity." Paper presented to Committee on Highway Capacity, Highway Research Board, Washington, D.C., January, 1973.

(18) Gazis, D., R. Herman, and Alexei Maradudin. "The Problem of the Amber Signal Light in Traffic Flow." Traffic Engineering, Vol. 30, No. 10 (July, 1960), pp. 19-26, 53.

(19) Pignataro, Louis J. Traffic Engineering--Theory and Practice. Englewood Cliffs, N.J.: Prentice-Hall, Inc., 1973.

(20) Bartle, R. M., V. Skoro, and D. L. Gerlough. "Starting Delay and Time Spacing of Vehicles Entering a Signalized Intersection." Bulletin 112. Washington, D.C.: Highway Research Board, 1956, pp. 33-41.

(21) Capelle, Donald G. and Charles Pinnell. "Capacity Study of Signalized Diamond Interchanges." Bulletin 291. Washington, D.C.: Highway Research Board, 1961, pp. 1-25.

(22) Behnam, Jahanbakhsh. "Gap Acceptance as a Criterion for Left Turn Phasing." Traffic Engineering, Vol. 42, No. 9 (Jan., 1972), pp. 40-42.

(23) Gerlough, D. L., and D. G. Campelle. "An Introduction to Traffic Flow Theory." Special Report 79. Washington, D.C.: Highway Research Board, 1964, pp. 51-98.

(24) Kaiser, Fred J., Jr. "Left Turn Gap Acceptance at an Unsignalized Intersection." (Student thesis manuscript, Bureau of Highway Traffic, Yale University, May, 1951.)

(25) Noblitt, Jack L. "A Study of Gaps Associated With a Left-Turning Truck." (Student thesis manuscript, Yale University, May, 1959.)

(26) Kell, James H. Data Obtained by the Institute of Traffic and Transportation Engineering. Berkeley: Univ. of California, 1961.

(27) Solberg, Per and J. C. Oppenlander. "Lag and Gap Acceptances at Stop-Controlled Intersections." Highway Research Record, No. 118 (1966), pp. 48-67.

(28) Weiss, G. H. and A. A. Maradudin. "Some Problems in Traffic Delay." Operational Research, Vol. 10 (Jan., Feb., 1962), pp. 74-104.

(29) Wagner, Frederick A., Jr. "An Evaluation of Fundamental Driver Decisions and Reactions at an Intersection." Highway Research Record, No. 118 (1966), pp. 68-84.

(30) Dart, Olin K., Jr. "Left-Turn Characteristics at Signalized Intersections on Four-Lane Arterial Streets." Highway Research Record, No. 230 (1968), pp. 45-59.

(31) Gerlough, D. L. "Simulation of Traffic Flow." Special Report No. 79. Washington, D.C.: Highway Research Board, 1964, pp. 97-118.

(32) Gerlough, D. L. "Traffic Inputs for Simulation on a Digital Computer." Highway Research Board, Proceedings, Vol. 38 (1959), pp. 480-492.

(33) Rumsey, A. F. and M. G. Hartley. "Simulation of a Pair of Intersections." Traffic Engineering and Control, Vol. 13, Nos. 11-12 (April-May, 1972), pp. 522-525.

(34) Dawson, R. F. and L. A. Chimini. "The Hyperlang Probability Distribution--A Generalized Traffic Headway Model." Highway Research Record, No. 230 (1968), pp. 1-14.

(35) Schuhl, Andre. "The Probability Theory Applied to Distribution of Vehicles on Two-Lane Highways." Poisson and Traffic. Saugatuck, Conn.: The Eno Foundation for Highway Traffic Control, 1955, pp. 59-75.

(36) Trautman, D. L. et al. "Analysis and Simulation of Vehicular Traffic Flow." Research Report No. 20. Los Angeles: Institute of Transportation and Traffic Engineering, Univ. of California, Dec., 1954.

(37) Gerlough, D. L. "Simulation of Freeway Traffic by an Electronic Computer." Highway Research Board, Proceedings, Vol. 35 (1956), pp. 543-547.

(38) Goode, H. H., C. H. Pollmar, and J. B. Wright. "The Use of a Digital Computer to Model a Signalized Intersection." Highway Research Board, Proceedings, Vol. 35 (1956), pp. 548-557.

(39) Wong, S. Y. "Traffic Simulation With a Digital Computer." Proceedings of the Western Joint Computer Conference, San Francisco, Calif., Feb., 1956, pp. 92-94.

(40) Gerlough, D. L. "Traffic Inputs for Simulation on a Digital Computer." Highway Research Board, Proceedings, Vol. 38 (1959), pp. 480-492.

(41) Lewis, R. M. "The Simulation of Vehicular Traffic at an Intersection on a Digital Electronic Computer." (Unpub. thesis, Rensselaer Polytechnic Institute, Troy, New York, May, 1959.)

(42) Benhard, F. G. "Simualtion of a Traffic Intersection on a Digital Computer." (Unpub. thesis, Univ. of California, Los Angeles, June, 1959.)

(43) Perchonok, P. A. and S. L. Levy. "Application of Digital Simulation Techniques to Freeway On-Ramp Traffic Operations." Highway Research Board, Proceedings, Vol. 39 (1960), pp. 506-523.

(44) Wohl, Martin. "Simulation--Its Application to Traffic Engineering." Traffic Engineering, Vol. 31, No. 1 (Oct., 1960), pp. 19-25.

(45) Glickstein, A., L. D. Findley and S. L. Levy. "Application of Computer Simulation Techniques to Interchange Design Problems." Bulletin 291. Washington, D.C.: Highway Research Board, 1961, pp. 139-162.

(46) Kell, James H. "Analyzing Vehicular Delay at Intersections Through Simulation." Bulletin 356. Washington, D.C.: Highway Research Board, 1962, pp. 28-39.

(47) Kell, James H. "Intersection Delay Obtained by Simulation Traffic on a Computer." Highway Research Record, No. 15 (1963), pp. 73-97.

(48) Lewis, Russell M. "A Proposed Headway Distribution for Traffic Simulation Studies." Traffic Engineering, Vol. 33, No. 5 (Feb., 1963), pp. 16-19, 48.

(49) Beilby, M. H. "Traffic Simulation by Digital Computer." Traffic Engineering and Control, Vol. 10, No. 1 (May, 1968), pp. 23-27.

(50) Story, Christopher E. R. "Simulation of Traffic by Digital Computer." Traffic Engineering and Control, Vol. 11, No. 10 (Feb., 1970), pp. 464-467.

(51) Klijnhout, J. J. "A Digital Simulation Model for Single Intersecsections With Traffic Lights." Traffic Engineering and Control, Vol. 13, No. 4 (Aug., 1971), pp. 147-150, 153.

(52) Katz, J. H. "Simulation of a Traffic Network." ACM Communications, Vol. 6, No. 8 (1963), pp. 480-486.

(53) Bayce, David E. and Seymour E. Goldstone. "A Regional Economic Simulation Model for Urban Transportation Planning." Highway Research Record, No. 149 (1966), pp. 29-41.

(54) Terrell, T. J. et al. "The UMIST Digital Traffic Simulator." Traffic Engineering and Control, Vol. 15, No. 19 (Nov., 1974), pp. 876-878, 884.

(55) Grigg, P. J. and M. G. Hartley. "Simulation of Traffic Flows in a Road Network." Traffic Engineering and Control, Vol. 16, No. 2 (Feb., 1975), pp. 75-77.

(56) Wigan, M. Ramsay. "Applications of Simulation to Traffic Problems." Simulation, Vol. 14, No. 3 (Mar., 1970), pp. 135-136.

(57) Blum, A. M. "A General Purpose Digital Simulator and Examples of Its Application: Part III--Digital Simulation of Urban Traffic." IBM Systems Journal, Vol. 3, No. 1 (1964), pp. 41-50.

(58) Blum, A. M. "A General Purpose Digital Traffic Simulator." Simulation, Vol. 14, No. 1 (Jan., 1970), pp. 9-25.

(59) Gribbins, Paul D., Philip L. Wylie, and Mark M. Boggs. "Evaluation of Urban Traffic Control Improvements by Time-Lapse Photography." Raleigh, N.C.: Highway Research Program, North Carolina Univ., December, 1972.

(60) Institute of Traffic Engineers. Transportation and Traffic Engineering Handbook. Englewood Cliffs, N.J.: Prentice-Hall, Inc., 1976.

APPENDIX A

STARTING DELAY AND HEADWAY OBSERVATION DATA

STARTING DELAY, FIRST VEHICLE IN QUEUE

Direction of Traffic	Lanes	No. of Vehicles Observed	Average time in seconds
Southbound	outside	9	2.93
	inside	10	3.23
	left-turn	24	3.90
Westbound	outside	13	3.25
	inside	15	3.46
	left-turn	9	3.30
Northbound	outside	19	3.30
	inside	8	3.78
	left-turn	22	3.79
Eastbound	outside	10	2.89
	inside	12	3.34
	left-turn	21	3.33

NUMBERS OF VEHICLE HEADWAYS OBSERVED

Direction of Traffic	Lanes	2^{nd} Vehicles	3^{rd} Vehicles	4^{th} Vehicles	5^{th} Vehicles
Southbound	outside	7	6	4	7
	inside	8	8	9	10
	left-turn	9	11	12	8
Westbound	outside	7	10	6	7
	inside	8	6	9	8
	left-turn	2	-	-	-
Northbound	outside	7	8	5	5
	inside	4	6	5	4
	left-turn	6	3	7	2
Eastbound	outside	8	13	14	7
	inside	2	4	-	3
	left-turn	5	6	10	4

HEADWAYS OF OBSERVED VEHICLES IN QUEUES

Direction of Traffic	Lanes	2nd Vehicles in Queue	3rd Vehicles in Queue	4th Vehicles in Queue	5th Vehicles in Queue
Southbound	outside	2.32	2.40	2.25	2.04
	inside	2.53	2.40	2.22	
	left-turn	2.66	2.09	2.18	2.01
Westbound	outside	3.11	2.71	2.23	2.29
	inside	2.42	2.45		
	left-turn	3.20	-	-	-
Northbound	outside	3.05	2.58	2.21	2.31
	inside	2.35	2.25		
	left-turn	2.43	2.00	2.10	1.80
Eastbound	outside	2.25	2.23	2.06	2.14
	inside	2.40	2.17	-	
	left-turn	2.40	2.31	1.94	1.83

STARTING DELAY OF QUEUED VEHICLES

Types of Traffic	Starting Delay 1st Vehicles	Headways			
		2nd Vehicles	3rd Vehicles	4th Vehicles	5th Vehicles
Through & Right-turn Vehicles	3.26	2.55	2.39	2.19	2.23
Left-turn Vehicles	3.58	2.49	2.20	2.07	1.92

APPENDIX B

A STUDY OF TRAVELING TIME ACROSS INTERSECTION PROPER

NUMBER OF VEHICLES FOR WHICH INTERSECTION TRAVELING TIME WAS MANUALLY OBSERVED

Direction of Traffic	Lanes	Non-stop Vehicles	1st Vehicles	2nd Vehicles	3rd Vehicles	4th Vehicles	5th Vehicles
Southbound	outside (Th.)[1]	3	3	3	1	2	2
	(R)[2]	5	6	2	1	4	1
	inside	3	4	9	13	5	6
	left-turn	8	10	11	13	10	-
Westbound	outside (Th)	4	4	3	5	7	4
	(R)	2	1	4	3	1	5
	inside	9	15	7	4	6	2
	left-turn	4	14	2	-	-	-
Northbound	outside (Th)	6	2	7	6	6	1
	(R)	-	2	1	-	1	2
	inside	4	1	8	6	1	4
	left-turn	2	1	10	8	1	-
Eastbound	outside (Th)	2	2	4	8	7	6
	(R)	-	4	1	3	4	4
	inside	4	6	6	4	3	7
	left-turn	3	6	11	5	2	-

[1] Straight through vehicles

[2] Right turning vehicles

OBSERVED INTERSECTION TRAVELING TIMES

Direction of Traffic	Lanes	Non-stop Vehicles	1st Vehicles	2nd Vehicles	3rd Vehicles	4th Vehicles	5th Vehicles
Southbound	outside (Th)[1]	1.86	-	3.56	3.30	3.05	2.70
	outside (R)[2]	2.18	-	2.15	1.32	1.65	1.25
	inside	2.13	2.84	3.17	3.00	2.68	3.15
	left-turn	2.32	2.20	3.16	3.19	2.91	-
Westbound	outside (Th)	2.45	2.73	3.06	2.80	2.82	2.75
	outside (R)	1.60	-	1.98	1.43	1.50	1.54
	inside	2.45	3.5	3.14	2.77	2.55	2.85
	left-turn	2.32	3.6	2.60	-	-	-
Northbound	outside (Th)	2.06	3.20	2.77	2.96	2.70	-
	outside (R)	-	-	1.70	-	-	-
	inside	2.22	-	3.26	2.91	2.70	2.95
	left-turn	2.30	-	3.00	3.08	3.50	-
Eastbound	outside (Th)	2.07	2.9	3.30	3.40	3.40	3.32
	outside (R)	-	1.3	1.70	1.10	1.33	1.15
	inside	1.98	2.7	3.28	3.10	2.86	3.02
	left-turn	2.46	2.9	3.45	3.30	3.20	-

[1] Straight through vehicles

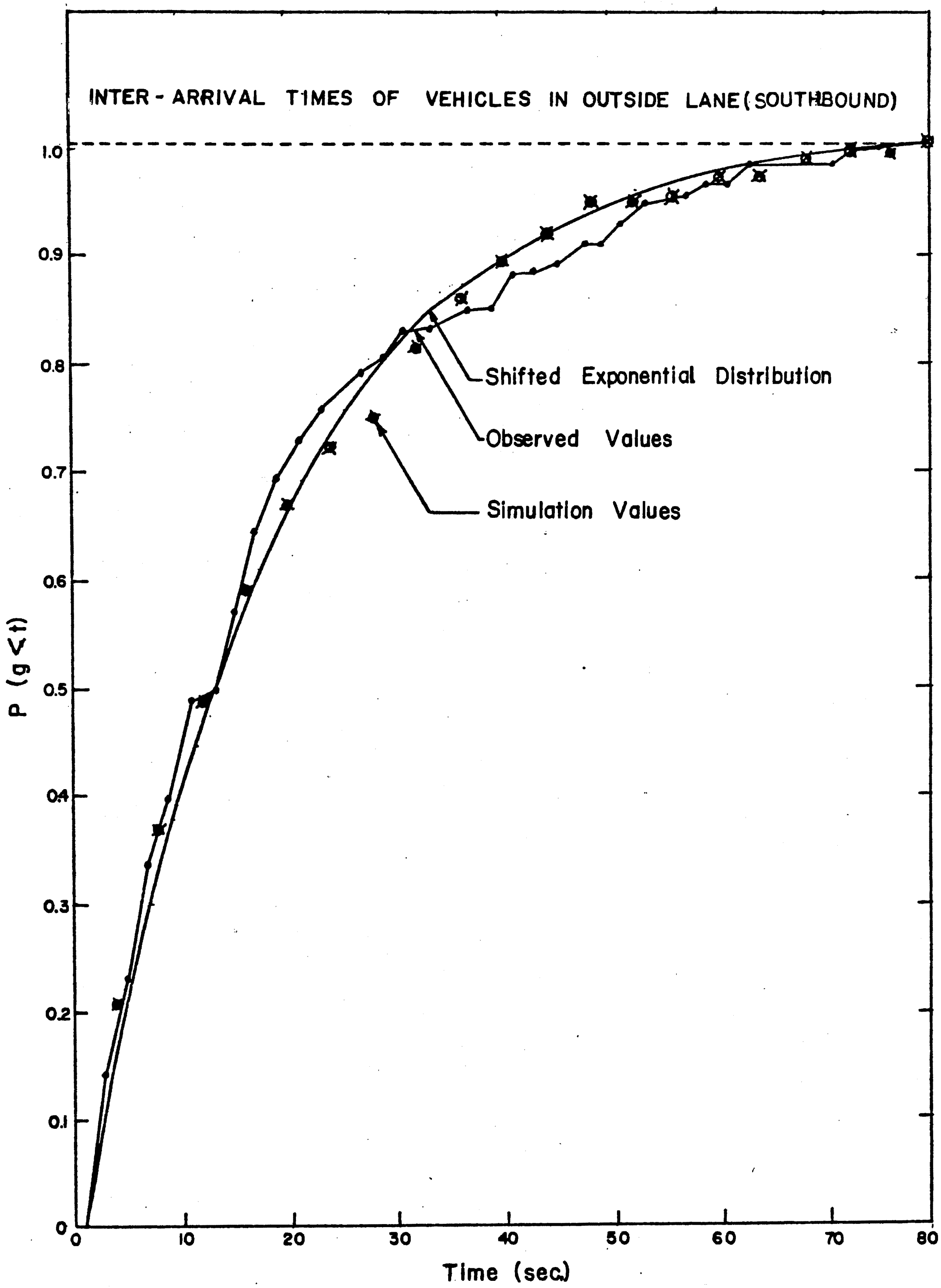
[2] Right turning vehicles

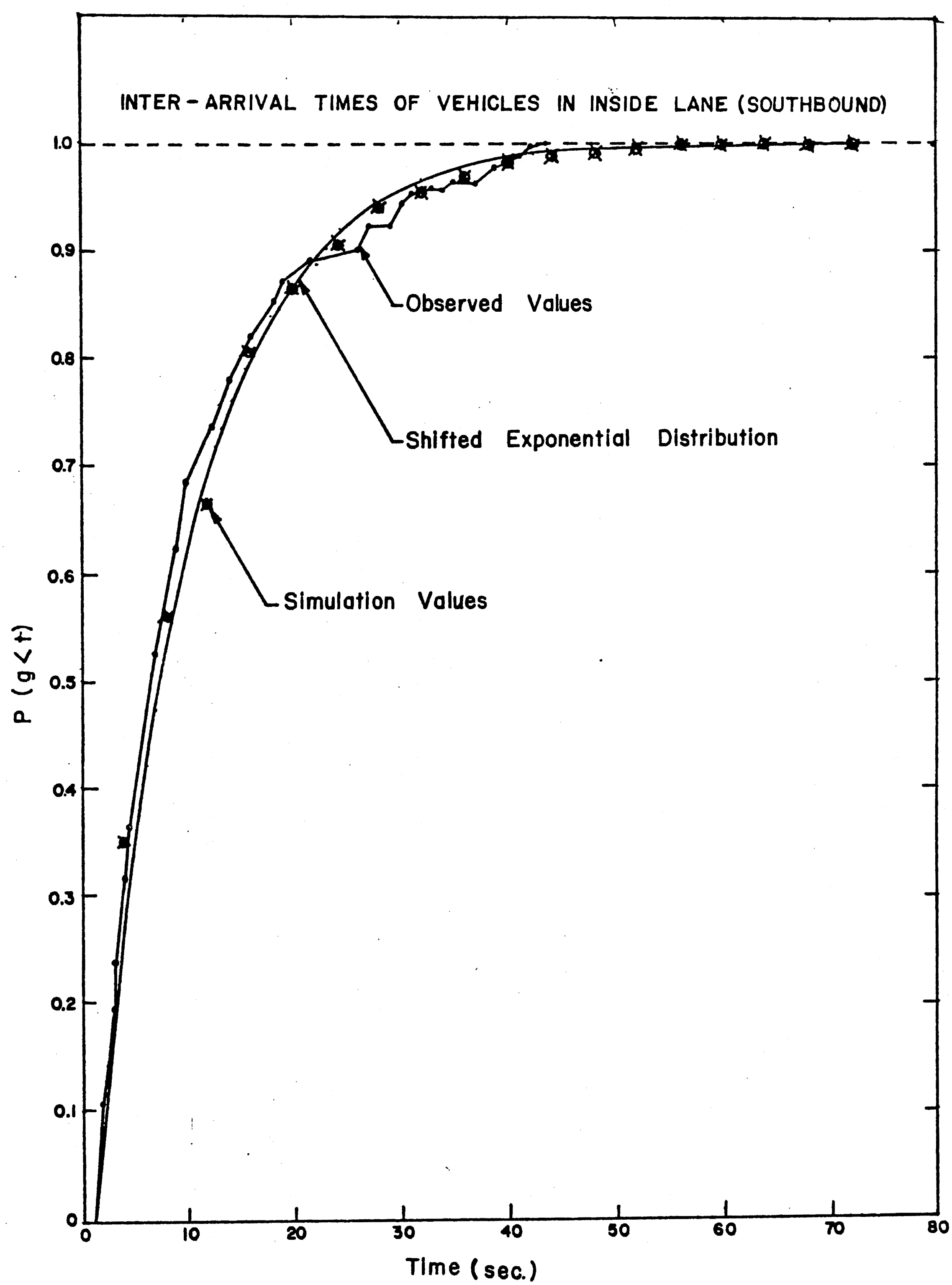
AVERAGE INTERSECTION TRAVELING TIMES

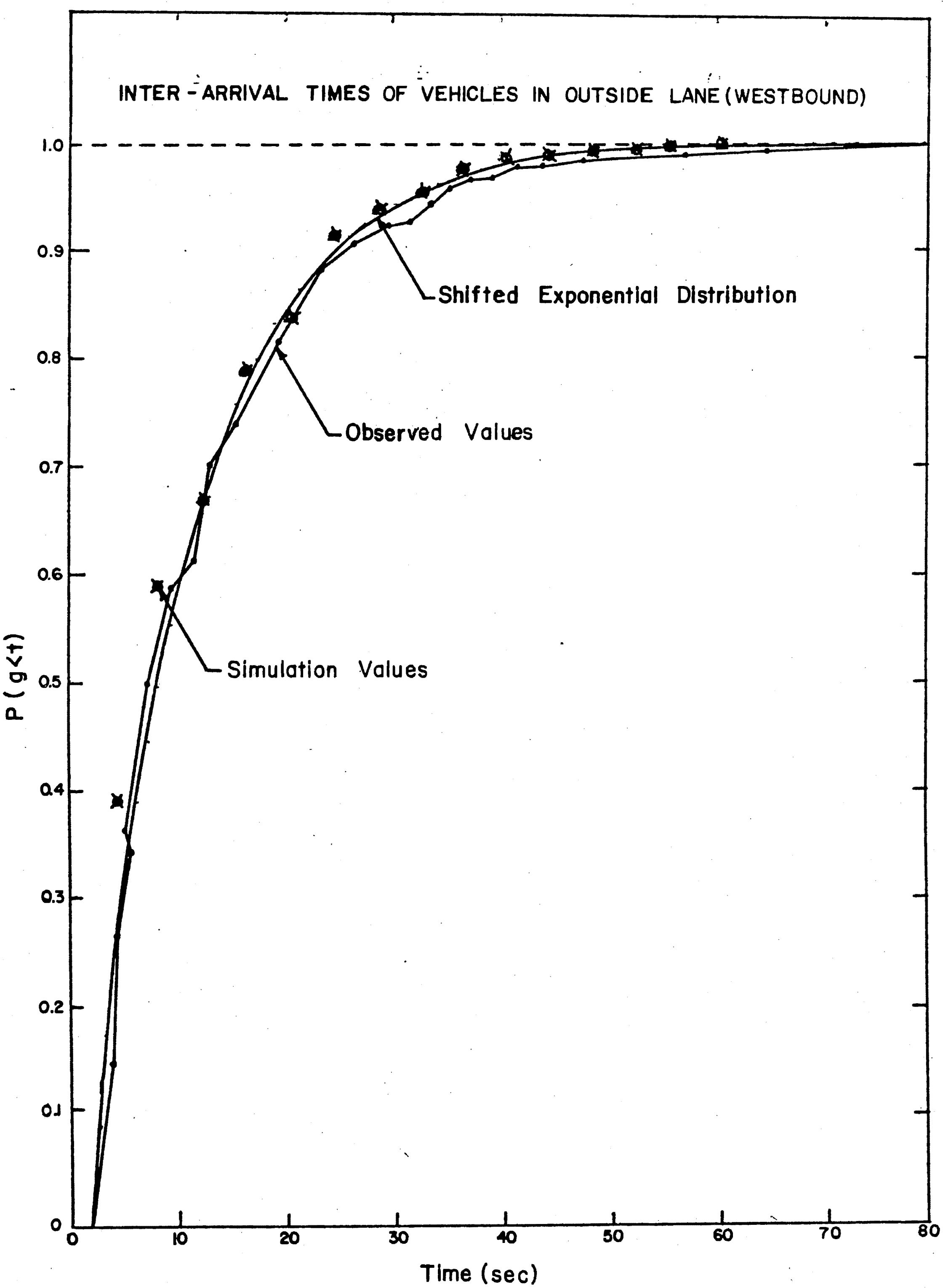
Vehicles	Non-stop Vehicles	1^{st} Vehicles	2^{nd} Vehicles	3^{rd} Vehicles	4^{th} Vehicles	5^{th} Vehicles
Right-turn	2.70	1.92	1.98	1.32	1.65	1.25
Through	2.13	2.80	3.21	2.99	2.86	3.03
Left-turn	2.80	2.80	3.05	3.19	3.05	-

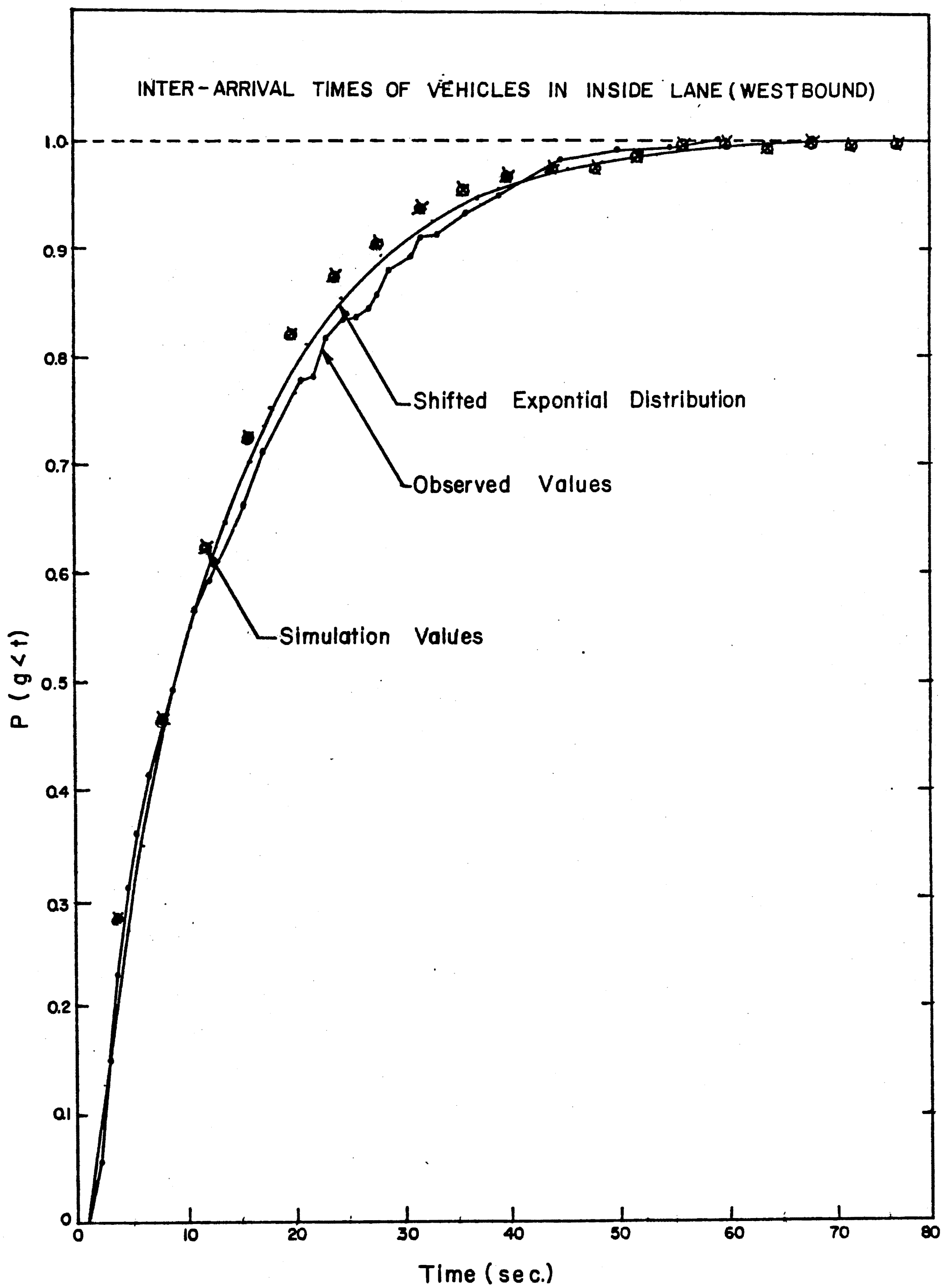
APPENDIX C

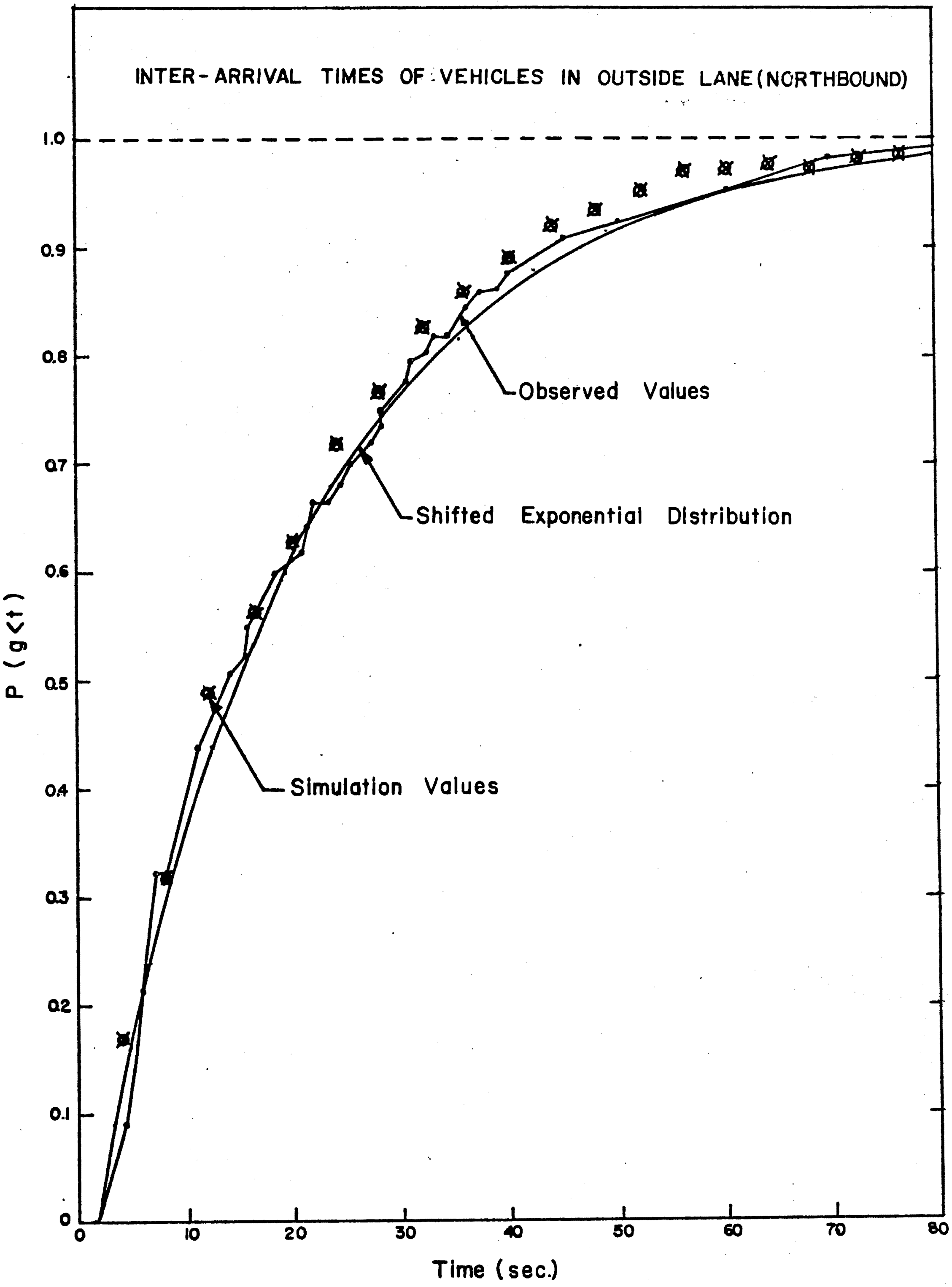
INTER-ARRIVAL TIMES OF VEHICLES

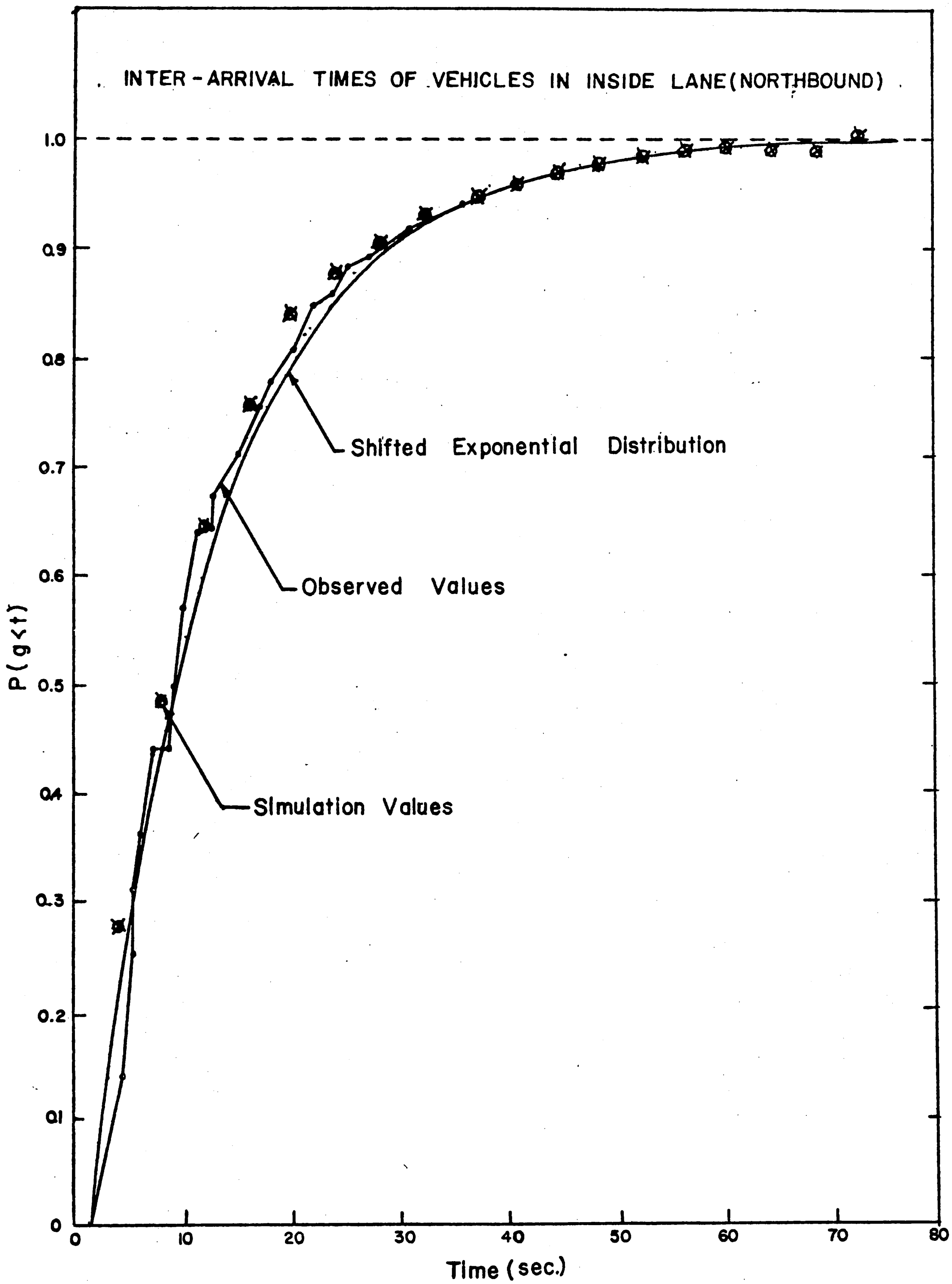

INTER - ARRIVAL TIMES OF VEHICLES IN OUTSIDE LANE (SOUTHBOUND)
Shifted Exponential Distribution
Observed Values
Simulation Values
P (g < t)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
10
20
30
40
50
60
70
80
Time (sec.)

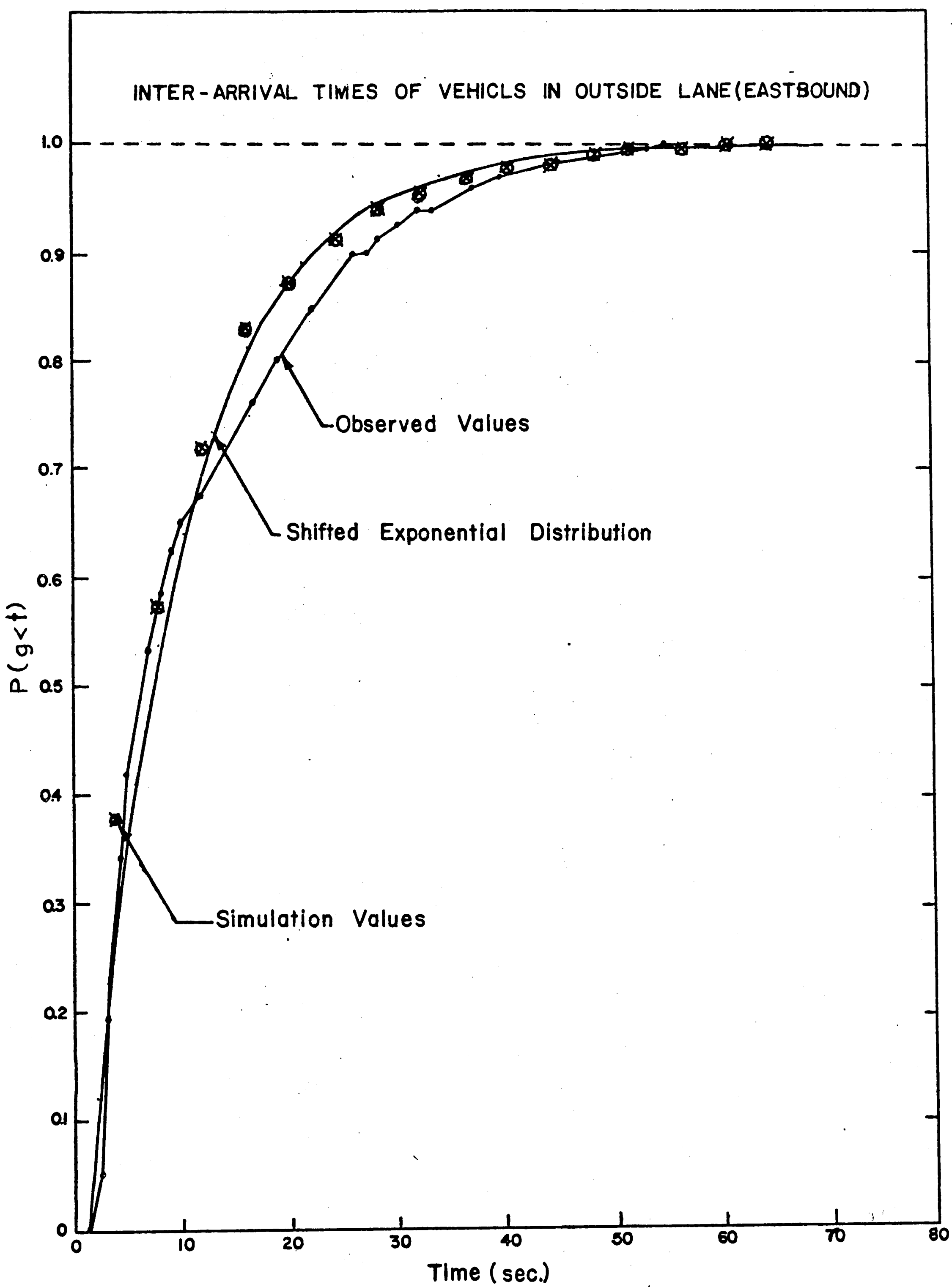

INTER - ARRIVAL TIMES OF VEHICLES IN INSIDE LANE (SOUTHBOUND)
Observed Values
Shifted Exponential Distribution
Simulation Values
P (g < t)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
10
20
30
40
50
60
70
80
Time (sec.)

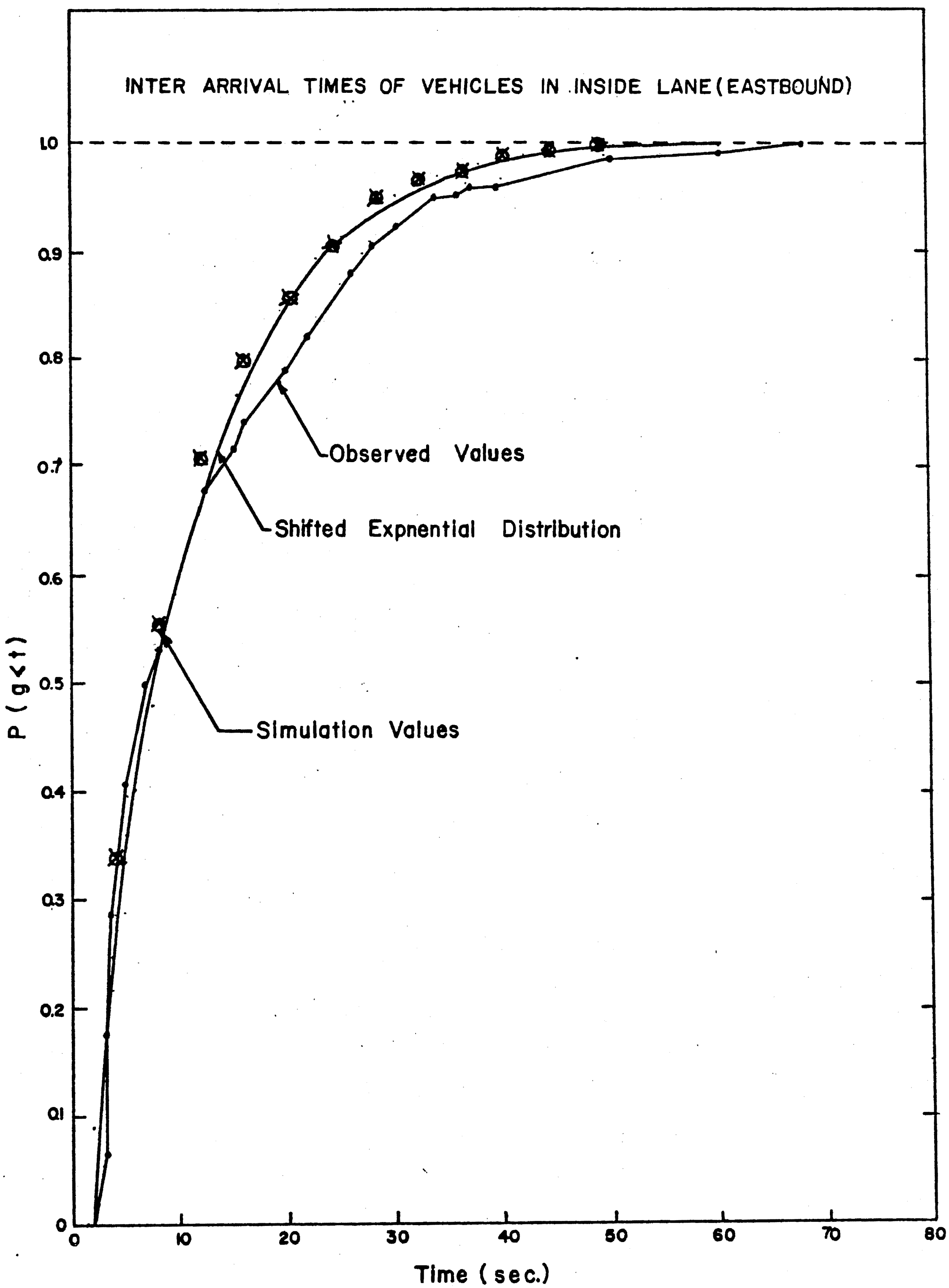

INTER-ARRIVAL TIMES OF VEHICLES IN OUTSIDE LANE (WESTBOUND)
Shifted Exponential Distribution
Observed Values
Simulation Values
P (g<t)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
10
20
30
40
50
60
70
80
Time (sec)


INTER-ARRIVAL TIMES OF VEHICLES IN INSIDE LANE (WESTBOUND)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
P (g < t)
0
10
20
30
40
50
60
70
80
Time (sec.)
Shifted Expontial Distribution
Observed Values
Simulation Values


INTER-ARRIVAL TIMES OF VEHICLES IN OUTSIDE LANE (NORTHBOUND)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
P (g<t)
Observed Values
Shifted Exponential Distribution
Simulation Values
0
10
20
30
40
50
60
70
80
Time (sec.)


INTER-ARRIVAL TIMES OF VEHICLES IN INSIDE LANE (NORTHBOUND)
Shifted Exponential Distribution
Observed Values
Simulation Values
P (g<t)
Time (sec.)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
10
20
30
40
50
60
70
80

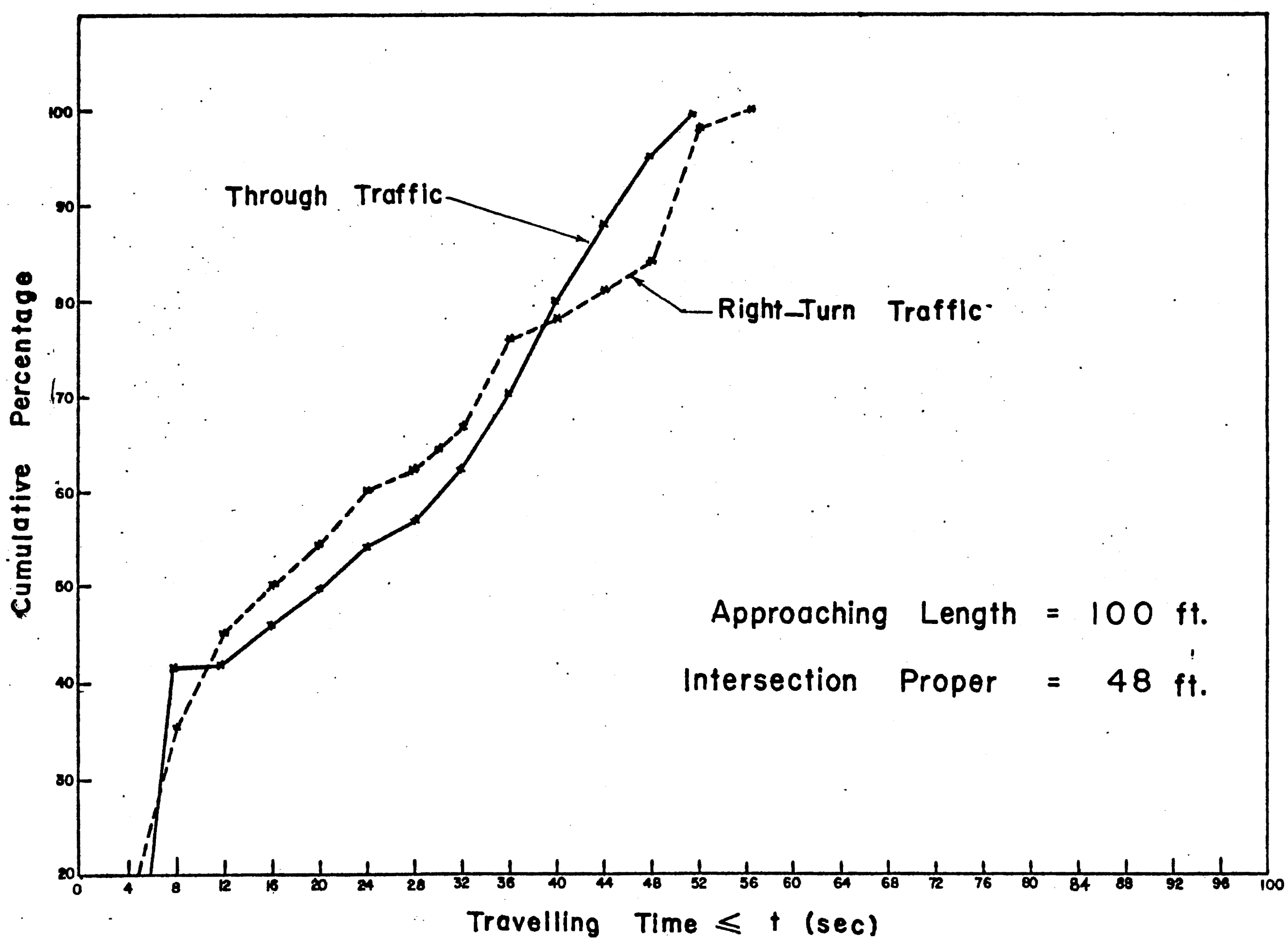

INTER-ARRIVAL TIMES OF VEHICLS IN OUTSIDE LANE(EASTBOUND)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
P(g<t)
Observed Values
Shifted Exponential Distribution
Simulation Values
0
10
20
30
40
50
60
70
80
Time (sec.)


INTER ARRIVAL TIMES OF VEHICLES IN INSIDE LANE (EASTBOUND)
Observed Values
Shifted Expnential Distribution
Simulation Values
P (g<t)
Time (sec.)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
10
20
30
40
50
60
70
80

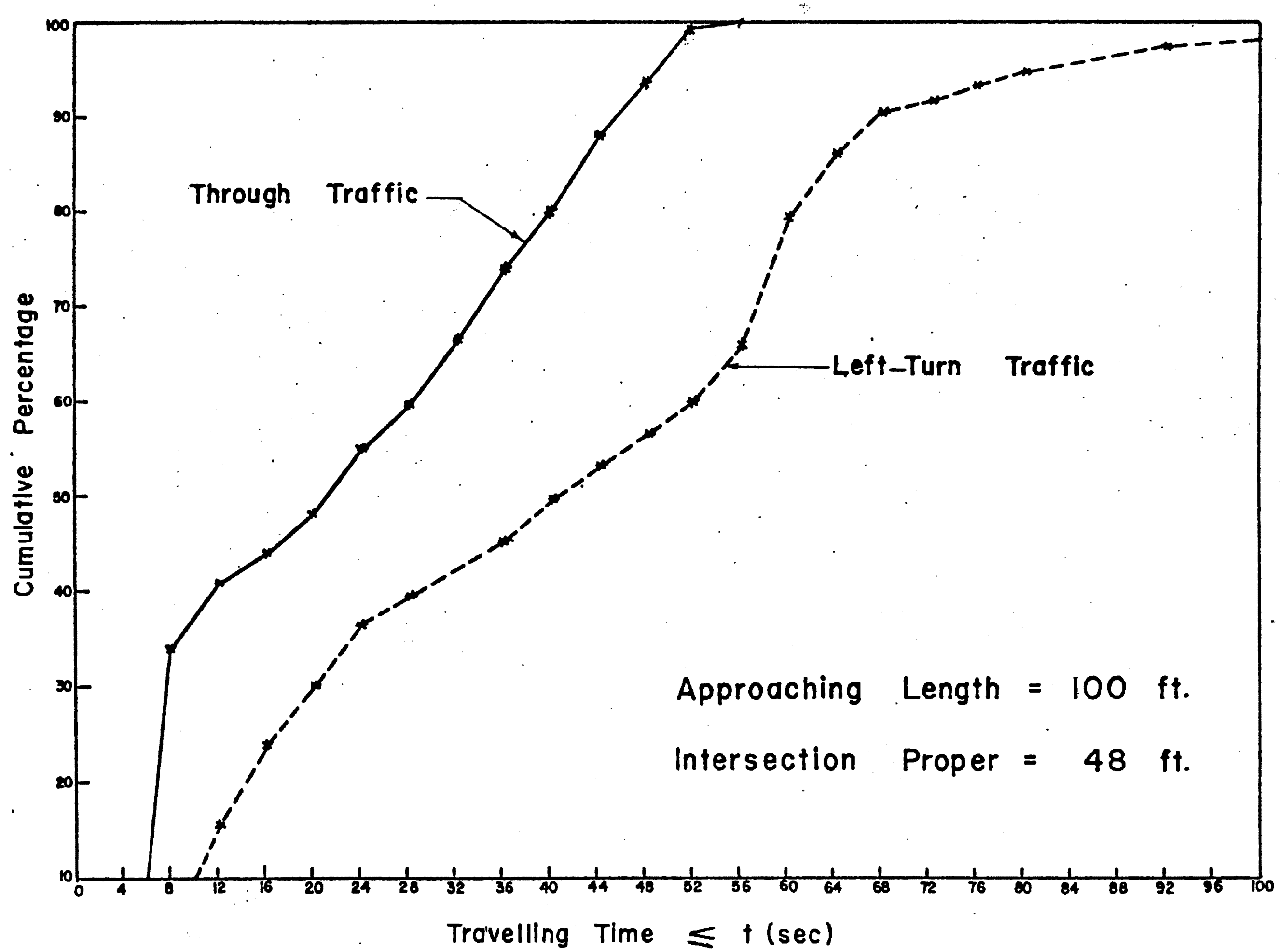
APPENDIX D

CUMULATIVE PERCENTAGE OF VEHICLE TRAVELING TIMES

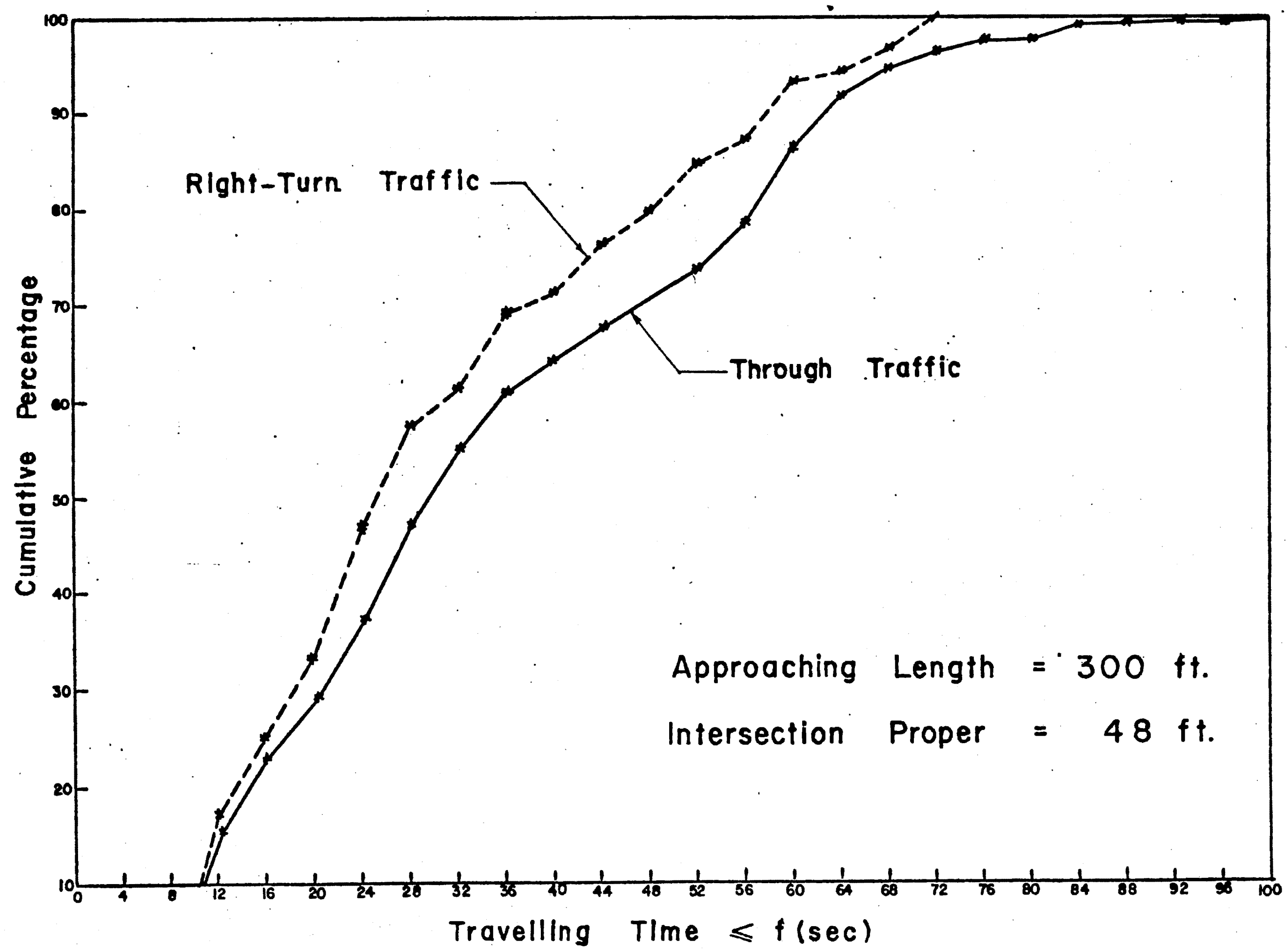
TRAVELING TIME OF VEHICLES IN OUTSIDE LANE (SOUTHBOUND)



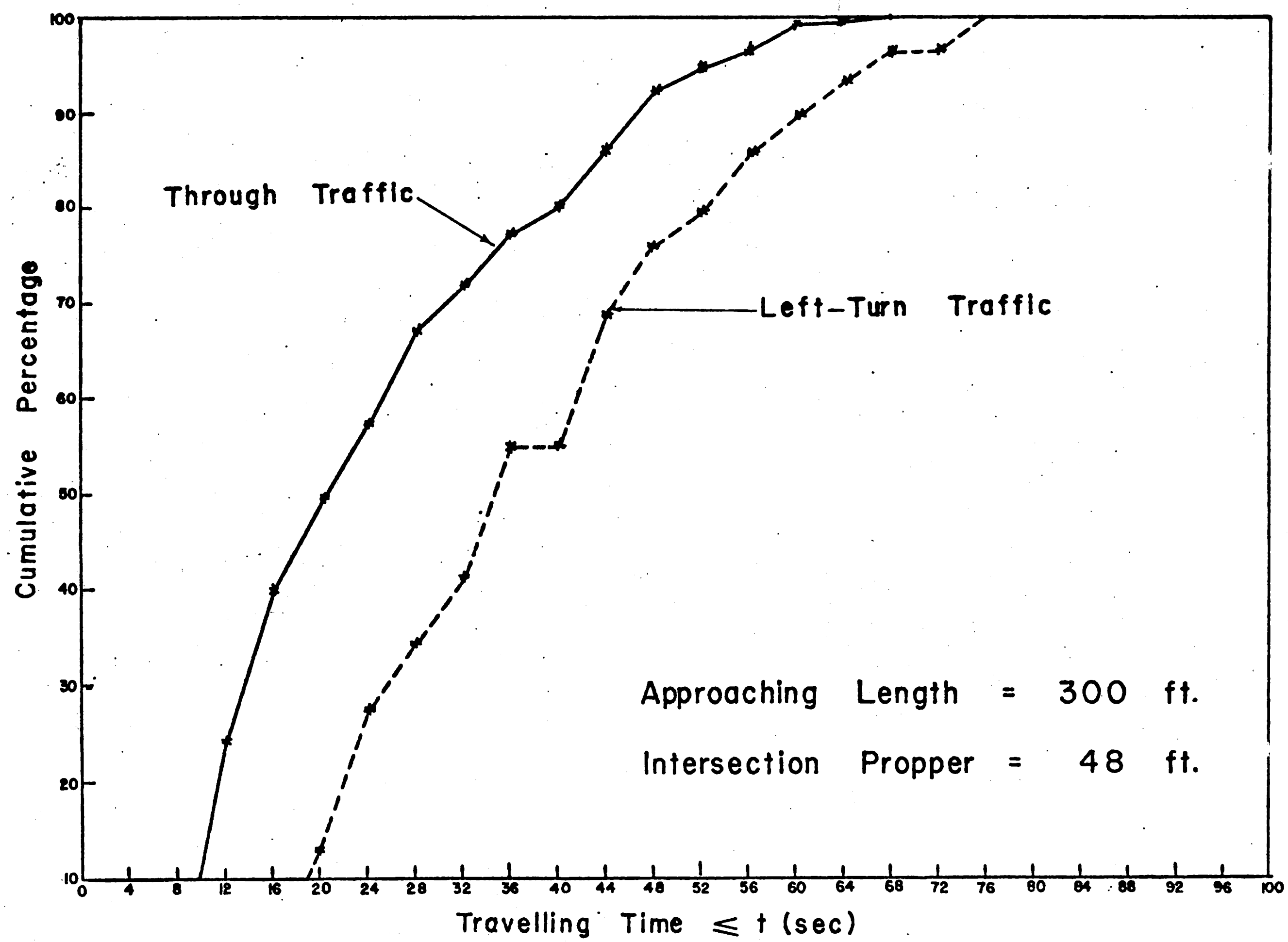
TRAVELING TIME OF VEHICLES IN INSIDE LANE (SOUTHBOUND)



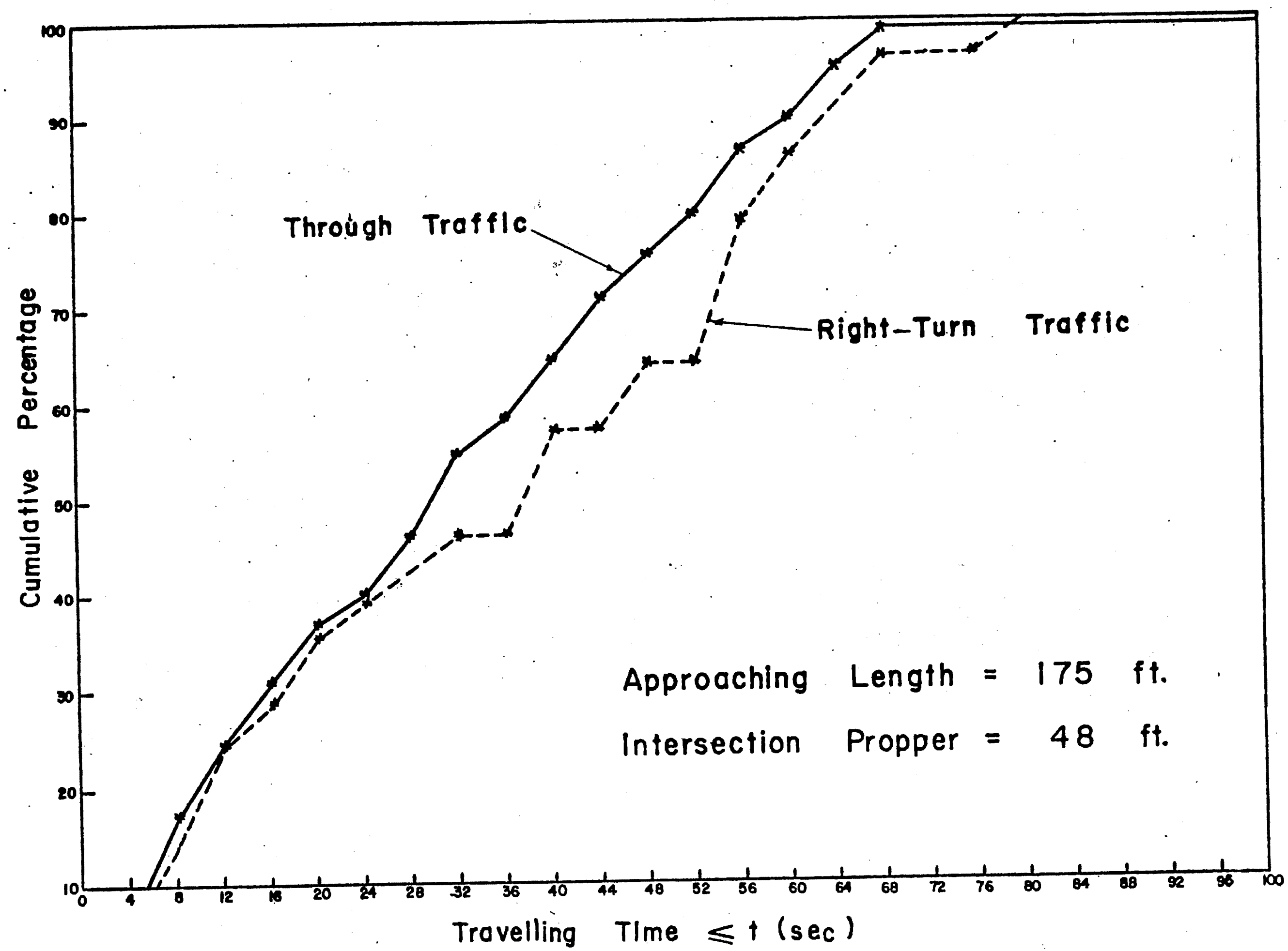
TRAVELING TIME OF VEHICLES IN OUTSIDE LANE (WESTBOUND)



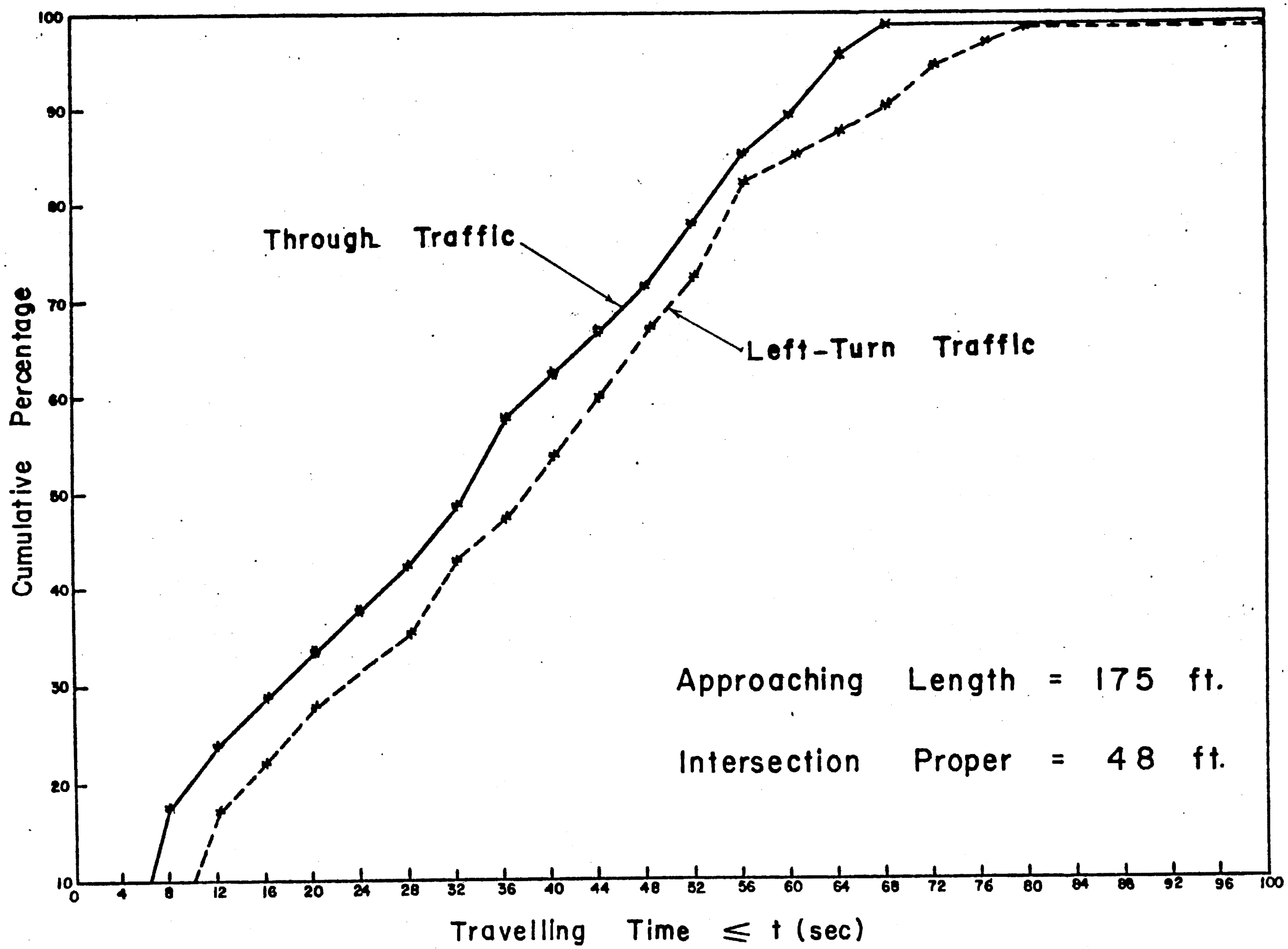
TRAVELING TIME OF VEHICLES IN INSIDE LANE (WESTBOUND)



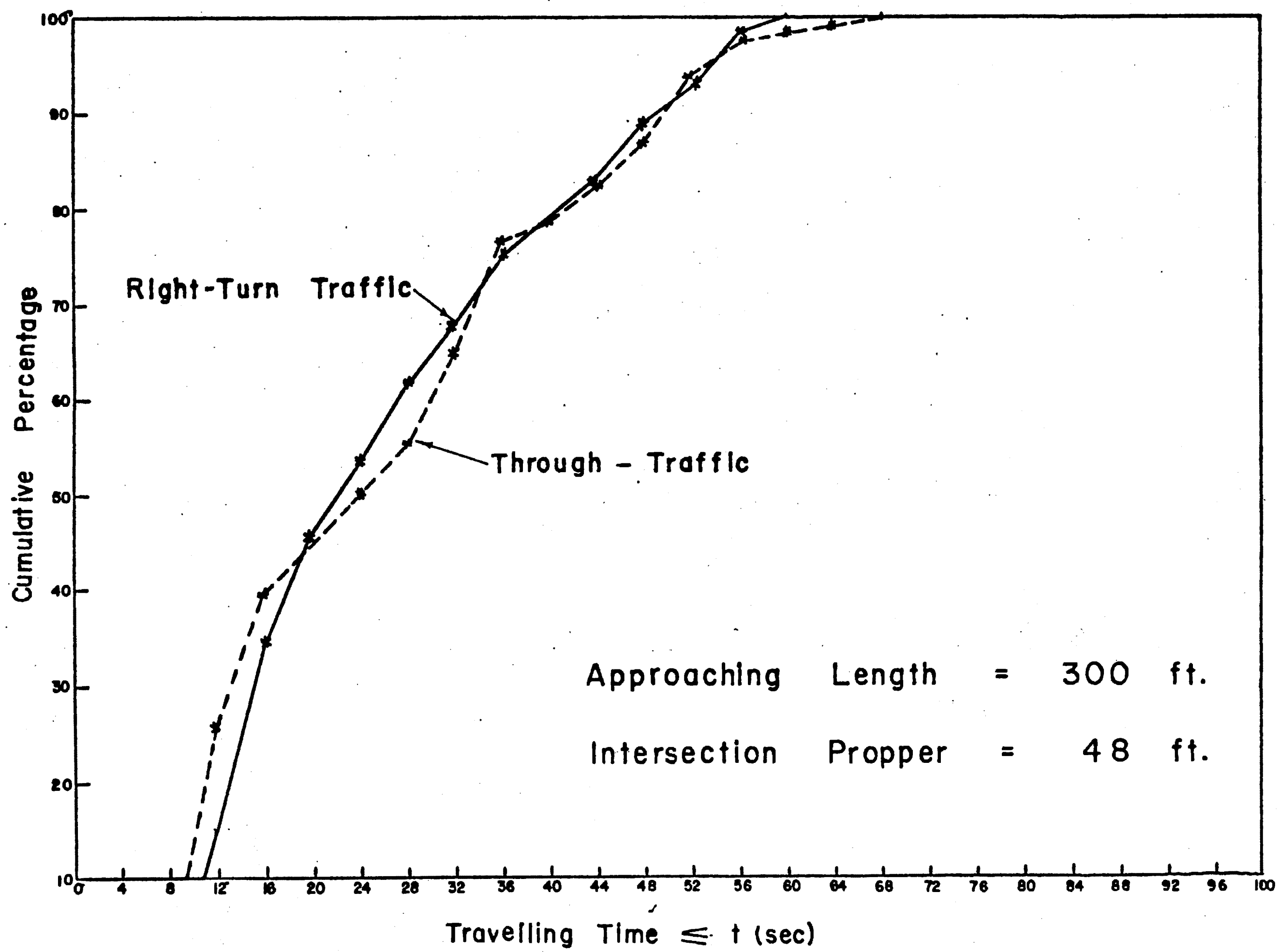
TRAVELING TIME OF VEHICLES IN OUTSIDE LANE (NORTHBOUND)



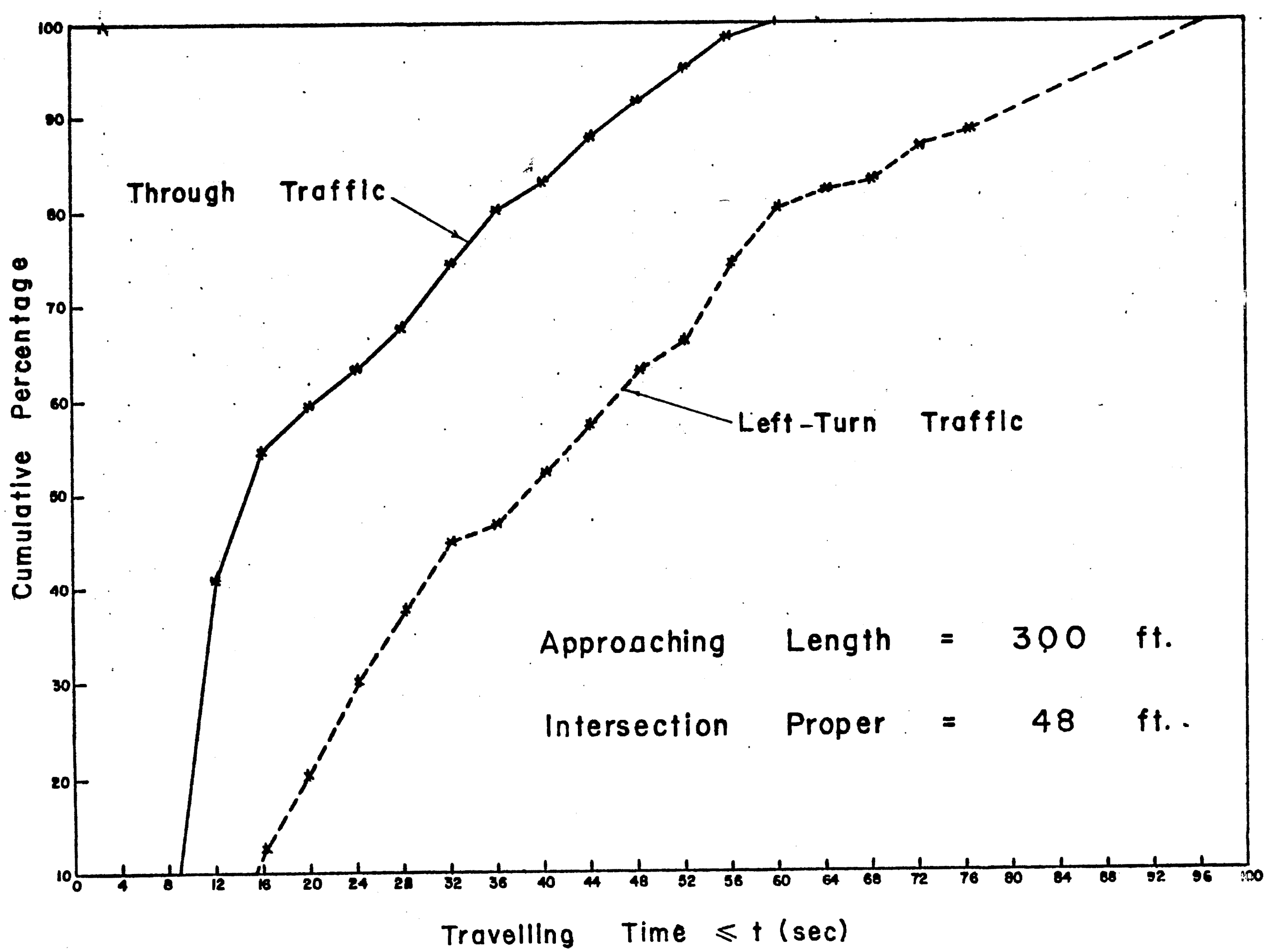
TRAVELING TIME OF VEHICLES IN INSIDE LANE (NORTHBOUND)



TRAVELING TIME OF VEHICLES IN OUTSIDE LANE (EASTBOUND)

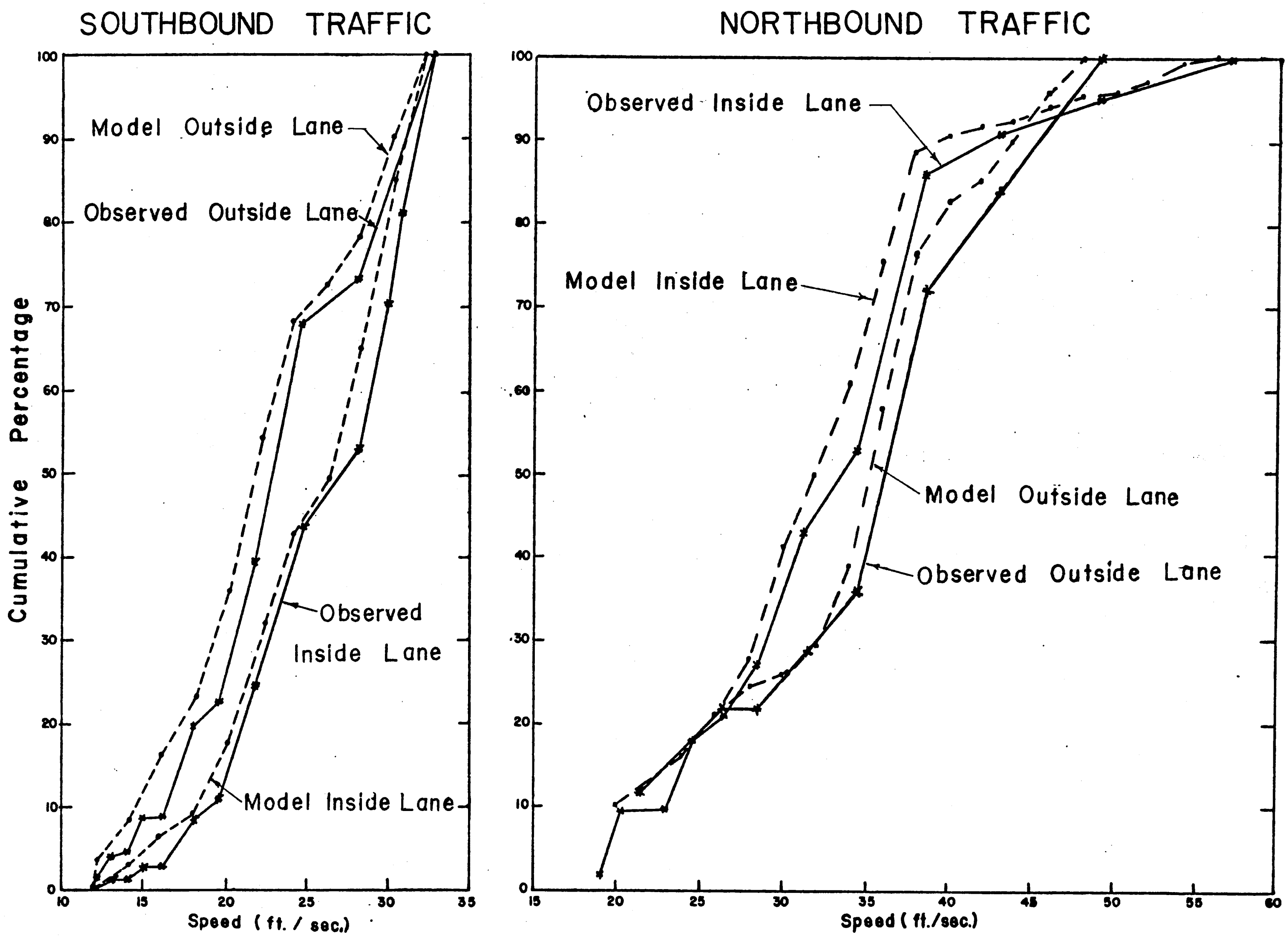


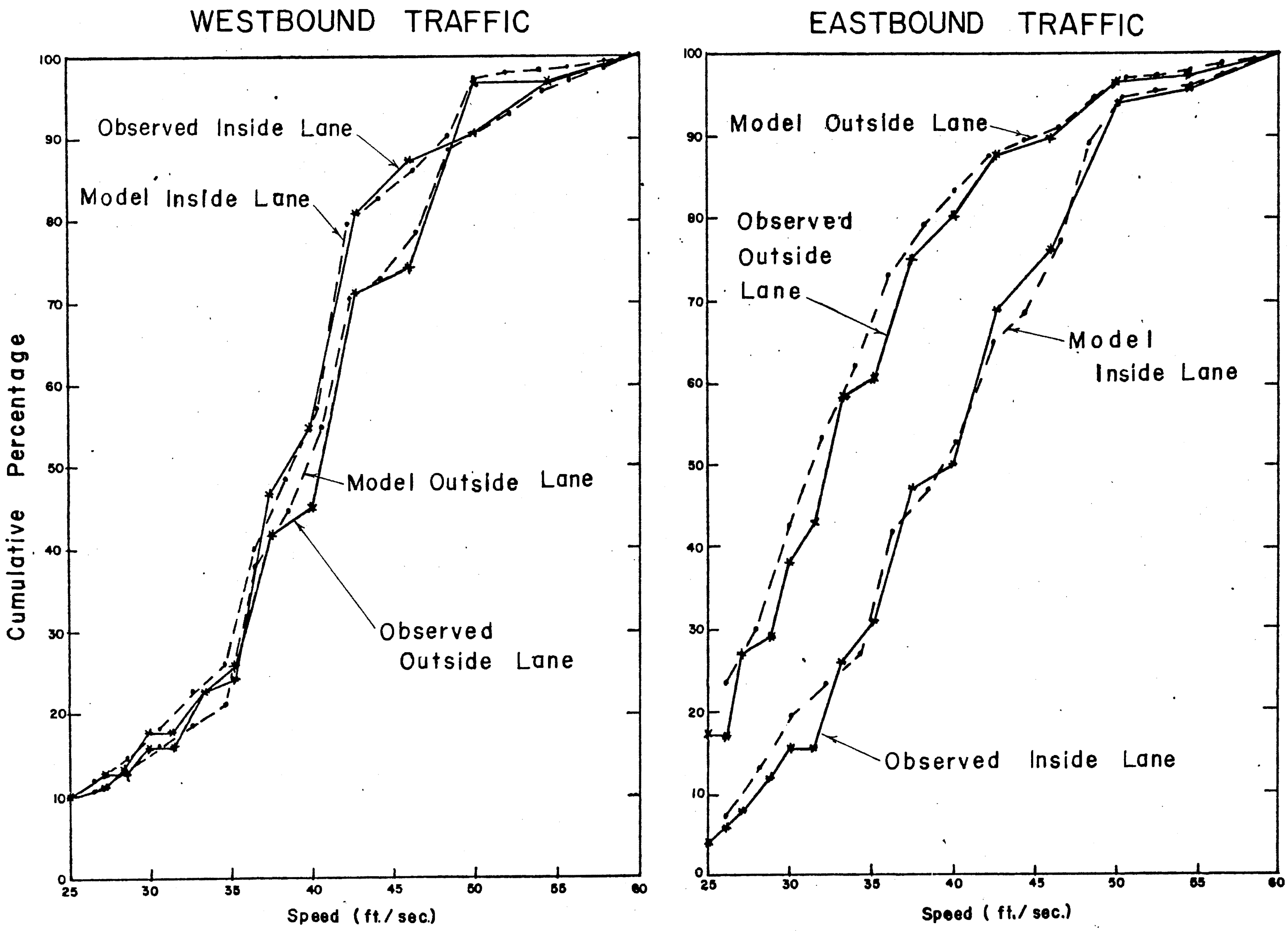
TRAVELING TIME OF VEHICLES IN INSIDE LANE (EASTBOUND)



APPENDIX E

VEHICLE SPEED DISTRIBUTIONS


SOUTHBOUND TRAFFIC
Model Outside Lane
Observed Outside Lane
Observed Inside Lane
Model Inside Lane
Cumulative Percentage
Speed (ft. / sec.)
NORTHBOUND TRAFFIC
Observed Inside Lane
Model Inside Lane
Model Outside Lane
Observed Outside Lane
Speed (ft./sec.)



WESTBOUND TRAFFIC
EASTBOUND TRAFFIC
Cumulative Percentage
Speed (ft./sec.)
Observed Inside Lane
Model Inside Lane
Model Outside Lane
Observed Outside Lane
Model Outside Lane
Observed Outside Lane
Model Inside Lane
Observed Inside Lane
0
10
20
30
40
50
60
70
80
90
100
25
30
35
40
45
50
55
60

APPENDIX F

STANDARD STATISTICAL OUTPUT ASSOCIATED WITH CLOCK AND BLOCK ENTITIES

RELATIVE CLOCK 36000 ABSOLUTE CLOCK 36000

BLOCK COUNTS

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
1	0	1	11	0	44	21	0	59	31	0	60	41	1	60
2	0	60	12	0	44	22	0	59	32	0	60	42	0	59
3	0	60	13	0	60	23	0	1	33	0	46	43	0	59
4	0	60	14	0	60	24	0	60	34	0	46	44	0	59
5	0	60	15	0	60	25	0	60	35	0	60	45	0	1
6	0	44	16	0	60	26	0	60	36	0	60	46	0	60
7	0	44	17	0	60	27	0	60	37	0	60	47	0	60
8	0	60	18	0	60	28	0	46	38	0	60	48	0	60
9	0	60	19	1	60	29	0	46	39	0	60	49	0	60
10	0	60	20	0	59	30	0	60	40	0	60	50	0	47
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
51	0	47	61	0	59	71	0	60	81	0	59	91	0	189
52	0	60	62	0	59	72	0	60	82	0	59	92	0	189
53	1	60	63	0	59	73	0	20	83	0	59	93	0	189
54	0	59	64	0	59	74	0	20	84	0	59	94	0	189
55	0	46	65	0	59	75	0	60	85	0	59	95	1	189
56	0	46	66	0	59	76	1	60	86	0	59	96	0	188
57	0	59	67	0	59	77	0	59	87	0	59	97	0	188
58	0	59	68	0	1	78	0	19	88	0	59	98	0	188
59	0	59	69	0	60	79	0	19	89	0	59	99	0	188
60	0	59	70	0	60	80	0	59	90	1	190	100	0	188
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
101	0	79	111	0	17	121	0	55	131	0	54	141	0	129
102	0	188	112	0	12	122	0	52	132	0	54	142	0	134
103	0	188	113	0	188	123	0	52	133	0	54	143	0	134
104	0	188	114	0	188	124	0	5	134	0	66	144	0	134
105	0	188	115	0	191	125	0	5	135	0	66	145	0	134
106	0	188	116	0	71	126	0	16	136	0	129	146	0	58
107	0	188	117	0	55	127	0	15	137	0	129	147	0	58
108	0	188	118	0	55	128	0	16	138	0	56	148	0	58
109	0	188	119	0	55	129	0	120	139	0	73	149	0	0
110	0	188	120	0	55	130	0	120	140	0	129	150	0	58
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
151	0	58	161	0	188	171	0	79	181	0	109	191	0	109
152	0	58	162	0	188	172	0	79	182	0	109	192	1	332
153	0	58	163	0	188	173	0	79	183	0	109	193	0	331
154	0	0	164	0	188	174	0	79	184	0	109	194	0	331
155	0	58	165	0	188	175	0	79	185	0	109	195	0	331
156	0	76	166	0	79	176	0	109	186	0	109	196	0	331
157	0	76	167	0	79	177	0	109	187	0	109	197	1	331
158	0	76	168	0	79	178	0	109	188	0	109	198	0	330
159	0	134	169	0	79	179	0	109	189	0	109	199	0	330
160	0	134	170	0	79	180	0	109	190	0	109	200	0	330
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
201	0	330	211	0	330	221	0	233	231	0	0	241	0	56
202	0	330	212	0	330	222	0	233	232	0	168	242	0	202
203	0	330	213	0	330	223	0	233	233	0	168	243	0	258
204	0	118	214	0	61	224	0	233	234	0	64	244	0	258
205	0	330	215	0	30	225	0	162	235	0	64	245	0	266
206	0	330	216	0	330	226	0	162	236	0	64	246	0	266
207	0	330	217	0	330	227	0	8	237	0	104	247	0	266
208	0	330	218	0	401	228	0	8	238	0	104	248	0	266
209	0	330	219	0	233	229	0	0	239	0	258	249	0	100
210	0	330	220	0	233	230	0	0	240	0	258	250	0	100
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
251	0	100	261	0	166	271	0	118	281	0	212	291	0	212
252	0	0	262	0	266	272	0	118	282	0	212	292	0	212
253	0	100	263	0	266	273	0	118	283	0	212	293	0	212
254	0	100	264	0	330	274	0	118	284	0	212	294	0	212
255	0	100	265	0	330	275	0	118	285	0	212	295	1	181
256	0	100	266	0	330	276	0	118	286	0	212	296	0	180
257	0	0	267	0	330	277	0	118	287	0	212	297	0	180
258	0	100	268	0	330	278	0	118	288	0	212	298	0	180
259	0	166	269	0	118	279	0	212	289	0	212	299	0	180
260	0	166	270	0	118	280	0	212	290	0	212	300	1	180

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
301	0	179	311	0	179	321	0	198	331	0	8	341	0	69
302	0	179	312	0	179	322	0	85	332	0	1	342	0	127
303	0	179	313	0	179	323	0	84	333	0	1	343	0	127
304	0	179	314	0	179	324	0	84	334	0	1	344	0	57
305	0	179	315	0	179	325	0	84	335	0	113	345	0	70
306	0	179	316	0	179	326	0	84	336	0	113	346	0	127
307	0	28	317	0	18	327	0	84	337	0	44	347	0	127
308	0	179	318	0	10	328	0	65	338	0	44	348	0	135
309	0	179	319	0	179	329	0	65	339	0	44	349	0	135
310	0	179	320	0	179	330	0	8	340	0	69	350	0	135
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
351	0	135	361	0	21	371	0	178	381	0	28	391	0	150
352	0	21	362	1	114	372	0	28	382	0	150	392	0	150
353	0	21	363	0	113	373	0	28	383	0	150	393	0	150
354	0	21	364	0	113	374	0	28	384	0	150	394	0	150
355	0	0	365	0	134	375	0	28	385	0	150	395	0	150
356	0	21	366	0	134	376	0	28	386	0	150	396	0	150
357	0	21	367	0	178	377	0	28	387	0	150	397	0	150
358	0	21	368	0	178	378	0	28	388	0	150	398	1	345
359	0	21	369	0	178	379	0	28	389	0	150	399	0	344
360	0	0	370	0	178	380	0	28	390	0	150	400	0	344
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
401	0	344	411	0	343	421	0	35	431	0	135	441	0	104
402	0	344	412	0	343	422	0	343	432	0	135	442	0	104
403	1	344	413	0	343	423	0	343	433	0	9	443	0	102
404	0	343	414	0	343	424	0	425	434	0	9	444	1	102
405	0	343	415	0	343	425	0	217	435	0	0	445	0	227
406	0	343	416	0	343	426	0	217	436	0	0	446	0	227
407	0	343	417	0	343	427	0	217	437	0	0	447	0	56
408	0	343	418	0	343	428	0	217	438	2	208	448	0	171
409	0	343	419	0	343	429	0	217	439	0	206	449	0	227
410	0	94	420	0	67	430	0	217	440	0	104	450	0	227
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
451	0	236	461	0	65	471	0	340	481	0	93	491	0	247
452	0	236	462	0	65	472	0	340	482	0	93	492	0	247
453	0	236	463	0	0	473	0	340	483	0	93	493	0	247
454	0	236	464	0	65	474	0	340	484	0	93	494	0	247
455	0	65	465	0	171	475	0	93	485	0	247	495	0	247
456	0	65	466	0	171	476	0	93	486	0	247	496	0	247
457	0	65	467	0	171	477	0	93	487	0	247	497	0	247
458	0	0	468	0	236	478	0	93	488	0	247	498	0	247
459	0	65	469	0	236	479	0	93	489	0	247	499	0	247
460	0	65	470	0	340	480	0	93	490	0	247	500	0	247
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
501	1	321	511	0	319	521	0	319	531	0	112	541	0	8
502	0	320	512	0	319	522	0	319	532	0	211	542	0	7
503	0	320	513	0	0	523	0	319	533	0	104	543	0	7
504	0	320	514	0	0	524	0	319	534	0	63	544	0	41
505	0	320	515	0	319	525	0	319	535	0	63	545	0	41
506	1	320	516	0	112	526	0	58	536	0	63	546	0	41
507	0	319	517	0	319	527	0	31	537	0	63	547	0	107
508	0	319	518	0	319	528	0	319	538	0	63	548	0	107
509	0	319	519	0	319	529	0	319	539	1	59	549	0	47
510	0	319	520	0	319	530	0	319	540	0	58	550	0	47
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
551	0	47	561	0	159	571	0	0	581	0	205	591	0	205
552	0	47	562	0	159	572	0	0	582	0	205	592	0	205
553	0	60	563	0	159	573	0	0	583	0	205	593	0	205
554	0	65	564	0	159	574	1	159	584	0	205	594	0	205
555	0	152	565	0	159	575	0	158	585	0	205	595	0	205
556	0	152	566	0	159	576	0	158	586	0	205	596	0	205
557	0	50	567	0	159	577	0	158	587	0	205	597	0	205
558	0	102	568	0	0	578	0	158	588	0	205	598	0	205
559	0	152	569	0	0	579	0	205	589	0	205	599	0	205
560	0	152	570	0	0	580	0	205	590	0	205	600	0	205

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
601	0	0	611	0	0	621	0	0	631	0	112	641	0	110
602	0	0	612	0	0	622	0	0	632	0	112	642	0	42
603	0	0	613	0	0	623	0	0	633	0	112	643	0	68
604	0	0	614	0	0	624	0	0	634	0	112	644	0	110
605	0	0	615	0	0	625	0	0	635	0	112	645	0	110
606	0	0	616	0	0	626	0	0	636	0	112	646	0	110
607	0	0	617	0	0	627	0	0	637	0	112	647	0	110
608	0	0	618	0	0	628	0	0	638	0	44	648	0	110
609	0	0	619	0	0	629	0	112	639	0	2	649	1	110
610	0	0	620	0	0	630	0	112	640	0	110	650	0	109

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
651	0	109	661	0	111	671	0	111	681	0	275	691	0	275
652	0	109	662	0	111	672	0	111	682	0	275	692	0	275
653	0	109	663	0	111	673	1	277	683	0	275	693	0	275
654	0	111	664	0	111	674	0	276	684	0	275	694	0	275
655	0	111	665	0	111	675	0	276	685	0	0	695	0	275
656	0	111	666	0	111	676	0	276	686	0	0	696	0	275
657	0	111	667	0	111	677	0	276	687	0	275	697	0	275
658	0	111	668	0	111	678	1	276	688	0	25	698	0	60
659	0	111	669	0	111	679	0	275	689	0	275	699	0	30
660	0	111	670	0	111	680	0	275	690	0	275	700	0	275

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
701	0	275	711	0	103	721	0	54	731	0	190	741	0	0
702	0	275	712	0	103	722	0	54	732	0	190	742	0	0
703	0	25	713	0	7	723	0	54	733	0	197	743	0	0
704	0	300	714	0	7	724	0	52	734	0	197	744	0	0
705	0	153	715	0	7	725	0	93	735	0	197	745	0	0
706	0	153	716	0	0	726	1	93	736	0	197	746	1	197
707	0	153	717	0	0	727	0	190	737	0	197	747	0	196
708	0	153	718	0	0	728	0	190	738	0	197	748	0	196
709	0	153	719	0	147	729	0	61	739	0	197	749	0	196
710	0	153	720	0	147	730	0	130	740	0	0	750	0	196

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
751	0	248	761	0	248	771	0	248	781	0	0	791	0	0
752	0	248	762	0	248	772	0	248	782	0	0	792	0	0
753	0	248	763	0	248	773	0	0	783	0	0	793	0	0
754	0	248	764	0	248	774	0	0	784	0	0	794	0	0
755	0	248	765	0	248	775	0	0	785	0	0	795	0	0
756	0	248	766	0	248	776	0	0	786	0	0	796	0	0
757	0	248	767	0	248	777	0	0	787	0	0	797	0	0
758	0	248	768	0	248	778	0	0	788	0	0	798	0	0
759	0	248	769	0	248	779	0	0	789	0	0	799	0	0
760	0	248	770	0	248	780	0	0	790	0	0	800	0	0

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
801	0	25	811	0	1	821	0	24	831	0	25	841	0	25
802	0	25	812	0	24	822	0	24	832	0	25	842	0	25
803	0	25	813	0	24	823	0	24	833	0	25	843	0	25
804	0	25	814	0	20	824	0	24	834	0	25	844	0	25
805	0	25	815	0	4	825	0	24	835	0	25	845	1	283
806	0	25	816	0	24	826	0	25	836	0	25	846	0	282
807	0	25	817	0	24	827	0	25	837	0	25	847	0	282
808	0	25	818	0	24	828	0	25	838	0	25	848	0	282
809	0	25	819	0	24	829	0	25	839	0	25	849	0	282
810	0	21	820	0	24	830	0	25	840	0	25	850	1	282

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
851	0	281	861	0	281	871	0	26	881	0	62	891	0	126
852	0	281	862	0	281	872	0	281	882	0	62	892	0	126
853	0	281	863	0	281	873	0	281	883	0	49	893	0	58
854	0	281	864	0	281	874	0	281	884	0	49	894	0	58
855	0	281	865	0	281	875	0	196	885	0	6	895	0	58
856	0	281	866	0	281	876	0	188	886	0	6	896	0	58
857	0	0	867	0	281	877	0	62	887	0	6	897	0	68
858	0	0	868	0	281	878	0	62	888	0	0	898	1	68
859	0	281	869	0	281	879	0	62	889	0	0	899	0	110
860	0	106	870	0	48	880	0	62	890	0	0	900	0	110

BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
901	0	52	911	0	116	921	0	115	931	0	173	941	0	173
902	0	58	912	0	0	922	0	115	932	0	173	942	0	173
903	0	110	913	0	0	923	0	173	933	0	173	943	0	173
904	0	110	914	0	0	924	0	173	934	0	173	944	0	173
905	0	116	915	0	0	925	0	173	935	0	173	945	0	0
906	0	116	916	0	0	926	0	173	936	0	173	946	0	0
907	0	116	917	0	0	927	0	173	937	0	173	947	0	0
908	0	116	918	1	116	928	0	173	938	0	173	948	0	0
909	0	116	919	0	115	929	0	173	939	0	173	949	0	0
910	0	116	920	0	115	930	0	173	940	0	173	950	0	0
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
951	0	0	961	0	0	971	0	0	981	0	106	991	0	100
952	0	0	962	0	0	972	0	0	982	0	49	992	0	100
953	0	0	963	0	0	973	0	106	983	0	6	993	1	100
954	0	0	964	0	0	974	0	106	984	0	100	994	0	99
955	0	0	965	0	0	975	0	106	985	0	100	995	0	99
956	0	0	966	0	0	976	0	106	986	0	43	996	0	99
957	0	0	967	0	0	977	0	106	987	0	57	997	0	99
958	0	0	968	0	0	978	0	106	988	0	100	998	0	105
959	0	0	969	0	0	979	0	106	989	0	100	999	0	105
960	0	0	970	0	0	980	0	106	990	0	100	1000	0	105
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
1001	0	105	1011	0	105	1021	0	341	1031	0	340	1041	0	340
1002	0	105	1012	0	105	1022	1	341	1032	0	94	1042	0	79
1003	0	105	1013	0	105	1023	0	340	1033	0	340	1043	0	50
1004	0	105	1014	0	105	1024	0	340	1034	0	340	1044	0	340
1005	0	105	1015	0	105	1025	0	340	1035	0	340	1045	0	340
1006	0	105	1016	0	105	1026	0	340	1036	0	340	1046	0	340
1007	0	105	1017	1	342	1027	0	340	1037	0	340	1047	0	94
1008	0	105	1018	0	341	1028	0	340	1038	0	340	1048	0	287
1009	0	105	1019	0	341	1029	0	0	1039	0	340	1049	0	138
1010	0	105	1020	0	341	1030	0	0	1040	0	340	1050	0	138
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
1051	0	138	1061	0	0	1071	0	168	1081	0	180	1091	0	180
1052	0	138	1062	0	0	1072	0	169	1082	0	180	1092	0	180
1053	0	138	1063	0	149	1073	0	55	1083	0	180	1093	0	180
1054	0	138	1064	0	149	1074	0	115	1084	0	0	1094	0	180
1055	0	97	1065	0	68	1075	0	168	1085	0	0	1095	0	246
1056	0	97	1066	0	68	1076	0	168	1086	0	0	1096	0	246
1057	0	13	1067	0	68	1077	0	180	1087	0	0	1097	0	246
1058	0	12	1068	0	66	1078	0	180	1088	0	0	1098	0	246
1059	0	12	1069	0	81	1079	0	180	1089	0	0	1099	0	246
1060	0	0	1070	0	81	1080	0	180	1090	0	180	1100	0	246
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
1101	0	246	1111	0	246	1121	0	0	1131	0	0	1141	0	0
1102	0	246	1112	0	246	1122	0	0	1132	0	0	1142	0	0
1103	0	246	1113	0	246	1123	0	0	1133	0	0	1143	0	0
1104	0	246	1114	0	246	1124	0	0	1134	0	0	1144	0	0
1105	0	246	1115	0	246	1125	0	0	1135	0	0	1145	0	94
1106	0	246	1116	0	246	1126	0	0	1136	0	0	1146	0	94
1107	0	246	1117	0	0	1127	0	0	1137	0	0	1147	0	94
1108	0	246	1118	0	0	1128	0	0	1138	0	0	1148	0	94
1109	0	246	1119	0	0	1129	0	0	1139	0	0	1149	0	94
1110	0	246	1120	0	0	1130	0	0	1140	0	0	1150	0	94
BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL	BLOCK	CURRENT	TOTAL
1151	0	94	1161	0	85	1171	0	90	1181	0	90			
1152	0	94	1162	0	85	1172	0	90	1182	1	90			
1153	0	94	1163	0	85	1173	0	90	1183	0	89			
1154	0	53	1164	0	85	1174	0	90	1184	0	89			
1155	0	6	1165	0	85	1175	0	90	1185	0	89			
1156	0	68	1166	0	85	1176	0	90	1186	0	89			
1157	0	68	1167	0	85	1177	0	90	1187	0	89			
1158	0	47	1168	1	85	1178	0	90	1188	0	89			
1159	0	41	1169	0	84	1179	0	90	1189	0	1			
1160	0	85	1170	0	90	1180	0	90	1190	0	1			

APPENDIX G

OUTPUTS OF TRAFFIC QUEUES

STATISTICS OF TRAFFIC QUEUE IN LANE 11

TABLE 1

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	1.864	1.289	110.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	4	6.77	6.7	93.2	-.000	-1.446
1	25	42.37	49.1	50.8	.536	-.670
2	17	28.81	77.9	22.0	1.072	.105
3	5	8.47	86.4	13.5	1.609	.880
4	4	6.77	93.2	6.7	2.145	1.656
5	4	6.77	100.0	.0	2.681	2.432

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 12

TABLE 4

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	1.983	1.601	117.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	11	18.64	18.6	81.3	-.000	-1.238
1	16	27.11	45.7	54.2	.504	-.613
2	15	25.42	71.1	28.8	1.008	.010
3	3	5.08	76.2	23.7	1.512	.634
4	8	13.55	89.8	10.1	2.017	1.259
5	6	10.16	100.0	.0	2.521	1.883

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 13

TABLE 7

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
60	1.799	1.445	108.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	14	23.33	23.3	76.6	-.000	-1.245
1	13	21.66	44.9	55.0	.555	-.553
2	14	23.33	68.3	31.6	1.111	.138
3	13	21.66	89.9	10.0	1.666	.830
4	2	3.33	93.3	6.6	2.222	1.522
5	4	6.66	100.0	.0	2.777	2.214

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 21

TABLE 8

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	3.355	1.945	198.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	4	6.77	6.7	93.2	-.000	-1.725
1	8	13.55	20.3	79.6	.297	-1.211
2	9	15.25	35.5	64.4	.595	-.697
3	10	16.94	52.5	47.4	.893	-.182
4	10	16.94	69.4	30.5	1.191	.331
5	9	15.25	84.7	15.2	1.489	.845
6	7	11.86	96.6	3.3	1.787	1.359
7	1	1.69	98.3	1.6	2.085	1.873
8	1	1.69	100.0	.0	2.383	2.387

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 22

TABLE 11

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	2.457	1.289	145.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	0	.00	.0	100.0	-.000	-1.906
1	14	23.72	23.7	76.2	.406	-1.130
2	21	35.59	59.3	40.6	.813	-.355
3	15	25.42	84.7	15.2	1.220	.420
4	3	5.08	89.8	10.1	1.627	1.196
5	4	6.77	96.6	3.3	2.034	1.972
6	2	3.38	100.0	.0	2.441	2.748

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 23

TABLE 14

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
60	.383	.613	23.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	40	66.66	66.6	33.3	-.000	-.625
1	18	29.99	96.6	3.3	2.608	1.005
2	1	1.66	98.3	1.6	5.217	2.637
3	1	1.66	100.0	.0	7.826	4.268

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 31

TABLE 15

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	1.915	1.148	113.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	4	6.77	6.7	93.2	-.000	-1.667
1	20	33.89	40.6	59.3	.522	-.796
2	18	30.50	71.1	28.8	1.044	.073
3	13	22.03	93.2	6.7	1.566	.944
4	3	5.08	98.3	1.6	2.088	1.815
5	0	.00	98.3	1.6	2.610	2.686
6	1	1.69	100.0	.0	3.132	3.556

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 32

TABLE 18

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	1.627	1.226	96.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	9	15.25	15.2	84.7	-.000	-1.326
1	22	37.28	52.5	47.4	.614	-.511
2	18	30.50	83.0	16.9	1.229	.304
3	4	6.77	89.8	10.1	1.843	1.119
4	4	6.77	96.6	3.3	2.458	1.934
5	2	3.38	100.0	.0	3.072	2.749

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 33

TABLE 21

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
60	1.533	1.371	92.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	16	26.66	26.6	73.3	-.000	-1.118
1	19	31.66	58.3	41.6	.652	-.388
2	10	16.66	74.9	25.0	1.304	.340
3	8	13.33	88.3	11.6	1.956	1.069
4	6	9.99	98.3	1.6	2.608	1.799
5	1	1.66	100.0	.0	3.260	2.528

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 41

TABLE 22

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	2.881	1.722	170.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	7	11.86	11.8	88.1	-.000	-1.672
1	6	10.16	22.0	77.9	.347	-1.092
2	12	20.33	42.3	57.6	.694	-.511
3	10	16.94	59.3	40.6	1.041	.068
4	13	22.03	81.3	18.6	1.388	.649
5	9	15.25	96.6	3.3	1.735	1.229
6	1	1.69	98.3	1.6	2.082	1.810
7	1	1.69	100.0	.0	2.429	2.390

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 42

TABLE 25

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
59	2.135	1.464	126.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	7	11.86	11.8	88.1	-.000	-1.457
1	15	25.42	37.2	62.7	.468	-.775
2	13	22.03	59.3	40.6	.936	-.092
3	17	28.81	88.1	11.8	1.404	.590
4	4	6.77	94.9	5.0	1.873	1.272
5	1	1.69	96.6	3.3	2.341	1.955
6	1	1.69	98.3	1.6	2.809	2.638
7	1	1.69	100.0	.0	3.277	3.320

REMAINING FREQUENCIES ARE ALL ZERO

STATISTICS OF TRAFFIC QUEUE IN LANE 43

TABLE 28

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
60	1.399	1.136	84.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
0	13	21.66	21.6	78.3	-.000	-1.231
1	23	38.33	59.9	40.0	.714	-.351
2	15	25.00	84.9	15.0	1.428	.527
3	6	9.99	94.9	5.0	2.142	1.407
4	2	3.33	98.3	1.6	2.857	2.287
5	1	1.66	100.0	.0	3.571	3.167

REMAINING FREQUENCIES ARE ALL ZERO

APPENDIX H

OUTPUTS OF THROUGH TRAFFIC TRAVELING TIME

THROUGH TRAFFIC TRAVELLING TIME IN LANE 11

TABLE 2

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
109	19.055	12.390	2077.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.209	-1.215
8	37	33.94	33.9	66.0	.419	-.892
12	11	10.09	44.0	55.9	.629	-.569
16	6	5.50	49.5	50.4	.839	-.246
20	5	4.58	54.1	45.8	1.049	.076
24	7	6.42	60.5	39.4	1.259	.399
28	13	11.92	72.4	27.5	1.469	.721
32	11	10.09	82.5	17.4	1.679	1.044
36	8	7.33	89.9	10.0	1.889	1.367
40	6	5.50	95.4	4.5	2.099	1.690
44	4	3.66	99.0	.9	2.309	2.013
48	1	.91	100.0	.0	2.519	2.336

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 12

TABLE 5

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
205	21.536	14.378	4415.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.185	-1.219
8	55	26.82	26.8	73.1	.371	-.941
12	13	6.34	33.1	66.8	.557	-.663
16	25	12.19	45.3	54.6	.742	-.385
20	11	5.36	50.7	49.2	.928	-.106
24	21	10.24	60.9	39.0	1.114	.171
28	18	8.78	69.7	30.2	1.300	.449
32	12	5.85	75.6	24.3	1.485	.727
36	14	6.82	82.4	17.5	1.671	1.005
40	13	6.34	88.7	11.2	1.857	1.284
44	9	4.39	93.1	6.8	2.043	1.562
48	4	1.95	95.1	4.8	2.228	1.840
52	0	.00	95.1	4.8	2.414	2.118
56	10	4.87	100.0	.0	2.600	2.396

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 21

TABLE 9

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
212	26.622	13.664	5644.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.150	-1.655
8	15	7.07	7.0	92.9	.300	-1.362
12	27	12.73	19.8	80.1	.450	-1.070
16	19	8.96	28.7	71.2	.600	-.777
20	18	8.49	37.2	62.7	.751	-.484
24	19	8.96	46.2	53.7	.901	-.191
28	26	12.26	58.4	41.5	1.051	.100
32	17	8.01	66.5	33.4	1.201	.393
36	16	7.54	74.0	25.9	1.352	.686
40	12	5.66	79.7	20.2	1.502	.979
44	10	4.71	84.4	15.5	1.652	1.271
48	18	8.49	92.9	7.0	1.802	1.564
52	13	6.13	99.0	.9	1.953	1.857
56	2	.94	100.0	.0	2.103	2.149

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 22

TABLE 12

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
248	25.983	13.246	6444.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.153	-1.659
8	7	2.82	2.8	97.1	.307	-1.357
12	45	18.14	20.9	79.0	.461	-1.055
16	23	9.27	30.2	69.7	.615	-.753
20	26	10.48	40.7	59.2	.769	-.451
24	23	9.27	49.9	50.0	.923	-.149
28	24	9.67	59.6	40.3	1.077	.152
32	18	7.25	66.9	33.0	1.231	.454
36	14	5.64	72.5	27.4	1.385	.756
40	27	10.88	83.4	16.5	1.539	1.058
44	12	4.83	88.3	11.6	1.693	1.360
48	13	5.24	93.5	6.4	1.847	1.662
52	14	5.64	99.1	.8	2.001	1.964
56	2	.80	100.0	.0	2.155	2.266

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 31

TABLE 18

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
150	22.106	13.074	3316.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.180	-1.364
8	43	28.66	28.6	71.3	.361	-1.078
12	7	4.66	33.3	66.6	.542	-.773
16	11	7.33	40.6	59.3	.723	-.467
20	12	7.99	48.6	51.3	.904	-.161
24	12	7.99	56.6	43.3	1.085	.144
28	10	6.66	63.3	36.6	1.266	.450
32	12	7.99	71.3	28.6	1.447	.756
36	16	10.66	81.9	18.0	1.628	1.062
40	14	9.33	91.3	8.6	1.809	1.368
44	8	5.33	96.6	3.3	1.990	1.674
48	5	3.33	100.0	.0	2.171	1.980

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 32

TABLE 19

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
173	21.456	13.687	3712.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.186	-1.275
8	51	29.47	29.4	70.5	.372	-.983
12	16	9.24	38.7	61.2	.559	-.690
16	10	5.78	44.5	55.4	.745	-.398
20	13	7.51	52.0	47.9	.932	-.106
24	8	4.62	56.6	43.3	1.118	.185
28	14	8.09	64.7	35.2	1.304	.478
32	16	9.24	73.9	26.0	1.491	.770
36	10	5.78	79.7	20.2	1.677	1.062
40	12	6.93	86.7	13.2	1.864	1.354
44	15	8.67	95.3	4.6	2.050	1.647
48	8	4.62	100.0	.0	2.237	1.939

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 41

TABLE 23

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
247	23.825	12.570	5885.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.167	-1.577
8	5	2.02	2.0	97.9	.335	-1.258
12	50	20.24	22.2	77.7	.503	-.940
16	40	16.19	38.4	61.5	.671	-.622
20	21	8.50	46.9	53.0	.839	-.304
24	28	11.33	58.2	41.7	1.007	.013
28	27	10.93	69.2	30.7	1.175	.332
32	11	4.45	73.6	26.3	1.343	.650
36	14	5.66	79.3	20.6	1.510	.968
40	15	6.07	85.4	14.5	1.678	1.286
44	17	6.88	92.3	7.6	1.846	1.604
48	8	3.23	95.5	4.4	2.014	1.923
52	9	3.64	99.1	.8	2.182	2.241
56	2	.80	100.0	.0	2.350	2.559

REMAINING FREQUENCIES ARE ALL ZERO

THROUGH TRAFFIC TRAVELLING TIME IN LANE 42

TABLE 26

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
246	24.784	13.292	6097.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.161	-1.563
8	17	6.91	6.9	93.0	.322	-1.262
12	53	21.54	28.4	71.5	.484	-.961
16	21	8.53	36.9	63.0	.645	-.660
20	19	7.72	44.7	55.2	.806	-.359
24	18	7.31	52.0	47.9	.968	-.059
28	16	6.50	58.5	41.4	1.129	.241
32	17	6.91	65.4	34.5	1.291	.542
36	27	10.97	76.4	23.5	1.452	.843
40	25	10.16	86.5	13.4	1.613	1.144
44	14	5.69	92.2	7.7	1.775	1.445
48	8	3.25	95.5	4.4	1.936	1.746
52	8	3.25	98.7	1.2	2.098	2.047
56	2	.81	99.5	.4	2.259	2.348
60	1	.40	100.0	.0	2.420	2.649

REMAINING FREQUENCIES ARE ALL ZERO

APPENDIX I

OUTPUTS OF RIGHT TURN TRAFFIC TRAVELING TIME

RIGHT-TURN TRAFFIC TRAVELLING TIME IN LANE 11

TABLE 3

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
79	19.949	13.167	1576.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	3	3.79	3.7	96.2	.200	-1.211
8	22	27.84	31.6	68.3	.401	-.907
12	7	8.86	40.5	59.4	.601	-.603
16	8	10.12	50.6	49.3	.802	-.299
20	5	6.32	56.9	43.0	1.002	.003
24	1	1.26	58.2	41.7	1.203	.307
28	6	7.59	65.8	34.1	1.403	.611
32	9	11.39	77.2	22.7	1.604	.915
36	9	11.39	88.6	11.3	1.804	1.218
40	3	3.79	92.4	7.5	2.005	1.522
44	4	5.06	97.4	2.5	2.205	1.826
48	2	2.53	100.0	.0	2.406	2.130

REMAINING FREQUENCIES ARE ALL ZERO

RIGHT-TURN TRAFFIC TRAVELLING TIME IN LANE 21

TABLE 10

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
118	27.864	13.160	3288.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.143	-1.813
8	10	8.47	8.4	91.5	.287	-1.509
12	10	8.47	16.9	83.0	.430	-1.205
16	13	11.01	27.9	72.0	.574	-.901
20	8	6.77	34.7	65.2	.717	-.597
24	8	6.77	41.5	58.4	.861	-.293
28	11	9.32	50.8	49.1	1.004	.010
32	11	9.32	60.1	39.8	1.148	.314
36	13	11.01	71.1	28.8	1.291	.618
40	9	7.62	78.8	21.1	1.435	.922
44	10	8.47	87.2	12.7	1.579	1.226
48	9	7.62	94.9	5.0	1.722	1.530
52	6	5.08	100.0	.0	1.866	1.833

REMAINING FREQUENCIES ARE ALL ZERO

RIGHT-TURN TRAFFIC TRAVELLING TIME IN LANE 31

TABLE 17

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
28	23.928	13.960	670.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.167	-1.427
8	8	28.57	28.5	71.4	.334	-1.140
12	1	3.57	32.1	67.8	.501	-.854
16	0	.00	32.1	67.8	.668	-.567
20	2	7.14	39.2	60.7	.835	-.281
24	4	14.28	53.5	46.4	1.002	.005
28	2	7.14	60.7	39.2	1.170	.291
32	3	10.71	71.4	28.5	1.337	.578
36	1	3.57	74.9	25.0	1.504	.864
40	3	10.71	85.7	14.2	1.671	1.151
44	2	7.14	92.8	7.1	1.838	1.437
48	2	7.14	100.0	.0	2.005	1.724

REMAINING FREQUENCIES ARE ALL ZERO

RIGHT-TURN TRAFFIC TRAVELLING TIME IN LANE 41

TABLE 24

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
93	22.752	11.949	2116.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.175	-1.569
8	1	1.07	1.0	98.9	.351	-1.234
12	30	32.25	33.3	66.6	.527	-.899
16	8	8.60	41.9	58.0	.703	-.565
20	9	9.67	51.6	48.3	.879	-.230
24	9	9.67	61.2	38.7	1.054	.104
28	7	7.52	68.8	31.1	1.230	.439
32	8	8.60	77.4	22.5	1.406	.773
36	5	5.37	82.7	17.2	1.582	1.108
40	8	8.60	91.3	8.6	1.758	1.443
44	3	3.22	94.6	5.3	1.933	1.778
48	3	3.22	97.8	2.1	2.109	2.112
52	2	2.15	100.0	.0	2.285	2.447

REMAINING FREQUENCIES ARE ALL ZERO

APPENDIX J

OUTPUTS OF LEFT TURN TRAFFIC TRAVELING TIME

LEFT-TURN TRAFFIC TRAVELLING TIME FROM LANE 12

TABLE 6

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
111	37.027	17.312	4110.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.108	-1.907
8	2	1.80	1.8	98.1	.216	-1.676
12	6	5.40	7.2	92.7	.324	-1.445
16	5	4.50	11.7	88.2	.432	-1.214
20	10	9.00	20.7	79.2	.540	-.983
24	7	6.30	27.0	72.9	.648	-.752
28	6	5.40	32.4	67.5	.756	-.521
32	12	10.81	43.2	56.7	.864	-.290
36	7	6.30	49.5	50.4	.972	-.059
40	8	7.20	56.7	43.2	1.080	.171
44	11	9.90	66.6	33.3	1.188	.402
48	9	8.10	74.7	25.2	1.296	.633
52	7	6.30	81.0	18.9	1.404	.864
56	6	5.40	86.4	13.5	1.512	1.095
60	4	3.60	90.0	9.9	1.620	1.326
64	3	2.70	92.7	7.2	1.728	1.558
68	5	4.50	97.2	2.7	1.836	1.789
72	2	1.80	99.0	.9	1.944	2.020
76	0	.00	99.0	.9	2.052	2.251
80	0	.00	99.0	.9	2.160	2.482
84	0	.00	99.0	.9	2.268	2.713
88	0	.00	99.0	.9	2.376	2.944
92	1	.90	100.0	.0	2.484	3.175

REMAINING FREQUENCIES ARE ALL ZERO

LEFT-TURN TRAFFIC TRAVELLING TIME FROM LANE 22

TABLE 13

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
25	39.199	21.625	980.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.102	-1.627
8	0	.00	.0	100.0	.204	-1.442
12	1	3.99	3.9	96.0	.306	-1.257
16	4	15.99	19.9	80.0	.408	-1.072
20	2	7.99	27.9	72.0	.510	-.887
24	2	7.99	35.9	64.0	.612	-.702
28	2	7.99	43.9	56.0	.714	-.517
32	1	3.99	47.9	52.0	.816	-.332
36	1	3.99	51.9	48.0	.918	-.147
40	1	3.99	55.9	44.0	1.020	.036
44	1	3.99	59.9	40.0	1.122	.221
48	1	3.99	63.9	36.0	1.224	.406
52	1	3.99	67.9	32.0	1.326	.591
56	1	3.99	71.9	28.0	1.428	.776
60	0	.00	71.9	28.0	1.530	.961
64	1	3.99	75.9	24.0	1.632	1.146
68	3	11.99	87.9	12.0	1.734	1.331
72	2	7.99	95.9	4.0	1.836	1.516
76	1	3.99	100.0	.0	1.938	1.701

REMAINING FREQUENCIES ARE ALL ZERO

LEFT-TURN TRAFFIC TRAVELLING TIME FROM LANE 32

TABLE 20

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
105	35.961	18.187	3776.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.111	-1.757
8	5	4.76	4.7	95.2	.222	-1.537
12	7	6.66	11.4	88.5	.333	-1.317
16	6	5.71	17.1	82.8	.444	-1.097
20	5	4.76	21.9	78.0	.556	-.877
24	10	9.52	31.4	68.5	.667	-.657
28	13	12.38	43.8	56.1	.778	-.437
32	3	2.85	46.5	53.3	.889	-.217
36	5	4.76	51.4	48.5	1.001	.002
40	8	7.61	59.0	40.9	1.112	.222
44	6	5.71	64.7	35.2	1.223	.441
48	9	8.57	73.3	26.6	1.334	.661
52	6	5.71	79.0	20.9	1.445	.881
56	3	2.85	81.9	18.0	1.557	1.101
60	10	9.52	91.4	8.5	1.668	1.321
64	3	2.85	94.2	5.7	1.779	1.541
68	3	2.85	97.1	2.8	1.890	1.761
72	1	.95	98.0	1.9	2.002	1.981
76	1	.95	99.0	.9	2.113	2.201
80	0	.00	99.0	.9	2.224	2.421
84	0	.00	99.0	.9	2.335	2.641
88	1	.95	100.0	.0	2.447	2.861

REMAINING FREQUENCIES ARE ALL ZERO

LEFT-TURN TRAFFIC TRAVELLING TIME FROM LANE 42

TABLE 27

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
89	38.887	16.687	3461.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	0	.00	.0	100.0	.102	-2.090
8	0	.00	.0	100.0	.205	-1.850
12	5	5.61	5.6	94.3	.308	-1.611
16	4	4.49	10.1	89.8	.411	-1.371
20	5	5.61	15.7	84.2	.514	-1.131
24	9	10.11	25.8	74.1	.617	-.892
28	6	6.74	32.5	67.4	.720	-.652
32	3	3.37	35.9	64.0	.822	-.412
36	9	10.11	46.0	53.9	.925	-.173
40	7	7.86	53.9	46.0	1.028	.066
44	4	4.49	58.4	41.5	1.131	.306
48	6	6.74	65.1	34.8	1.234	.546
52	4	4.49	69.6	30.3	1.337	.785
56	12	13.48	83.1	16.8	1.440	1.025
60	7	7.86	91.0	8.9	1.542	1.265
64	4	4.49	95.5	4.4	1.645	1.504
68	1	1.12	96.6	3.3	1.748	1.744
72	3	3.37	100.0	.0	1.851	1.984

REMAINING FREQUENCIES ARE ALL ZERO

APPENDIX K

OUTPUTS OF VEHICLE ARRIVAL-TIME DISTRIBUTION

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 11

TABLE 29

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
188	18.457	16.875	3470.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	36	19.14	19.1	80.8	.216	-.856
8	33	17.55	36.7	63.2	.433	-.619
12	22	11.70	48.4	51.5	.650	-.382
16	20	10.63	59.0	40.9	.866	-.145
20	15	7.97	67.0	32.9	1.083	.091
24	11	5.85	72.8	27.1	1.300	.328
28	5	2.65	75.5	24.4	1.517	.565
32	11	5.85	81.3	18.6	1.733	.802
36	8	4.25	85.6	14.3	1.950	1.039
40	7	3.72	89.3	10.6	2.167	1.276
44	5	2.65	92.0	7.9	2.383	1.513
48	4	2.12	94.1	5.8	2.600	1.750
52	1	.53	94.6	5.3	2.817	1.987
56	1	.53	95.2	4.7	3.034	2.224
60	3	1.59	96.8	3.1	3.250	2.461
64	0	.00	96.8	3.1	3.467	2.698
68	3	1.59	98.4	1.5	3.684	2.935
72	1	.53	98.9	1.0	3.900	3.172
76	0	.00	98.9	1.0	4.117	3.409
80	0	.00	98.9	1.0	4.334	3.646
84	2	1.06	100.0	.0	4.551	3.884

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 12

TABLE 30

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
319	10.824	10.449	3453.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	112	35.10	35.1	64.8	.369	-.653
8	65	20.37	55.4	44.5	.739	-.270
12	31	9.71	65.2	34.7	1.108	.112
16	48	15.04	80.2	19.7	1.478	.495
20	19	5.95	86.2	13.7	1.847	.878
24	15	4.70	90.9	9.0	2.217	1.260
28	8	2.50	93.4	6.5	2.586	1.643
32	4	1.25	94.6	5.3	2.956	2.026
36	5	1.56	96.2	3.7	3.325	2.409
40	4	1.25	97.4	2.5	3.695	2.792
44	3	.94	98.4	1.5	4.064	3.174
48	1	.31	98.7	1.2	4.434	3.557
52	1	.31	99.0	.9	4.803	3.940
56	2	.62	99.6	.3	5.173	4.323
60	0	.00	99.6	.3	5.543	4.706
64	0	.00	99.6	.3	5.912	5.088
68	0	.00	99.6	.3	6.282	5.471
72	1	.31	100.0	.0	6.651	5.854

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 21

TABLE 31

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
330	10.406	9.988	3434.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	131	39.69	39.6	60.3	.384	-.641
8	65	19.69	59.3	40.6	.768	-.240
12	25	7.57	66.9	33.0	1.153	.159
16	41	12.42	79.3	20.6	1.537	.560
20	16	4.84	84.2	15.7	1.921	.960
24	24	7.27	91.5	8.4	2.306	1.360
28	9	2.72	94.2	5.7	2.690	1.761
32	4	1.21	95.4	4.5	3.075	2.161
36	7	2.12	97.5	2.4	3.459	2.562
40	3	.90	98.4	1.5	3.843	2.962
44	1	.30	98.7	1.2	4.228	3.363
48	1	.30	99.0	.9	4.612	3.763
52	2	.60	99.6	.3	4.997	4.164
56	0	.00	99.6	.3	5.381	4.564
60	1	.30	100.0	.0	5.765	4.965

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 22

TABLE 32

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
275	12.603	12.179	3466.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	77	27.99	27.9	72.0	.317	-.706
8	50	18.18	46.1	53.8	.634	-.377
12	44	15.99	62.1	37.8	.952	-.049
16	27	9.81	71.9	28.0	1.269	.278
20	27	9.81	81.8	18.1	1.586	.607
24	14	5.09	86.9	13.0	1.904	.935
28	9	3.27	90.1	9.8	2.221	1.264
32	10	3.63	93.8	6.1	2.538	1.592
36	3	1.09	94.9	5.0	2.856	1.920
40	4	1.45	96.3	3.6	3.173	2.249
44	2	.72	97.0	2.9	3.491	2.577
48	0	.00	97.0	2.9	3.808	2.906
52	3	1.09	98.1	1.8	4.125	3.234
56	2	.72	98.9	1.0	4.443	3.563
60	1	.36	99.2	.7	4.760	3.891
64	0	.00	99.2	.7	5.077	4.219
68	1	.36	99.6	.3	5.395	4.548
72	0	.00	99.6	.3	5.712	4.876
76	0	.00	99.6	.3	6.030	5.205
80	1	.36	100.0	.0	6.347	5.533

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 31

TABLE 33

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
179	19.597	17.250	3508.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	30	16.75	16.7	83.2	.204	-.904
8	26	14.52	31.2	68.7	.408	-.672
12	31	17.31	48.6	51.3	.612	-.440
16	13	7.26	55.8	44.1	.816	-.208
20	11	6.14	62.0	37.9	1.020	.023
24	16	8.93	70.9	29.0	1.224	.255
28	9	5.02	75.9	24.0	1.428	.487
32	11	6.14	82.1	17.8	1.632	.718
36	5	2.79	84.9	15.0	1.836	.950
40	6	3.35	88.2	11.7	2.041	1.182
44	5	2.79	91.0	8.9	2.245	1.414
48	3	1.67	92.7	7.2	2.449	1.646
52	3	1.67	94.4	5.5	2.653	1.878
56	3	1.67	96.0	3.9	2.857	2.110
60	1	.55	96.6	3.3	3.061	2.342
64	0	.00	96.6	3.3	3.265	2.574
68	0	.00	96.6	3.3	3.469	2.805
72	2	1.11	97.7	2.2	3.673	3.037
76	0	.00	97.7	2.2	3.877	3.269
80	4	2.23	100.0	.0	4.082	3.501

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 32

TABLE 34

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
281	12.370	11.761	3476.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	77	27.40	27.4	72.5	.323	-.711
8	58	20.64	48.0	51.9	.646	-.371
12	46	16.37	64.4	35.5	.970	-.031
16	31	11.03	75.4	24.5	1.293	.308
20	25	8.89	84.3	15.6	1.616	.648
24	9	3.20	87.5	12.4	1.940	.988
28	7	2.49	90.0	9.9	2.263	1.328
32	9	3.20	93.2	6.7	2.586	1.668
36	5	1.77	95.0	4.9	2.910	2.009
40	2	.71	95.7	4.2	3.233	2.349
44	2	.71	96.4	3.5	3.556	2.689
48	3	1.06	97.5	2.4	3.880	3.029
52	4	1.42	98.9	1.0	4.203	3.369
56	1	.35	99.2	.7	4.527	3.709
60	0	.00	99.2	.7	4.850	4.049
64	0	.00	99.2	.7	5.173	4.389
68	0	.00	99.2	.7	5.497	4.729
72	2	.71	100.0	.0	5.820	5.069

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 41

TABLE 35

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
343	10.064	10.218	3452.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	131	38.19	38.1	61.8	.397	-.593
8	66	19.24	57.4	42.5	.794	-.201
12	49	14.28	71.7	28.2	1.192	.189
16	40	11.66	83.3	16.6	1.589	.580
20	13	3.79	87.1	12.8	1.987	.972
24	14	4.08	91.2	8.7	2.384	1.363
28	9	2.62	93.8	6.1	2.782	1.755
32	5	1.45	95.3	4.6	3.179	2.146
36	6	1.74	97.0	2.9	3.577	2.538
40	0	.00	97.0	2.9	3.974	2.929
44	2	.58	97.6	2.3	4.371	3.320
48	5	1.45	99.1	.8	4.769	3.712
52	1	.29	99.4	.5	5.166	4.103
56	1	.29	99.7	.2	5.564	4.495
60	0	.00	99.7	.2	5.961	4.886
64	1	.29	100.0	.0	6.359	5.278

REMAINING FREQUENCIES ARE ALL ZERO

ARRIVAL-TIME DISTRIBUTION FOR VEHICLES IN LANE 42

TABLE 36

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
340	10.099	9.183	3434.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
4	116	34.11	34.1	65.8	.396	-.664
8	73	21.47	55.5	44.4	.792	-.228
12	52	15.29	70.8	29.1	1.188	.206
16	31	9.11	79.9	20.0	1.584	.642
20	21	6.17	86.1	13.8	1.980	1.078
24	16	4.70	90.8	9.1	2.376	1.513
28	13	3.82	94.7	5.2	2.772	1.949
32	7	2.05	96.7	3.2	3.168	2.384
36	3	.88	97.6	2.3	3.564	2.820
40	4	1.17	98.8	1.1	3.960	3.255
44	3	.88	99.7	.2	4.356	3.691
48	1	.29	100.0	.0	4.752	4.126

REMAINING FREQUENCIES ARE ALL ZERO

APPENDIX L

OUTPUTS OF VEHICLE SPEED DISTRIBUTION

SPEED DISTRIBUTION OF VEHICLES IN LANE 11

TABLE 37

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
188	22.547	5.562	4239.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.088	-3.693
4	0	.00	.0	100.0	.177	-3.334
6	0	.00	.0	100.0	.266	-2.974
8	0	.00	.0	100.0	.354	-2.615
10	0	.00	.0	100.0	.443	-2.255
12	7	3.72	3.7	96.2	.532	-1.896
14	9	4.78	8.5	91.4	.620	-1.536
16	14	7.44	15.9	84.0	.709	-1.177
18	13	6.91	22.8	77.1	.798	-.817
20	24	12.76	35.6	64.3	.887	-.458
22	34	18.08	53.7	46.2	.975	-.098
24	26	13.82	67.5	32.4	1.064	.261
26	8	4.25	71.8	28.1	1.153	.620
28	12	6.38	78.1	21.8	1.241	.980
30	23	12.23	90.4	9.5	1.330	1.339
32	18	9.57	100.0	.0	1.419	1.699

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 12

TABLE 38

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
319	25.354	4.996	8088.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.078	-4.674
4	0	.00	.0	100.0	.157	-4.274
6	0	.00	.0	100.0	.236	-3.873
8	0	.00	.0	100.0	.315	-3.473
10	0	.00	.0	100.0	.394	-3.073
12	5	1.56	1.5	98.4	.473	-2.672
14	3	.94	2.5	97.4	.552	-2.272
16	13	4.07	6.5	93.4	.631	-1.872
18	8	2.50	9.0	90.9	.709	-1.471
20	27	8.46	17.5	82.4	.788	-1.071
22	44	13.79	31.3	68.6	.867	-.671
24	36	11.28	42.6	57.3	.946	-.271
26	17	5.32	47.9	52.0	1.025	.129
28	54	16.92	64.8	35.1	1.104	.529
30	66	20.69	85.5	14.4	1.183	.929
32	46	14.42	100.0	.0	1.262	1.330

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 21

TABLE 39

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
330	38.475	7.890	12697.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.051	-4.622
4	0	.00	.0	100.0	.103	-4.369
6	0	.00	.0	100.0	.155	-4.115
8	0	.00	.0	100.0	.207	-3.862
10	0	.00	.0	100.0	.259	-3.608
12	0	.00	.0	100.0	.311	-3.355
14	0	.00	.0	100.0	.363	-3.101
16	0	.00	.0	100.0	.415	-2.848
18	0	.00	.0	100.0	.467	-2.594
20	0	.00	.0	100.0	.519	-2.341
22	0	.00	.0	100.0	.571	-2.088
24	0	.00	.0	100.0	.623	-1.834
26	39	11.81	11.8	88.1	.675	-1.581
28	13	3.93	15.7	84.2	.727	-1.327
30	11	3.33	19.0	80.9	.779	-1.074
32	17	5.15	24.2	75.7	.831	-.820
34	10	3.03	27.2	72.7	.883	-.567
36	46	13.93	41.2	58.7	.935	-.313
38	14	4.24	45.4	54.5	.987	-.060
40	35	10.60	56.0	43.9	1.039	.193
42	50	15.15	71.2	28.7	1.091	.446
44	7	2.12	73.3	26.6	1.143	.700
46	19	5.75	79.0	20.9	1.195	.953
48	38	11.51	90.6	9.3	1.247	1.207
50	24	7.27	97.8	2.1	1.299	1.460
52	2	.60	98.4	1.5	1.351	1.713
54	2	.60	99.0	.9	1.403	1.967
56	0	.00	99.0	.9	1.455	2.220
58	2	.60	99.6	.3	1.507	2.474
60	1	.30	100.0	.0	1.559	2.727

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 22

TABLE 40

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
275	38.552	7.808	10602.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.051	-4.681
4	0	.00	.0	100.0	.103	-4.424
6	0	.00	.0	100.0	.155	-4.168
8	0	.00	.0	100.0	.207	-3.912
10	0	.00	.0	100.0	.259	-3.656
12	0	.00	.0	100.0	.311	-3.400
14	0	.00	.0	100.0	.363	-3.144
16	0	.00	.0	100.0	.415	-2.888
18	0	.00	.0	100.0	.466	-2.632
20	0	.00	.0	100.0	.518	-2.375
22	0	.00	.0	100.0	.570	-2.119
24	0	.00	.0	100.0	.622	-1.863
26	29	10.54	10.5	89.4	.674	-1.607
28	8	2.90	13.4	86.5	.726	-1.351
30	11	3.99	17.4	82.5	.778	-1.095
32	7	2.54	19.9	80.0	.830	-.839
34	4	1.45	21.4	78.5	.881	-.583
36	52	18.90	40.3	59.6	.933	-.326
38	23	8.36	48.7	51.2	.985	-.070
40	25	9.09	57.8	42.1	1.037	.185
42	61	22.18	79.9	20.0	1.089	.441
44	9	3.27	83.2	16.7	1.141	.697
46	8	2.90	86.1	13.8	1.193	.953
48	7	2.54	88.7	11.2	1.245	1.209
50	5	1.81	90.5	9.4	1.296	1.465
52	8	2.90	93.4	6.5	1.348	1.722
54	9	3.27	96.7	3.2	1.400	1.978
56	4	1.45	98.1	1.8	1.452	2.234
58	4	1.45	99.6	.3	1.504	2.490
60	1	.36	100.0	.0	1.556	2.746

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 31

TABLE 41

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
179	34.050	7.527	6095.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.058	-4.257
4	0	.00	.0	100.0	.117	-3.992
6	0	.00	.0	100.0	.176	-3.726
8	0	.00	.0	100.0	.234	-3.460
10	0	.00	.0	100.0	.293	-3.195
12	0	.00	.0	100.0	.352	-2.929
14	0	.00	.0	100.0	.411	-2.663
16	0	.00	.0	100.0	.469	-2.397
18	0	.00	.0	100.0	.528	-2.132
20	0	.00	.0	100.0	.587	-1.866
22	25	13.96	13.9	86.0	.646	-1.600
24	6	3.35	17.3	82.6	.704	-1.335
26	8	4.46	21.7	78.2	.763	-1.069
28	7	3.91	25.6	74.3	.822	-.803
30	2	1.11	26.8	73.1	.881	-.538
32	6	3.35	30.1	69.8	.939	-.272
34	17	9.49	39.6	60.3	.998	-.006
36	34	18.99	58.6	41.3	1.057	.259
38	33	18.43	77.0	22.9	1.115	.524
40	11	6.14	83.2	16.7	1.174	.790
42	4	2.23	85.4	14.5	1.233	1.056
44	13	7.26	92.7	7.2	1.292	1.321
46	7	3.91	96.6	3.3	1.350	1.587
48	6	3.35	100.0	.0	1.409	1.853

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 32

TABLE 42

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
281	32.099	7.871	9020.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.062	-3.824
4	0	.00	.0	100.0	.124	-3.569
6	0	.00	.0	100.0	.186	-3.315
8	0	.00	.0	100.0	.249	-3.061
10	0	.00	.0	100.0	.311	-2.807
12	0	.00	.0	100.0	.373	-2.553
14	0	.00	.0	100.0	.436	-2.299
16	0	.00	.0	100.0	.498	-2.045
18	0	.00	.0	100.0	.560	-1.791
20	31	11.03	11.0	88.9	.623	-1.537
22	7	2.49	13.5	86.4	.685	-1.283
24	8	2.84	16.3	83.6	.747	-1.029
26	14	4.98	21.3	78.6	.809	-.774
28	20	7.11	28.4	71.5	.872	-.520
30	39	13.87	42.3	57.6	.934	-.266
32	23	8.18	50.5	49.4	.996	-.012
34	31	11.03	61.5	38.4	1.059	.241
36	41	14.59	76.1	23.8	1.121	.495
38	36	12.81	88.9	11.0	1.183	.749
40	5	1.77	90.7	9.2	1.246	1.003
42	4	1.42	92.1	7.9	1.308	1.257
44	1	.35	92.5	7.4	1.370	1.511
46	5	1.77	94.3	5.6	1.433	1.766
48	3	1.06	95.3	4.6	1.495	2.020
50	1	.35	95.7	4.2	1.557	2.274
52	4	1.42	97.1	2.8	1.619	2.528
54	6	2.13	99.2	.7	1.682	2.782
56	2	.71	100.0	.0	1.744	3.036

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 41

TABLE 43

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
343	33.032	7.421	11330.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.060	-4.181
4	0	.00	.0	100.0	.121	-3.911
6	0	.00	.0	100.0	.181	-3.642
8	0	.00	.0	100.0	.242	-3.372
10	0	.00	.0	100.0	.302	-3.103
12	0	.00	.0	100.0	.363	-2.833
14	0	.00	.0	100.0	.423	-2.564
16	0	.00	.0	100.0	.484	-2.294
18	0	.00	.0	100.0	.544	-2.025
20	0	.00	.0	100.0	.605	-1.755
22	0	.00	.0	100.0	.666	-1.486
24	0	.00	.0	100.0	.726	-1.216
26	84	24.49	24.4	75.5	.787	-.947
28	23	6.71	31.1	68.8	.847	-.678
30	42	12.24	43.4	56.5	.908	-.408
32	54	15.74	59.1	40.8	.968	-.139
34	13	3.79	62.9	37.0	1.029	.130
36	37	10.78	73.7	26.2	1.089	.399
38	21	6.12	79.8	20.1	1.150	.669
40	14	4.08	83.9	16.0	1.210	.938
42	16	4.66	88.6	11.3	1.271	1.208
44	4	1.16	89.7	10.2	1.332	1.477
46	8	2.33	92.1	7.8	1.392	1.747
48	14	4.08	96.2	3.7	1.453	2.016
50	6	1.74	97.9	2.0	1.513	2.286
52	1	.29	98.2	1.7	1.574	2.555
54	0	.00	98.2	1.7	1.634	2.825
56	3	.87	99.1	.8	1.695	3.094
58	0	.00	99.1	.8	1.755	3.364
60	3	.87	100.0	.0	1.816	3.633

REMAINING FREQUENCIES ARE ALL ZERO

SPEED DISTRIBUTION OF VEHICLES IN LANE 42

TABLE 44

ENTRIES IN TABLE	MEAN ARGUMENT	STANDARD DEVIATION	SUM OF ARGUMENTS	
340	39.149	8.187	13311.000	NON-WEIGHTED

UPPER LIMIT	OBSERVED FREQUENCY	PER CENT OF TOTAL	CUMULATIVE PERCENTAGE	CUMULATIVE REMAINDER	MULTIPLE OF MEAN	DEVIATION FROM MEAN
2	0	.00	.0	100.0	.051	-4.537
4	0	.00	.0	100.0	.102	-4.293
6	0	.00	.0	100.0	.153	-4.048
8	0	.00	.0	100.0	.204	-3.804
10	0	.00	.0	100.0	.255	-3.560
12	0	.00	.0	100.0	.306	-3.316
14	0	.00	.0	100.0	.357	-3.071
16	0	.00	.0	100.0	.408	-2.827
18	0	.00	.0	100.0	.459	-2.583
20	0	.00	.0	100.0	.510	-2.338
22	0	.00	.0	100.0	.561	-2.094
24	0	.00	.0	100.0	.613	-1.850
26	25	7.35	7.3	92.6	.664	-1.606
28	20	5.88	13.2	86.7	.715	-1.361
30	21	6.17	19.4	80.5	.766	-1.117
32	14	4.11	23.5	76.4	.817	-.873
34	11	3.23	26.7	73.2	.868	-.629
36	51	14.99	41.7	58.2	.919	-.384
38	17	4.99	46.7	53.2	.970	-.140
40	19	5.58	52.3	47.6	1.021	.103
42	42	12.35	64.7	35.2	1.072	.348
44	12	3.52	68.2	31.7	1.123	.592
46	29	8.52	76.7	23.2	1.174	.836
48	42	12.35	89.1	10.8	1.226	1.080
50	19	5.58	94.7	5.2	1.277	1.325
52	3	.88	95.5	4.4	1.328	1.569
54	3	.88	96.4	3.5	1.379	1.813
56	7	2.05	98.5	1.4	1.430	2.058
58	5	1.47	100.0	.0	1.481	2.302

REMAINING FREQUENCIES ARE ALL ZERO

**** OUTPUT EDITOR REQUEST CARDS

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VITA

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