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TRANSPORTATION
INSTITUTE

STATE DEPARTMENT
OF HIGHWAYS AND
PUBLIC TRANSPORTATION

COOPERATIVE
RESEARCH

# A STUDY OF THE EFFECTS OF DESIGN AND OPERATIONAL PERFORMANCE OF SIGNAL SYSTEMS--FINAL REPORT

in cooperation with the
Department of Transportation
Federal Highway Administration

RESEARCH REPORT 203-2F
STUDY 2-18-75-203
PERFORMANCE OF SIGNAL SYSTEMS

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# A STUDY OF THE EFFECTS OF DESIGN AND OPERATIONAL PERFORMANCE OF SIGNAL SYSTEMS -- FINAL REPORT

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Research Report Number 203-2F

Effects of Design on Operational Performance
of Signal Systems

Research Study Number 2-18-75-203

Sponsored by
State Department of Highways and Public Transportation
In Cooperation with the
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Texas Transportation Institute
Texas A&M University
College Station, Texas

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ABSTRACT

This report presents the findings of a research project entitled "Effects of Design on Operational Performance of Signal Systems" sponsored by the State Department of Highways and Public Transportation in Texas in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Areas covered include the following: peaking characteristics of volumes at intersections in Texas during rush hour traffic conditions, left turn capacity of an approach having no protected signal phasing as related to opposing traffic volumes and intersections of different geometric design, effects of signal phasing and length of left turn bay on intersection approach capacity, and development of a new field evaluation technique for signalized intersections. In addition, research was conducted to improve the Department's PASSER-II signal progression program. Platoon movement along an arterial street and the effects of progression on vehicle delay are investigated.

Key Words: Intersection Design, Intersection Operations, Peaking Characteristics, Left Turn Capacity, Intersection Capacity, Signal Progression, Platoon Dispersion

## SUMMARY

The ability of a signalized intersection to move traffic is determined by the physical features of the intersection as well as the type of signalization used. Also, the geometric design of the intersection directly affects the ability of the signalization to move traffic. Thus, total system design of a signalized intersection involves concurrent evaluation of the proposed geometric design and traffic control devices as they will function together in the field as an integrated unit. To better understand these relationships, the State Department of Highways and Public Transportation in Texas in cooperation with the Federal Highway Administration sponsored a research project entitled "Effects of Design on Operational Performance of Signal Systems." This report presents documentation and results of this research project.

The first section of the report defines peaking characteristics of volumes at intersections in Texas during rush hour traffic conditions. Peaking factors that should be used in the determination of the design period volumes are based on the population of the city in which the intersection is located.

Left turn capacity of an approach having no protected signal phasing is related to opposing traffic volumes and intersections of different geometric design. Left turn capacities on approaches with and without left turn lanes are addressed. A mathematical model was developed to calculate the left turn capacity of an intersection. Parameters used in the model were calibrated from field studies conducted in several Texas cities.

A periodic scan computer simulation program was developed to investigate the effects of signal phasing and length of left turn bay on capacity. After the simulation program was tested for realism, inputs (phase sequence, volume, cycle length and length of left turn lane) were varied in order to evaluate

their interrelationships over a broad range of conditions. Relationships between each of the variables are presented.

A traffic flow, field evaluation technique is presented. This procedure evaluates the operational measures of effectiveness of saturation (volume-to-capacity) ratio, probability of clearing queues and average vehicle delay from traffic characteristics which can be easily measured at the intersection by only one observer. Development of this new evaluation technique is presented in the report.

Research was conducted to improve the Department's PASSER-II signal progression program. The major task undertaken was to add an arterial signal system evaluation routine to the basic program. Primary research emphasis was directed toward characterizing the movement of progressive platoons along an arterial and developing a mathematical model for estimating delay on the arterial through movements where progression is provided. Webster's delay equation was modified to estimate the effects of progression on delay. Platoon movement down an arterial has been characterized by several equations presented in this report. Results of this portion of the research effort have been incorporated into the PASSER-II program.

## Implementation

This report provides documentation of research results currently being used in the development of the latest edition of the highway and public transportation design manual of the Texas State Department of Highways and Public Transportation. Extensions to the Department's traffic signal computer program PASSER-II are also described. The basic program is currently operational on the district's remote computer terminals.

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# INTRODUCTION

This report presents the documentation of reasearch conducted within the research project entitled "Effects of Design on Operational Performance of Signal Systems." Much of the results of this research were used in the development of an earlier project report (1), entitled "A Guide for Designing and Operating Signalized Intersections in Texas." Since the earlier project report was a design guide, no documentation was provided in it.

Included in a subsequent section of this report is a description of the research conducted on extensions made to the State Department of Highways and Public Transportation (SDHPT) arterial progression, computer program, PASSER-II (2), developed in an earlier research project.

## Scope of the Design Guide

The design guide (1) presented a methodology for designing signalized intersections to serve rush hour traffic demands. Physical design and signalization alternatives were identified and methods for evaluation were provided. The guide began with a description of the procedures used to convert given traffic volume data for the design year into equivalent peak design period turning movement volumes. It was necessary to develop a set of peaking factors for Texas cities (Doc.) (Documentation to follow). All volumes were then converted into equivalent passenger car volumes. This allowed turning movement volumes which have different capacities to be converted into equivalent movements (Doc.) having slightly larger equivalent volumes but the same saturation flow per lane. All types of signalized intersections can be analyzed in this manner.

Capacities of left turning phases at signalized intersections were estimated (Doc.) based on considerable field data collected during this research. Capacities for left turns with and without left turn bays were provided. In addition, guidelines were provided (Doc.) for designing the length of storage bay required for a given left turning volume. Decreases in capacity were given as the length of the left turn storage was reduced below minimum desirable values.

The critical lane analysis technique was applied to the proposed design and signalization plan. The resulting sum of critical lane volumes could then be checked against established maximum values for each Level of Service to determine the acceptability of the design. Guidelines and example problems were presented to assist the engineer in determining satisfactory design alternatives. Signalization alternatives were also described.

Operational performance characteristics of the intersection were related to signalization and design alternatives in subsequent sections of the report. The selected design Level of Service criteria were discussed. A signalization timing plan was developed and evaluated for one of the design example problems.

In the last section of the report, a new traffic flow, field evaluation technique was presented. This procedure (Doc.) evaluated the operational measures of effectiveness of saturation ratio, probability of clearing queues and average vehicle delay from traffic characteristics which could be easily measured at the intersection by only one observer.

## PEAKING FACTOR, PF

During the development of the design guide, the need arose to define the peaking characteristics of volumes at intersections during rush hour traffic. This need was due to the established design criterion of designing signalized intersections for average flow conditions during the peak period (15 minutes) of the design hour. In order to design for peak period conditions, it was necessary to know the peak period flow rates. These design flow rates were calculated from:

$$DPV_m = DHV_m \cdot PF \qquad (1)$$

where:

$DPV_m$ = Average flow on movement "m" during the design period (peak 15 minutes) of the design hour, cars/hr.

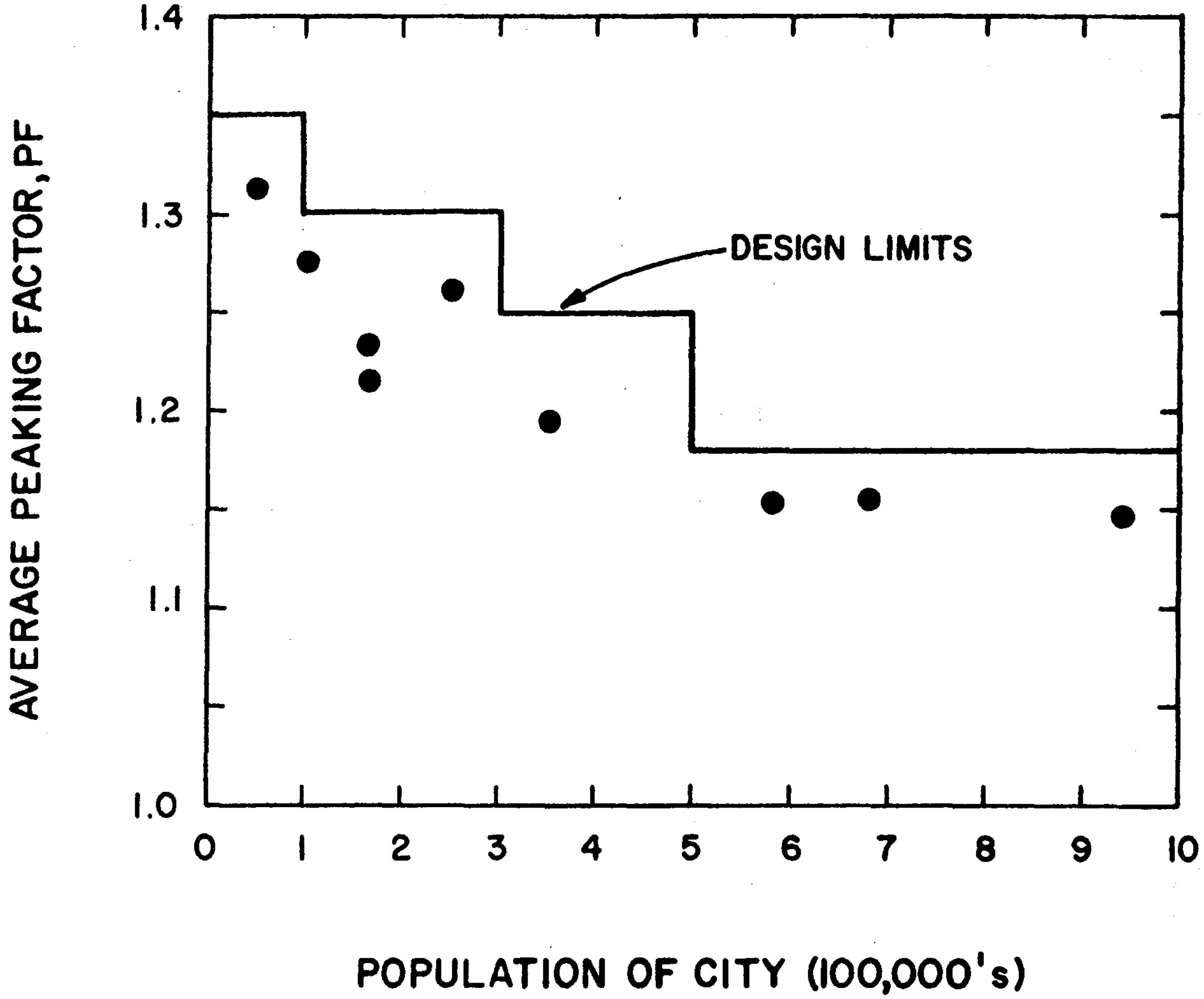
$DHV_m$ = Design hour volume on movement "m", cars/hr.

PF = Peaking factor for intersection.

Peaking factors are initially developed from traffic volume data. Drew studied peaking characteristics in Texas in 1961 (3) and found that peaking factors varied primarily with the population of the city, although other factors were also considered. The Highway Capacity Manual, 1965 (4) also considered peaking characteristics during the peak 15 minute period of the design hour. Population of the city was discussed as being related to peaking characteristics. Specific peaking characteristics were related to population for freeway design; however, no definite relationships were provided for intersections. The peak hour factors used in the Manual to define peaking characteristics are actually reciprocals of peaking factors.

In addition to these previous data sources, average peaking factors were determined for two cities in Texas from 1974 traffic volume data. A total of 92 peaking factors were measured in Austin (5) and 16 in Bryan-College Station.

A summary of the peaking data and peaking factor design limits established in this research is shown in Figure 1. Population of the city (latest census)




PEAKING FACTOR VARIATION WITH POPULATION OF TEXAS CITIES

FIGURE 1

where the intersection is located is used to select the appropriate peaking factor. In reality, the peaking factor should be selected which reflects the amount of volume peaking expected during the design hour. An intersection located near a major traffic generator which would cause a high rate of flow over a short period of time would perhaps have a peaking factor of 1.4 to 1.6 (4). A small municipality surrounded by a large city would probably have peaking characteristics similar to that of the larger city. Knowledge of the peaking characteristics in the locality where the intersection is to be located is obviously desirable.

## LEFT TURN CAPACITY

The 1965 edition of the Highway Capacity Manual (6) states that the left turn capacity of an unprotected movement with a left turn lane of adequate length is "equal to the difference between 1,200 vehicles and the total opposing traffic volume in terms of passenger cars per hour of green, but not less than two vehicles per cycle." If a left turn lane is not provided, an adjustment factor based on the percent of traffic turning left is used to determine the capacity of that approach. Opposing traffic is not considered. The Australian method (7, 8) utilizes a left turn equivalency factor for opposed left turners based on opposing vehicular volume.

In this portion of the research effort, left turn capacity of an approach having no protected signal phasing is related to opposing traffic volumes and intersections of different geometric design. Left turn capacities on approaches with and without left turn lanes are addressed. A mathematical model was developed to calculate the left turn capacity of an intersection. Parameters used in the model were calibrated from field studies conducted in several Texas cities.

## Development of Model

A literature review and the experience of the research team indicated that the left turn capacity of an intersection was primarily related to the amount of traffic opposing the left turn movement. Unless an exclusive turning phase is provided, left turning vehicles must turn across the intersection through gaps that occur in the opposing traffic stream. For higher opposing flow rates, fewer gaps of acceptable size for turning occur. At an intersection controlled by a two-phase (unprotected left turn) signal, the left turning movement is blocked for a period of time by the dissipation of the opposing queue which builds up during the red phase of the cycle. Therefore, the left turn capacity of an intersection controlled by a two-phase signal is a function not only of the probability of gaps occuring in the opposing traffic stream, but also the available time during which turning can occur. An equation to express the above concept can be written as follows:

$$Q_L = \frac{T_A}{C} \cdot Q_{LH} \qquad (2)$$

where:

$Q_L$ = Left turn capacity of an approach, cars/hr.

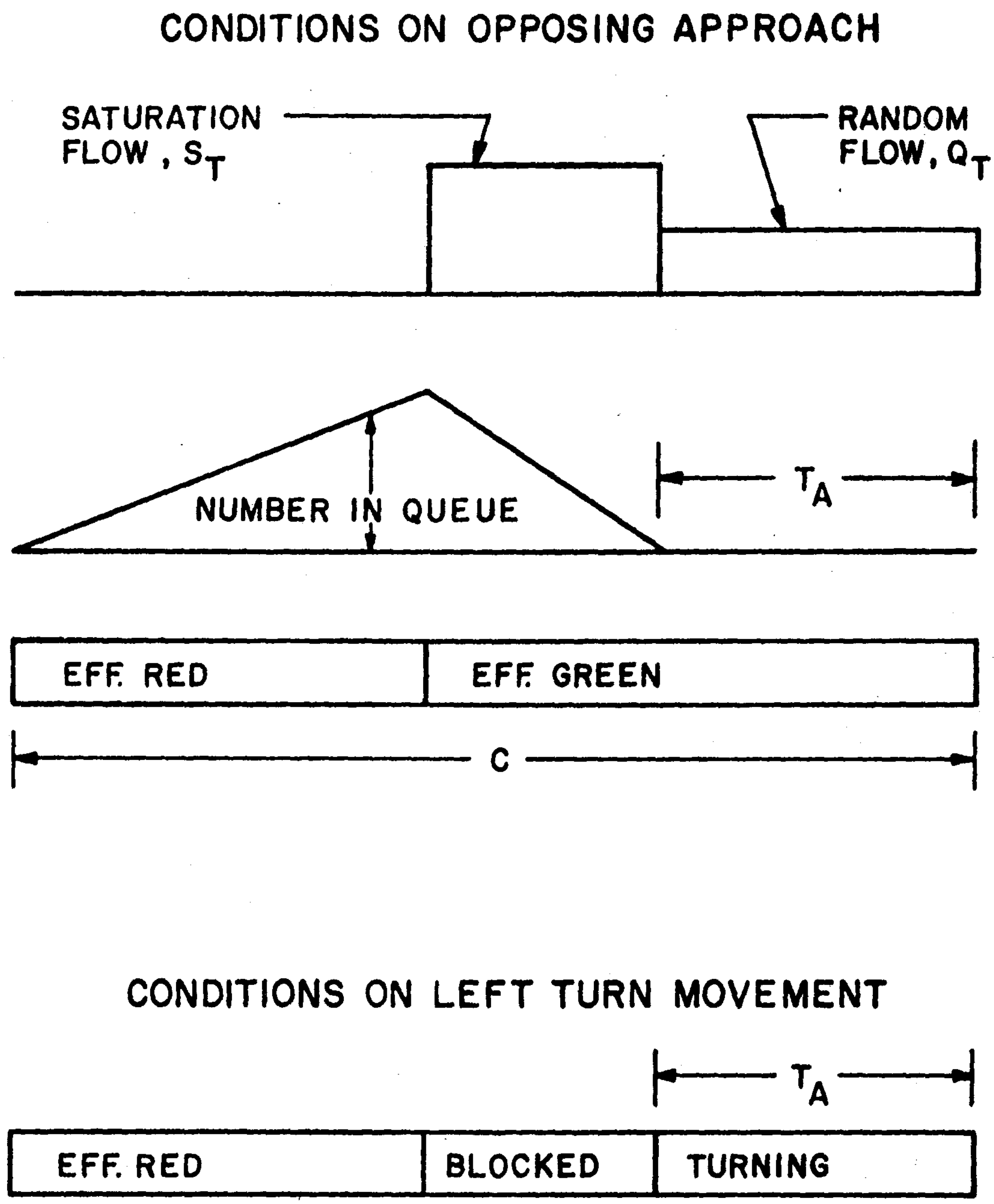
$T_A$ = Available time per cycle during which turning may occur, sec.

$C$ = Cycle length, sec.

$Q_{LH}$ = Left turn capacity of an approach across free-flow, random traffic, cars/hr. of available green time.

## Time Available for Turning, $T_A$

For two-phase (unprotected) signal operation, the events shown in Figure 2 occur on the approach opposite the left turn movement of interest.




# LEFT TURN CAPACITY CONDITIONS

FIGURE 2

At a point during the amber time, left turns and opposing through traffic are stopped and a queue starts to build on the opposing approach at the average arrival rate of $Q_T/3600$ vehicles per second. The portion of the amber time not used by through traffic can be thought of as lost time to the signal. The queue continues to build on the opposing approach at the same rate during the red period of the signal. A second lost time occurs at the beginning of the green due to the time it takes for vehicles in the queue to start moving. At this time, the queue has reached its maximum length.

After the lost time at the beginning of the green interval, the queue begins to clear at the rate of the saturation flow ($S_T$) minus the average arrival rate ($Q_T$) converted to vehicles per second. After the queue has cleared, normal flow resumes at the average arrival rate for the remainder of the green time plus a portion of the amber time. During this time interval, $T_A$, left turning vehicles may cross the opposing traffic movement as acceptable gaps occur in the opposing flow, which is assumed to be random.

If the average number of arrivals on each lane of the opposing approach were equal on a cycle-by-cycle basis, the queues in each lane would clear simultaneously. However, this is not the case. Therefore, the time available for turning should be based on the time required for the lane with the longest queue to clear the intersection.

Bellis (9) estimated that under capacity (high volume) conditions the percentage distribution of traffic among lanes was 55-45 on a two-lane approach and 40-35-25 on a three-lane approach. If only one vehicle arrived during a cycle (low volume), the percentage distribution would be 100-0 or 100-0-0. Using these two boundary conditions, the percentage distribution of traffic in the highest volume lane for various volume conditions can be estimated in the following manner:

Two-lane approach : $P = 0.55 + 0.45e^{-.18m}$

Three-lane approach : $P = 0.40 + 0.60e^{-.13m}$

where:

$P$ = Percent traffic in highest volume lane (expressed as a decimal).

$m$ = Average number of arrivals per cycle $(Q_T \cdot C)/3600$.

$Q_T$ = Total opposing volume, cars/hr.

$C$ = Cycle length, sec.

As previously discussed, the left turn movement cannot begin until the longest opposing queue has cleared the intersection. The time required to clear the longest opposing queue in a lane is:

$$T_Q = \frac{P \cdot Q_T \ (L_1 + R + L_2)}{S_T - P \cdot Q_T} \qquad (3)$$

where:

$T_Q$ = Time for longest opposing traffic queue to clear, sec.

$L_1$ = Portion of amber time not used by through traffic, sec.

$R$ = Length of red phase of cycle, sec.

$L_2$ = Initial lost time at the beginning of the green interval, sec.

$S_T$ = Saturation flow of longest opposing queue, 1750 cars/hr. per lane (5).

Therefore, the time that is available for left turning per cycle is:

$$T_A = G + A - L_1 - L_2 - T_Q \qquad (4)$$

where:

$T_A$ = Time available for left turning per cycle, sec.

$G$ = Length of green phase of cycle, sec.

$A$ = Length of amber phase of cycle, sec.

Free-Flow Turning Capacity, $Q_{LH}$

Once the longest queue of opposing traffic has dissipated, free-flowing vehicles continue to approach the intersection in a random manner forming gaps of various sizes in the opposing traffic stream. Drivers waiting at the signal wishing to turn left reject these gaps until one of adequate length for turning arrives. An acceptable gap is assumed to be one equal to or larger than the critical gap, $T_c$ (that gap for which an equal percentage of turning traffic will accept a smaller gap as will reject a larger one).

More than one vehicle may turn through an acceptable gap if it is of sufficient length. The time between consecutive vehicles turning through the same gap is defined as the turning headway, H. If a uniform arrival rate can be expected during periods of free flow, the negative exponential distribution can be used to represent the probability of gap occurrence. Based on this concept, Drew (10) presents the following equation which can be used to determine the left turn capacity of an intersection during free-flow conditions:

$$Q_{LH} = Q_T \left[ \frac{e^{-q_T T_c}}{1 - e^{-q_T H}} \right] \tag{5}$$

where:

$Q_T$ = Total opposing traffic (through and right) cars/hr.

$q_T$ = Total opposing traffic (through and right) cars/sec.

$T_c$ = Critical gap, sec.

$H$ = Turning headway, sec.

An example problem illustrating the calculation of the left turn capacity of an unprotected left turn movement is shown in Appendix A.

## Parameter Studies

Data were collected at several intersections to determine the parametric values of the model by using a portable video tape recording system. This permitted the recording and analysis of a greater number of traffic measures than could have been accomplished by the limited number of data collectors available for use in the field. Figure 3 shows the portable video camera system in use by members of the research team. The playback unit and monitor shown in Figure 4 were used to replay the videotapes recorded at the study sites. A stop-watch was used to time vehicle movements and signal intervals. Twelve intersections were filmed during the course of the study, six with and six without separate left turn lanes. Tables 1 and 2 summarize the study locations.

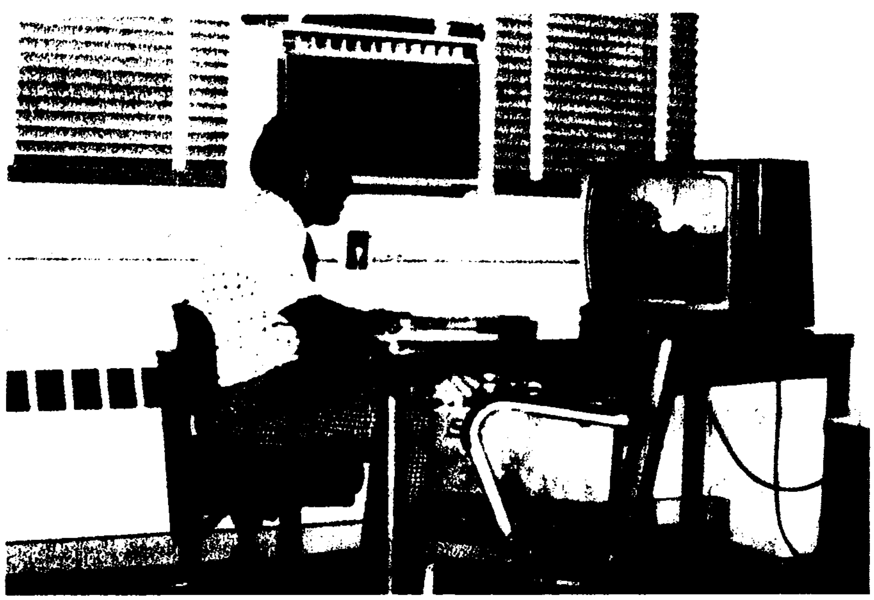
## Results of Field Studies

Observations in the field and from the video tapes resulted in the following data and conclusions about vehicles turning left at intersections controlled by a two-phase signal.

*Critical Gap*, $T_c$- Of the six intersections studied that had left turn lanes, useable critical gap data was collected for three of the intersections. The largest gap rejected and the smallest gap accepted for each left turning vehicle were recorded. From these data, graphs of the cumulative totals for rejected and accepted gaps intersect at a value which approximates the critical gap (10). The critical gap for each of the three intersections is shown in Figure 5. Figure 6 represents the same data combined. Based on

FIELD DATA COLLECTION

FIGURE 3



FIELD DATA REDUCTION

FIGURE 4

TABLE 1

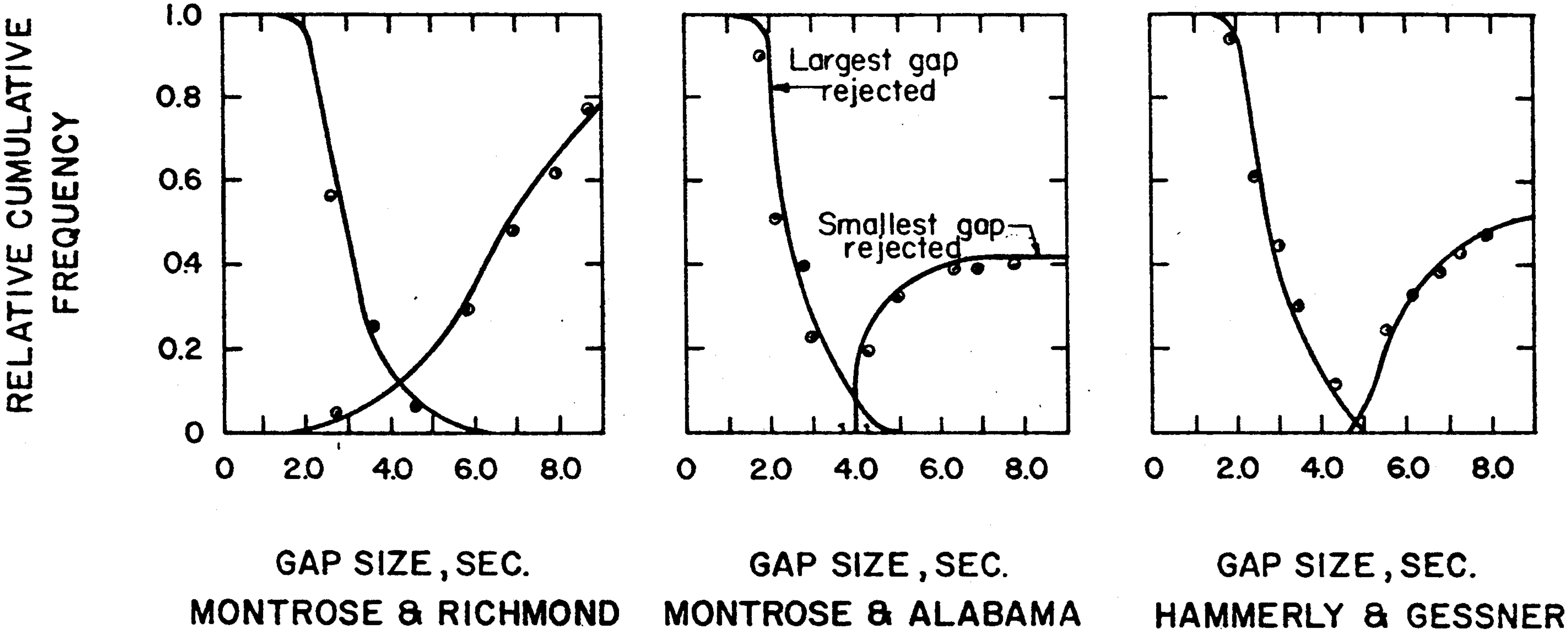
INTERSECTIONS STUDIED WITH SEPARATE LEFT TURN LANES

| Intersection | Location | Date | Through Lanes | Cycles Recorded |
|---|---|---|---|---|
| 15th at Congress | Austin | October 10, 1974 | 2 | 22 |
| 26th at San Jacinto | Austin | December 4, 1974 | 3 | 13 |
| Texas Ave. at Coulter | Bryan | January 9, 1975 | 2 | 19 |
| Hammerly at Gessner | Houston | March 26, 1975 | 2 | 27 |
| Montrose at Alabama | Houston | March 27, 1975 | 2 | 20 |
| Montrose at Richmond | Houston | April 15, 1975 | 2 | 44 |

TABLE 2

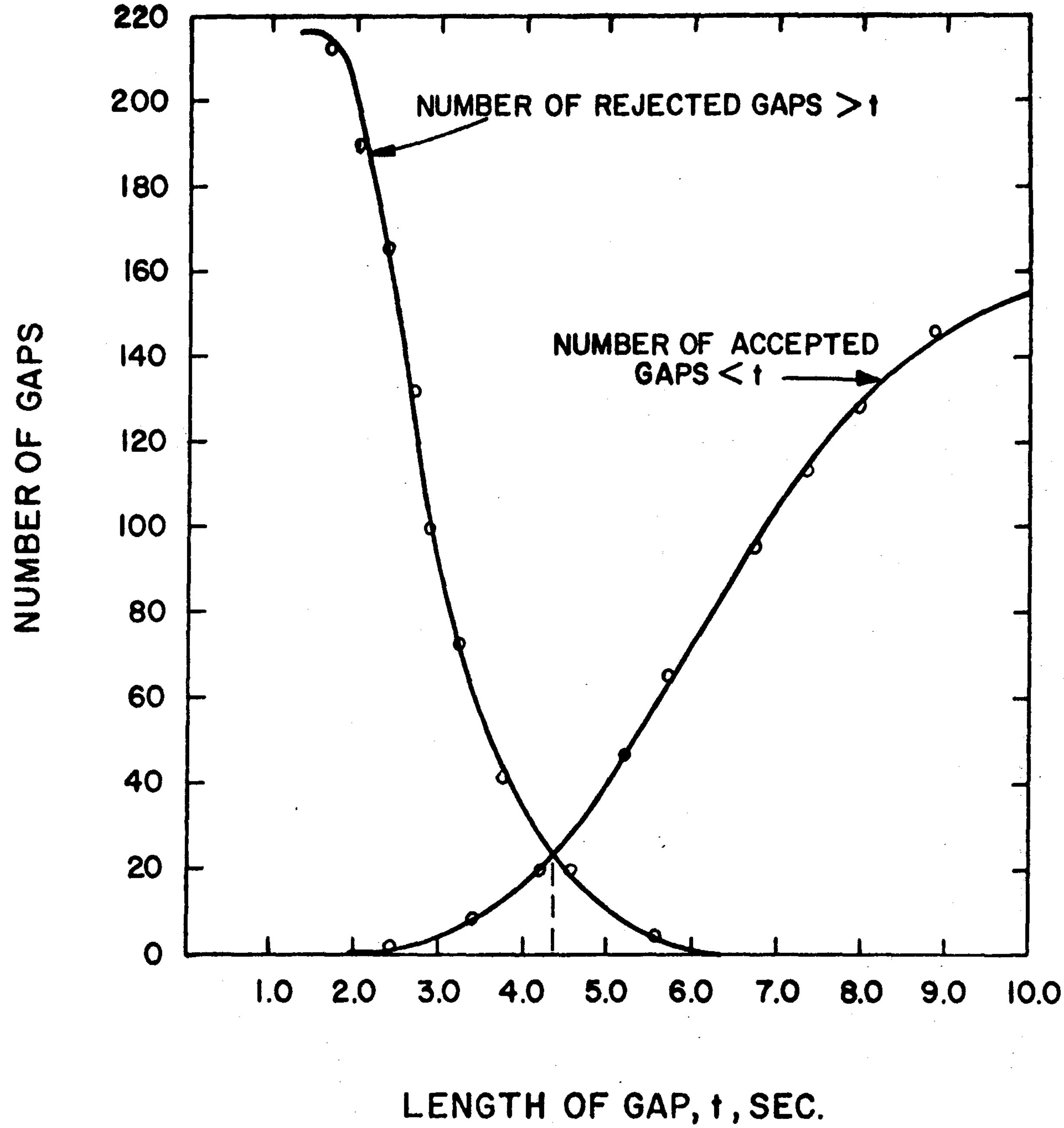
INTERSECTIONS STUDIED WITHOUT LEFT TURN LANES

| Intersection | Location | Date | Through Lanes | Cycles Recorded |
|---|---|---|---|---|
| 1st at Oltorf | Austin | December 3, 1974 | 2 | 20 |
| College Avenue at Sulphur Springs | Bryan | January 7, 1975 | 2 | 19 |
| College Ave. at Dodge | Bryan | January 8, 1975 | 2 | 35 |
| College Ave. at Dodge | Bryan | January 15, 1975 | 2 | 28 |
| SH 21 at 19th St. | Bryan | January 16, 1975 | 1 | 16 |
| College Ave. at Carson | Bryan | January 27, 1975 | 2 | 50 |
| 38th at Lamar | Austin | January 28, 1975 | 2 | 31 |




CRITICAL GAP DATA FOR THREE INTERSECTIONS WITH LEFT TURN LANES AND TWO-PHASE SIGNALS

FIGURE 5




PAIRS OF ACCEPTED AND REJECTED GAPS
FOR THREE INTERSECTIONS WITH
LEFT TURN LANES

FIGURE 6

these results, a value for the critical gap of 4.5 seconds was selected as a reasonable value for use in the left turn capacity equation.

*Turning Vehicle Headway*, H - Headways between left turning vehicles were found by measuring the time between completion of the turning movement of successive vehicles turning through the same gap. Only those cycles during which more than one vehicle turned through the same gap resulted in useable data. Table 3 is a summary of the headways which were measured for vehicles turning left from left turn lanes. For intersections without left turn lanes, the average turning headways were slightly higher as shown in Table 4. The results of this analysis indicate that values of 2.5 seconds for "H" at intersections with left turn lanes and 2.6 seconds for "H" at intersections without left turn lanes would be appropriate for use in the left turn capacity equation.

*Lane Distribution* - As the field data collection progressed, there appeared to be a variation in the proportion of vehicles using each through lane on a cycle-by-cycle basis. This occurred over a period of time even when volumes in each lane were approximately equal. Lane distribution is significant in the queue clearance portion of the model because left turners cannot begin to turn during each cycle until the longest lane queue has cleared the intersection.

To measure this characteristic, vehicle volumes per lane were recorded from the video tape on a cycle-by-cycle basis. For each cycle the higher volume was divided by the total volume on the approach to get the percentage of vehicles in the longest queue, P, expressed as a decimal. These values were averaged over the number of cycles recorded. Table 5 shows values for "P" along with corresponding expanded hourly volumes. This summary contains

TABLE 3

SUMMARY OF LEFT TURNING VEHICLE HEADWAYS FROM PROTECTED LEFT TURN LANES

| Location | Number of Headways | Average Headway (H) |
| --- | --- | --- |
| 15th at Congress | 15 | 2.71 |
| Texas Ave. at Coulter | 29 | 2.79 |
| Hammerly at Gessner | 111 | 2.60 |
| Montrose at Alabama | 10 | 2.44 |
| Montrose at Richmond | 146 | 2.31 |
| Total | 311 | 2.48 sec. |

TABLE 4

SUMMARY OF LEFT TURNING VEHICLE HEADWAYS FROM THE MEDIAN THROUGH LANE

| Location | Number of Headways | Average Headway (H) |
| --- | --- | --- |
| 1st and Oltorf | 10 | 2.56 |
| College Ave. at Dodge | 20 | 2.72 |
| College Ave. at Carson | 5 | 2.32 |
| Total | 35 | 2.62 sec. |

TABLE 5

AVERAGE PERCENTAGE OF VEHICLES IN HIGHER VOLUME LANE ON A

CYCLE-BY-CYCLE BASIS

| Location | Direction | Hourly Volume, Q | Percent in Higher Lane, P |
|---|---|---|---|
| SH 6 at SH 21 | SB | 312 | .705 |
| Montrose at Alabama (March 27, 1975) | NB | 473 | .594 |
| SH 6 at SH 21 | NB | 506 | .651 |
| Hammerly at Gessner | WB | 511 | .608 |
| Montrose at Alabama (March 27, 1975) | SB | 529 | .628 |
| Hammerly at Gessner | EB | 600 | .614 |
| Montrose at Alabama (March 15, 1975) | SB | 820 | .562 |
| Montrose at Richmond | NB | 925 | .562 |
| | SB | 975 | .559 |
| Montrose at Alabama (March 15, 1975) | NB | 1002 | .543 |

data from an extra hand count made at the intersection of State Highways 6 and 21 in Bryan in order to gain information at lower volume levels. As expected the percentage of volume in the longest queued lane decreased as approach volume increased. The observed and modelled lane distributions are illustrated in Figure 7. For the case of no left turn lane and two lanes of opposing flow, the equations presented in this paper slightly underestimated the length of the longest lane queue in high volume cases. A complex mathematical equation was used to determine the lane distribution in three instances.

## Observations from Field Studies

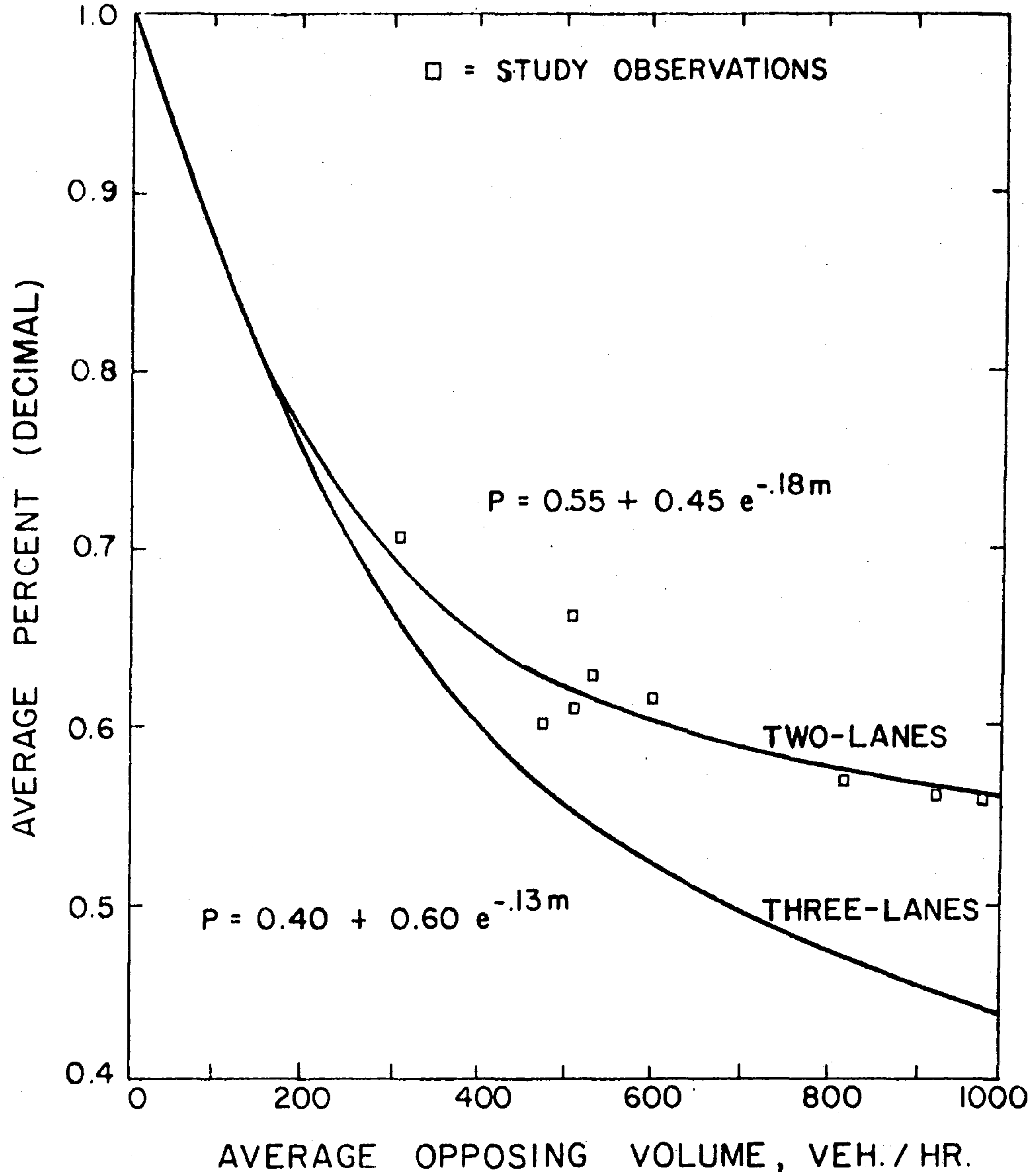
Prior to the data collection phase of this study, two questions were raised concerning the following:

1.) How frequently do vehicles turn left before the opposing queue begins to move?

2.) How many vehicles turn left on the amber and start of red intervals each cycle?

*Turning Before the Opposing Queue Starts* - In 1966 Dart (11) observed 220 signal cycles containing left turners at the head of the queue. Based on these observations, he concluded that when the lead vehicle was a left turner the probability of it "jumping the gun" (turning before the opposing queue) was about 0.145. A much lower rate of occurrence was found to occur during the course of the field data collection as shown in Tables 6 and 7. The relatively small number of vehicles "jumping the gun" do not appear to significantly increase the left turn capacity of an intersection. Therefore, no adjustment is made for this phenomenon in the final capacity analysis.

*Left Turns on Amber and Red* - For relatively light opposing traffic, sufficient time is usually available for left turns to be made during the




AVERAGE PERCENT OF TRAFFIC IN HIGHER VOLUME LANE ON ONE APPROACH

FIGURE 7

TABLE 6

OCCURRENCE OF LEFT TURNING VEHICLES TURNING BEFORE THE OPPOSING QUEUE STARTED FOR APPROACHES WITH LEFT TURN LANES

| Location | Number of Cycles with Left Turns at Head of Queue | Number of Vehicles Turning Before Opposing Queue Enters | Rate of Occurrence |
|---|---|---|---|
| 15th at Congress | 27 | 0 | 0 |
| 26th at San Jacinto | 11 | 0 | 0 |
| Texas Ave. at Coulter | 30 | 0 | 0 |
| Hammerly at Gessner | 52 | 1 | .019 |
| Montrose at Alabama | 20 | 0 | 0 |
| Montrose at Richmond | 84 | 4 | .048 |

TABLE 7

OCCURRENCE OF LEFT TURNING VEHICLES TURNING BEFORE THE OPPOSING QUEUE STARTED FOR APPROACHES WITHOUT LEFT TURN LANES

| Location | Number of Cycles with Left Turns at Head of Queue | Number of Vehicles Turning Before Opposing Queue Enters | Rate of Occurrence |
|---|---|---|---|
| 1st at Oltorf | 15 | 1 | 0.067 |
| College at Sulphur Springs | 20 | 0 | 0 |
| College at Dodge (1/8/75) | 26 | 4 | 0.154 |
| College at Dodge (1/15/75) | 38 | 4 | 0.105 |
| College at Carson | 44 | 3 | 0.068 |
| 38th at Lamar | 35 | 0 | 0 |

green interval. As opposing traffic increases, less and less green time is available for left turns until the queue fails to clear the intersection during the green portion of the cycle. However, observations made during data collection indicated that at this point the lead left turner in the queue will usually be waiting in the intersection and will choose to turn either on the amber or on the beginning of the red portion of the cycle. In fact, for high opposing volumes, this movement appears to be the major source of left turn capacity. The Highway Capacity Manual (6) reflects this concept in the section where it gives the left turn capacity as "no less than two vehicles per cycle". The Australians (7, 8) indicate that at least 1.5 vehicles per cycle can turn left during this time period. The State Department of Highways and Public Transportation in Texas (12) has been using a value of 1.6 left turners per cycle as a minimum in some capacity analyses.

To be certain that these turns were indeed occurring, the following data, as summarized in Tables 8 and 9, were gathered from the video recordings. The data indicate that if a lead driver in the queue is not given an opportunity to turn left during the green interval, he will turn on the amber or the red. As the opposing volumes rise to near capacity, the only left turn capacity that remains is the vehicles which clear on the amber or the red. Results of this study indicate that the left turn capacity of an intersection averaged 1.41 vehicles per cycle turning on amber and red when a left turn lane was present and 1.03 vehicles per cycle turning on amber and red when no turning lane was provided. As this study was rather limited, no change from the currently used value of 1.6 vehicles per cycle was made where a left turn lane is provided. However, a minimum turning volume per cycle of 1.0 was selected where no left turn lane is provided.

TABLE 8

TURNS ON AMBER AND RED FOR INTERSECTIONS WITH LEFT TURN LANES

| Location | | Opposing Volume | Cycles With | | | Total Veh. Cleared |
|---|---|---|---|---|---|---|
| | | | Left on Amber | Left on Red | Both | |
| Montrose at | (NB) | 975 | 3 | 11 | 3 | 29 |
| Richmond | (SB) | 925 | 3 | 12 | 4 | 26 |
| Montrose at | (NB) | 475 | 0 | 1 | 0 | 1 |
| Alabama | (SB) | 448 | 3 | 0 | 0 | 3 |
| Hammerly at | (NB) | 511 | 6 | 1 | 3 | 13 |
| Gessner | (SB) | 600 | 8 | 1 | 4 | 21 |
| Texas at | (NB) | 94 | 2 | 9 | 0 | 2 |
| Coulter | (SB) | 198 | 0 | 1 | 1 | 3 |
| 15th at Congress | (EB) | 655 | 6 | 8 | 3 | 23 |
| 26th at San Jacinto | (EB) | 295 | 0 | 11 | 0 | 13 |

TABLE 9

TURNS ON AMBER AND RED FOR INTERSECTIONS WITHOUT LEFT TURN LANES

| Location | | Opposing Volume | Cycles With | | | Total Veh. Cleared |
|---|---|---|---|---|---|---|
| | | | Left on Amber | Left on Red | Both | |
| College at | (NB) | 525 | 0 | 0 | 0 | 0 |
| Carson | (SB) | 437 | 5 | 0 | 0 | 5 |
| College at | (NB) | 451 | 3 | 0 | 0 | 3 |
| Dodge | (SB) | 432 | 4 | 1 | 1 | 7 |
| 1st at Oltorf | (SB) | 225 | 1 | 1 | 0 | 2 |
| 38th at | (EB) | 569 | 0 | 11 | 0 | 11 |
| Lamar | (WB) | 463 | 0 | 8 | 0 | 8 |

## Model Results

The capacity of a left turning movement made from a left turn lane without protected signal phasing is given in Table 10 as estimated by the model. The capacity values are the maximum possible sustained flow rates which could occur during the peak 15-minute period of the design hour by passenger cars only. Trucks in the opposing volume and effective left turn demand must be converted into equivalent passenger cars. Any left turn bay must be sufficiently long so that no blockages occur between the left turn queue and through movement vehicles.

In the design guide (1), the effects of different left turn capacities are accounted for by calculating left turning equivalents, similar to the Australian method (7, 8). These equivalents were calculated for given conditions from

$$E = \frac{1750 \cdot G}{CAP \cdot C} \tag{6}$$

where

- E = Left turning equivalent factor for left turns from a left turn lane but no protected signal phase.
- C = Cycle length, sec.
- G = Phase green, sec.
- CAP = Left turn capacity of unprotected signal phase (Table 10), cars/hr.

An attempt was made at estimating the capacity of a left turning movement without either protected signal phasing or a left turn lane. The field studies had indicated that the previous model could be applied with some modifications and simplifying assumptions. Change in critical gaps ($T_C$ = 4.5 sec.) was neither observed nor assumed. The minimum turning

TABLE 10

LEFT TURNING CAPACITY OF SINGLE PHASE

WITH UNPROTECTED TURNING AND ADEQUATE BAY LENGTH

| | Total Opposing Through and Right Turning Volumes, cars/hr. | | | | |
|---|---|---|---|---|---|
| | 200 | 400 | 600 | 800 | 1000 |
| G/C = .3 | | | | | |
| N = 1 | 232 | 82 | 82 | 82 | 82 |
| N = 2 | 260 | 159 | 82 | 82 | 82 |
| N = 3 | 261 | 168 | 105 | 82 | 82 |
| G/C = .4 | | | | | |
| N = 1 | 368 | 204 | 82 | 82 | 82 |
| N = 2 | 392 | 276 | 187 | 114 | 82 |
| N = 3 | 393 | 285 | 207 | 147 | 100 |
| G/C = .5 | | | | | |
| N = 1 | 503 | 333 | 181 | 82 | 82 |
| N = 2 | 524 | 394 | 292 | 208 | 138 |
| N = 3 | 525 | 401 | 309 | 236 | 177 |
| G/C = .6 | | | | | |
| N = 1 | 639 | 463 | 307 | 164 | 82 |
| N = 2 | 655 | 512 | 397 | 302 | 223 |
| N = 3 | 656 | 518 | 410 | 324 | 254 |
| G/C = .7 | | | | | |
| N = 1 | 775 | 593 | 434 | 291 | 153 |
| N = 2 | 787 | 630 | 502 | 396 | 307 |
| N = 3 | 788 | 634 | 512 | 413 | 331 |

G/C = Actual green/cycle length.

N = Number of opposing through lanes.

headways between consecutive left turning vehicles were observed to be slightly longer ($H = 2.6$ sec.) than when made from a left turn lane. To simplify the capacity and equivalence calculations, it was assumed that 50 percent of the traffic in the median lane, from which left turns are made, were left turning vehicles. This assumption is more representative of heavy volume conditions than light flow operations.

The minimum effective left turning headway across long gaps becomes

$$H_{eff} = 2.06 + 2.60 \text{ sec.}$$

since it is assumed that every other vehicle is going through at a minimum headway of 2.06 seconds (5). With $T_c = 4.5$ seconds and $H_{eff} = 4.66$ seconds used in the model, the left turn capacities of an approach having no left turn lane, no protected left turn signal phasing and 50 percent of median lane turning left, were calculated as presented in Table 11.

The left turn equivalent factors for these conditions used in the design guide (1) were calculated from the following development:

$$P_L \cdot H_L + P_T \cdot H_T = H_{ave.} = \frac{3600}{\frac{CAP}{P_L} \cdot \frac{C}{G}} \qquad (7)$$

where

$P_L$ = Percent of median lane traffic turning left ($P_L = 0.5$).

$P_T$ = Percent of median lane traffic going through ($P_T = 1 - P_L = 0.5$).

$H_L$ = Average left turn headway at capacity, sec.

$H_T$ = Average through headway at capacity (2.06 sec.), sec.

$H_{ave}$ = Average median lane headway, sec.

CAP = Left turn capacity (Table 11), cars/hr.

G = Approach signal green, sec.

C = Cycle length, sec.

TABLE 11

ESTIMATED CAPACITY OF LEFT TURNING MOVEMENT

WITHOUT PROTECTED SIGNAL PHASE OR LEFT TURN LANE

| | Total Opposing Volumes*, cars/hr. | | | | |
|---|---|---|---|---|---|
| | 200 | 400 | 600 | 800 | 1000 |
| G/C = .3 | | | | | |
| N = 1 | 132 | 50 | 50 | 50 | 50 |
| N = 2 | 148 | 95 | 53 | 50 | 50 |
| N = 3 | 149 | 101 | 66 | 50 | 50 |
| G/C = .4 | | | | | |
| N = 1 | 209 | 122 | 50 | 50 | 50 |
| N = 2 | 223 | 166 | 119 | 50 | 50 |
| N = 3 | 223 | 171 | 131 | 97 | 50 |
| G/C = .5 | | | | | |
| N = 1 | 286 | 200 | 114 | 50 | 50 |
| N = 2 | 298 | 237 | 185 | 122 | 50 |
| N = 3 | 298 | 241 | 195 | 156 | 122 |
| G/C = .6 | | | | | |
| N = 1 | 363 | 278 | 194 | 108 | 50 |
| N = 2 | 373 | 307 | 251 | 187 | 122 |
| N = 3 | 373 | 311 | 259 | 214 | 175 |
| G/C = .7 | | | | | |
| N = 1 | 441 | 356 | 274 | 192 | 105 |
| N = 2 | 447 | 378 | 317 | 252 | 188 |
| N = 3 | 448 | 381 | 323 | 273 | 228 |

G/C = Actual green/cycle length.

N = Number of opposing through lanes.

* = Includes through and right turns for N = 1.
Includes lefts, throughs, and right turns for N = 2 and 3.

The left turn equivalent factor, E, for left turns with no bay or protected signal phasing can now be calculated by solving for $H_L$ in the previous equation and dividing by $H_T$. Thus,

$$E = \frac{H_L}{H_T} = \frac{3600 \cdot G}{CAP \cdot C \cdot H_T} - \frac{P_T}{P_L} \qquad (8)$$

Since it is assumed that $P_L = P_T = 0.5$ and since $H_T = 2.06$ seconds (5), then

$$E = \frac{1750 \cdot G}{CAP \cdot C} - 1 \qquad (9)$$

A summary of the calculated left turning equivalents, E, for unprotected turning with and without left turn lanes is presented in Table 12. Two-phase equivalents were based on a G/C of 0.51 and three-phase equivalents on a G/C of 0.36. A 70 second cycle length was used in all cases.

The left turn capacity of an approach to an intersection which is controlled by a two-phase signal should not be expected to exceed those values given in Tables 10 and 11 unless field studies at the intersection indicate that higher values are possible. The left turn capacities given are based on the model previously developed and operating characteristics of Texas drivers.

If a left turn lane is not provided and the left turning volume exceeds 80 percent of the given practical capacity in Table 11, a channelized or otherwise designated left turn lane may initially be considered. If the left turn demand is heavy simultaneously with a heavy opposing approach volume, then a separate protected left turn signal may also be required at the intersection to reduce the magnitude of the delays suffered by the left turning traffic.

TABLE 12

LEFT TURNING EQUIVALENTS, E

| Intersection ● Signal Phasing | Traffic Movement | Number of Opposing Thru Lanes | Opposing Volume, CPH+, ECV | | | | |
|---|---|---|---|---|---|---|---|
| | | | 200 | 400 | 600 | 800 | 1000 |
| No Protected Turning | | | | | | | |
| ● Two-Phase | | | | | | | |
| ● No Bay | Left & Thru | 1 | 2.0 | 3.3 | 6.5 | 16.0* | 16.0* |
| | Left & Thru | 2 | 1.9 | 2.6 | 3.6 | 6.0 | 16.0* |
| | Left & Thru | 3 | 1.8 | 2.5 | 3.4 | 4.5 | 6.0 |
| ● With Bay | Left | 1 | 1.7 | 2.6 | 4.7 | 10.4* | 10.4* |
| | Left | 2 | 1.6 | 2.2 | 2.9 | 4.1 | 6.2 |
| | Left | 3 | 1.6 | 2.1 | 2.8 | 3.6 | 4.8 |
| ● Three-Phase | | | | | | | |
| ● No Bay | Left & Thru | 1 | 2.2 | 4.5 | 11.0* | 11.0* | 11.0* |
| | Left & Thru | 2 | 2.0 | 3.1 | 4.7 | 11.0* | 11.0* |
| | Left & Thru | 3 | 2.0 | 2.9 | 4.2 | 6.0 | 11.0* |
| ● With Bay | Left | 1 | 1.8 | 3.3 | 8.2* | 8.2* | 8.2* |
| | Left | 2 | 1.7 | 2.4 | 3.6 | 5.9 | 8.2* |
| | Left | 3 | 1.7 | 2.4 | 3.3 | 4.6 | 6.8 |
| Protected Turning | | | | | | | |
| ● No Bay | Left | Any | 1.2 | | 1.2 | | 1.2 |
| ● With Bay | Left | Any | 1.03 | | 1.03 | | 1.03 |

+ Includes total thru volume on the approach opposing the left turn being analyzed. The opposing volume also includes any turning volume(s) (left and/or right) for which no separate turning lane (bay) is provided.

* Turning capacity only at end of phase. Not recommended for design. Add additional thru lane, turning lane, or protected left turn phasing.

## EFFECTS OF SIGNAL PHASING AND LENGTH OF LEFT TURN BAY ON CAPACITY

Field observations of traffic flow at signalized intersections, having a protected left turn bay, suggest that the capacity of left turn phases can be reduced during some rush hour traffic conditions partially due to through vehicles blocking the entry of turning vehicles into the left turn bay. At times, the left turn bay may be blocked during the red phase of the signal such that the bay cannot fill; whereas, at other times, vehicles may even be blocked from entering on a portion of the left turn green phase. As traffic blockages begin to occur, the left turns may also begin to impede through vehicles and the effects on capacity and intersection congestion are compounded.

Reductions in left turn capacity generally occur as average traffic demands increase beyond some level associated with the storage length of the left turn bay and the cycle length of the signal. Shorter left turn bay lengths and longer cycle lengths are more susceptible to reductions in capacity. Shorter left turn bays mean fewer vehicles can be stored before blockages occur; whereas, longer cycles require more vehicles to be stored for a given volume level before a green is displayed.

Some signal phasing sequences, which improved traffic flow and left turn capacity over what previously existed, have occasionally been implemented. These improvements have been attained primarily by trial and error methods. Little information is readily available that describes the possible improvements that can be made by increasing the length of the left turn bay or by changing the phasing sequence.

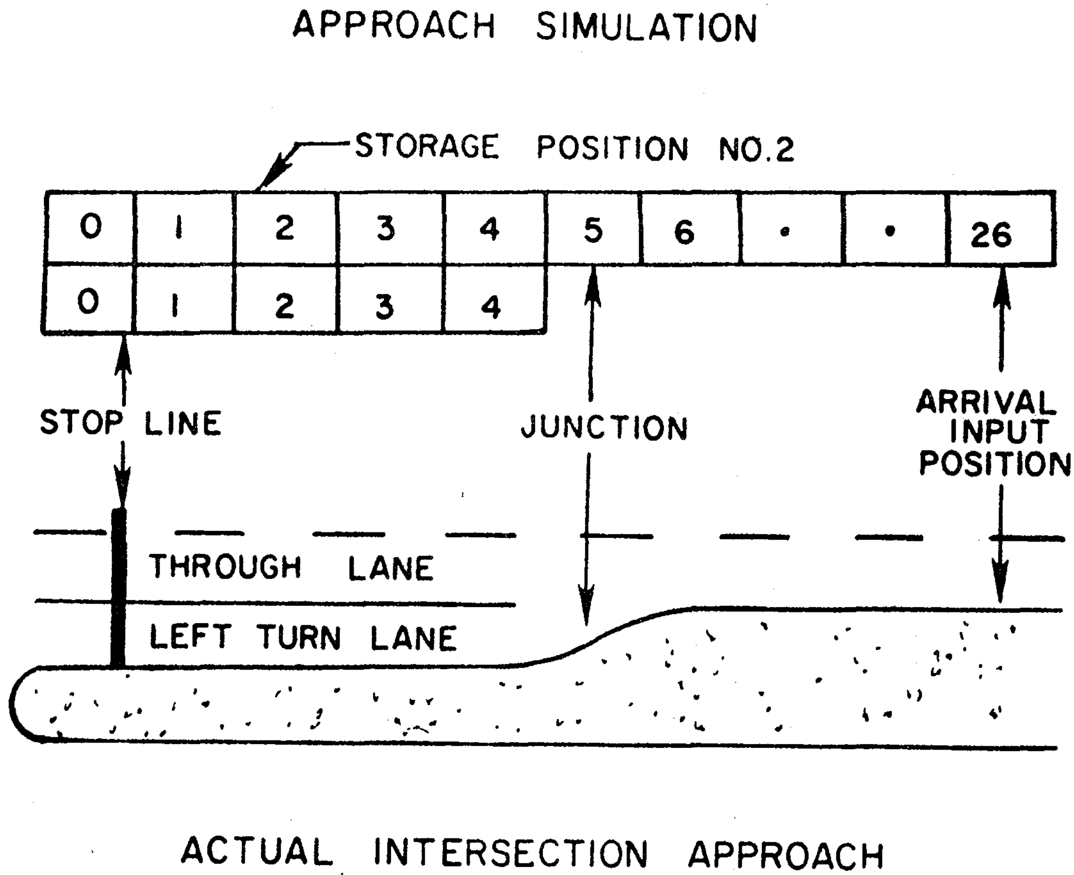
Basic design criteria for the length of the left turn bay have been previously related to the Poisson approach (13), but design trade-off relationships are not provided. Operational corrective treatments for an existing situation are also limited and not emphasized.

The mathematical analysis of the movement of through and left turning vehicles through an intersection under various traffic flow conditions, design configurations and signal phase sequences is extremely complicated, which no doubt is the principal reason for the lack of pertinent design and operations information on the subject.

## Approach

The periodic scan computer simulation approach was selected to investigate the previously identified left turn capacity problem. Due to the many variables involved and project time and budget constraints, it was recognized from the beginning that this study could not be a completely exhaustive one and that some questions would undoubtedly remain to be answered. Answers were sought, however, to basic cause-effect relationships and trends among: 1) capacity, 2) demand volume, 3) signal phasing, and 4) length of left turn bay.

Traffic operations were simulated on only one approach to the intersection, which included a protected left turn lane and adjacent through lane. A schematic of the approach model is depicted in Figure 8. The junction of the left turn and through lanes is represented by the first single storage position upstream of the left turn bay. The junction can be varied in the simulation program. Arriving vehicles are progressed through the left turning and adjacent through approaches by moving vehicles from one queue storage position to the next in discrete movements according to a defined strategy. These queue positions were defined to represent an average storage length of a passenger car stopped at the signal waiting for the green to be displayed.




# SIMULATION MODEL OF ACTUAL INTERSECTION APPROACH

FIGURE 8

Queue Characteristics

Field studies were conducted in College Station, Texas, to determine average vehicle storage spacing characteristics in feet per vehicle. Stations every 25 feet were marked along the median of the divided approaches and distances to the end of each queue were manually estimated for each cycle studied together with the number of vehicles in the queue to the recorded point. Vehicle queue lengths up to 429 feet long were measured. There were no significant grades on the approaches to the intersections and few trucks were in the traffic studied. A summary of these average storage lengths are presented in Table 13. A slightly conservative value of 25 feet per vehicle was assumed in the simulation program. (Left turn and through storage lengths were assumed to be the same.)

TABLE 13

AVERAGE PASSENGER CAR STORAGE LENGTHS OBSERVED

| Study Location | Left Turn Lane, Feet/Vehicle | Through Lane, Feet/Vehicle |
|---|---|---|
| University @ S. College | 23.9 | 25.2 |
| Texas @ University | 23.3 | 24.1 |
| AASHO Blue Book (14) | 25.0 | 25.0 |

Queue movement characteristics were also important inputs to the simulation model. A vehicle approaching the end of a queue was assumed to stop instantaneously when it reached the last unoccupied storage position. The stopped vehicle remained at that position until a specified time after the signal turned green. At this time, the vehicle began to move immediately at a speed that would result in the vehicle crossing the effective stop line at the front of the queue at the correct vehicle clearance time for the given vehicle position in the queue.

Studies were conducted at three high-type intersections in College Station of queue movement and clearance characteristics. These results are summarized in Figure 9. Also shown are two representative equations for describing the data. These equations are:

$$T_f = 2.0 + 1.0\ N_p \qquad (10)$$

and,

$$T_c = 2.0 + 2.0\ N_p \qquad (11)$$

where:

$T_f$ = Time after start of green for the vehicle in queue storage position number $N_p$ to begin moving forward, sec.

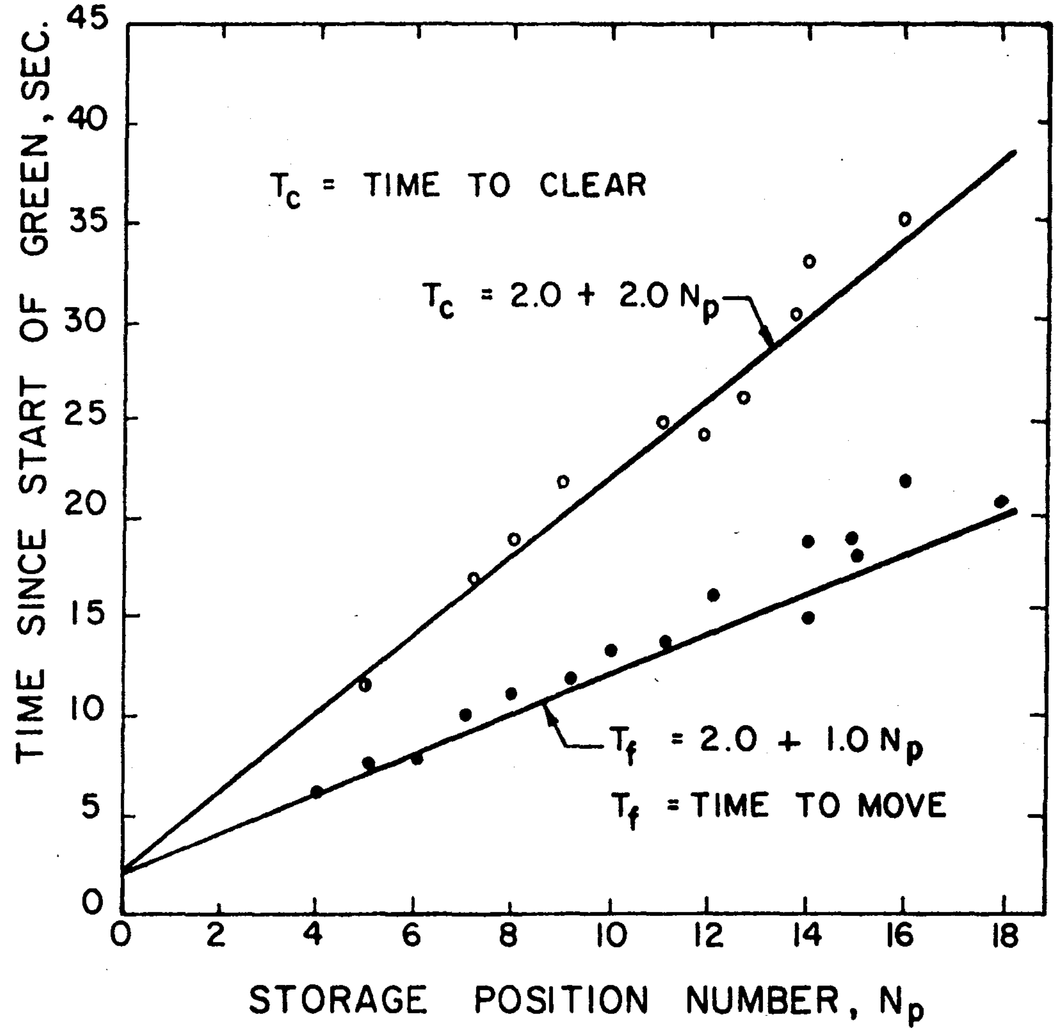
$T_c$ = Time after start of green for the vehicle in queue storage position number $N_p$ to clear the stop line on the approach, sec.

$N_p$ = Queue storage position number (Figure 8) for either left turn or through vehicles.

These equations were specifically selected to expedite the simulation process. They are obviously descriptive of the measured characteristics, as shown in Figure 9, but they were not determined by a formal optimization process such as linear regression. The simulation process was greatly simplified by assuming that all the coefficients of the previous two equations had integer values.

## Simulation Inputs

The following variables are inputs to the intersection approach simulation program:




MOVEMENT CHARACTERISTICS OF
LEFT TURNING VEHICLES
AT HIGH-TYPE INTERSECTIONS

FIGURE 9

1. Total lane approach volume, veh./hr.
2. Percent of total approach volume turning left.
3. Cycle length of signal, sec.
4. Length of left turn bay storage, cars.
5. Green time of left turn signal, sec.
6. Green time of through movement signal, sec.
7. Leading or lagging left turns (single or dual).

Simulation Model

The following is a brief outline of the simulation model in statement format:

1. The left turn and adjacent through lanes are assumed to be divisible into discrete car length storage positions, as was illustrated in Figure 8.
2. The length of the left turn lane is defined by the first upstream single storage position, or the junction.
3. The simulation scans the system every second in the periodic scan mode, updating from front to back all storage positions that should be changed. Operational measures of effectiveness are recorded.
4. Vehicles are assumed to arrive according to the Poisson distribution and are input to the system at storage position 26.
5. Vehicles were not permitted to enter the system at headways less than 2.0 seconds.
6. Every input car (vehicle) is tagged as being a left turn or through car in a random manner at the desired average rate of left turners.
7. Every storage position can have only one of three states:
   a. Empty

b. Moving (M)

c. Stopped or queued (Q)

8. Moving cars (M) can move forward only into an empty position.

9. Where possible, all moving cars (M) move forward into the next position every one second scan period.

10. When a moving car (M) cannot move forward into the next position, the status of the car (and storage position) is changed to a queued car, and is delayed one second.

11. When a queued car occupies the next position immediately behind another queued car for the scan period being analyzed, the car remains queued and is delayed one second.

12. When the signal is red, vehicle position zero acts like a queued car such that no cars may leave position one (1) and enter the intersection.

13. When the signal turns green, vehicle position zero is immediately set to the moving state. Two scanning periods later, the queued car in position one (1) is changed to the moving state (M), if a vehicle is present.

14. When a queued car (Q) is behind a moving car (M) or an empty space, its status is changed to a moving car (M) but it does not move forward until the next scan period. It is, therefore, delayed one second.

(The execution of these queue behavior rules are illustrated in Figure 10. The movement and clearance times of the queues obey the characteristic equations, Eqs. 10 and 11, as required to simulate the actual traffic conditions).

Signal — Queue Storage Position, $N_p$

Time

Periodic Scan
One Second Time Intervals

| 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|
| R | . | . | . | . | . | . | . | . | . | . |
| R | . | . | . | . | . | . | . | . | . | M |
| R | . | . | . | . | . | . | . | . | M | . |
| R | . | . | . | . | . | . | . | M | . | M |
| R | . | . | . | . | . | . | M | . | M | . |
| R | . | . | . | . | . | M | . | M | . | M |
| R | . | . | . | . | M | . | M | . | M | . |
| R | . | . | . | M | . | M | . | M | . | M |
| R | . | . | M | . | M | . | M | . | M | . |
| R | . | M | . | M | . | M | . | M | . | . |
| R | M | . | M | . | M | . | M | . | . | . |
| R | Q | M | . | M | . | M | . | . | . | . |
| R | Q | Q | M | . | M | . | . | . | . | . |
| R | Q | Q | Q | M | . | . | . | . | . | . |
| G | Q | Q | Q | Q | . | . | . | . | . | . |
| G | Q | Q | Q | Q | . | . | . | . | . | . |
| G | M | Q | Q | Q | . | . | . | . | . | . |
| G | . | M | Q | Q | . | . | . | . | . | . |
| G | M | . | M | Q | . | . | . | . | . | . |
| G | . | M | . | M | . | . | . | . | . | . |
| G | M | . | M | . | . | . | . | . | . | . |
| G | . | M | . | . | . | . | . | . | . | . |
| G | M | . | . | . | . | . | . | . | . | . |
| G | . | . | . | . | . | . | . | . | . | . |

Direction of Movement

EXAMPLE OF ONE LANE QUEUE MOVEMENT

FIGURE 10

15. Cars at the junction position can be either left turners or through cars. Left turners obey the status of the next lower position in the left turn lane while through vehicles obey the status of the next lower position in the through lane. If a through vehicle is queued in the junction position, then no left turn vehicle can enter the left turn bay until the through vehicle has cleared the junction, and vice versa. Through vehicles can block left turners and left turns can block throughs.

## Simulation Outputs

Several traffic flow measures of effectiveness are calculated by the simulation program. These are:

1. Output volume for each movement, veh./hr.
2. Delay per vehicle for each movement, sec./veh.
3. Frequency plots of queue length and individual delay for each vehicle.

## Program Testing

A computer program was written in a combination of FORTRAN IV and ASSEMBLER to reduce simulation costs. This program was tested for realism in two ways. Firstly, computer printouts were made of the simulated movement of vehicles on the approaches as the signals changed from green to red over several cycles. Movements of individual vehicles were observed for realism and obedience to the simulation rules for movement, blockage and stoppage. Secondly, unimpeded delays calculated from the simulation program were found to be consistent with the results obtained from Webster's theoretical delay equation. In addition, subsequent simulated delay calculations followed expected trends as queue interactions and blockages occurred.

Simulation Results

The simulation results were most encouraging with consistent trends and realistic outcomes. Many of the results were determined over 300 simulated cycles of operation for each data point. No less than 60 cycles were ever used. Five cycles were used to initialize the simulation model before the analysis cycles were simulated from which average values of the measures of effectiveness were calculated.

*Delay* - The initial analysis phase of the simulation study focused primarily on evaluating the effects of left turn bay length and signal phasing on average vehicle delay. Two signal phasing arrangements were studied. These were the leading left turn phase sequence and the lagging left turn phase sequence. Cycle lengths of 60 and 80 seconds were studied. Approximately equal nominal volume-to-capacity (saturation) ratios were simulated for both the left turn and through movements. A nominal saturation ratio is defined as being the normal demand on the movement divided by the phase's capacity when the left turn bay is sufficiently long such that no blockages or interactions occur between the left turns and through movement. In other words, the left turn saturation flow is assumed to be 1700 CPHG, the nominal value for long bay lengths (5).

Simulation results of these delay studies are presented in Tables 14 - 17. Delay increased as expected with increasing volume, nominal saturation ratio and cycle length. Delay also increased as the length of the left turn bay is reduced. Lagging green resulted in a slight reduction in delay for the conditions studied. Nominal volume-to-capacity (saturation) ratios of about 0.6 - 0.8 appear to be critical for bay lengths of 5 to 10 vehicles (125 - 250 feet) insofar as experiencing increased blockages and delay are concerned. These results indicate that the actual volume-to-capacity ratio for the shorter bay

TABLE 14

SIMULATED AVERAGE DELAY PER VEHICLE PER MOVEMENT WHERE GREEN TIMES PROPORTIONED TO YIELD UNIFORM DEMAND TO CAPACITY RATIOS FOR A 60-SECOND CYCLE LENGTH WITH A LEADING LEFT TURN

| Volume to Capacity, X | Left Turn Demand, VPH | Through Demand, VPH | Left Turn Bay Length, Vehicles | Left Turn Delay, Sec./Veh. | Through Delay, Sec./Veh. |
|---|---|---|---|---|---|
| .21 | 80 | 120 | 1 | 22 | 16 |
| | | | 5 | 22 | 16 |
| | | | 10 | 21 | 16 |
| | | | 20 | 21 | 16 |
| .42 | 160 | 240 | 1 | 39 | 26 |
| | | | 5 | 24 | 17 |
| | | | 10 | 24 | 17 |
| | | | 20 | 23 | 17 |
| .64 | 240 | 360 | 1 | 133 | 112 |
| | | | 5 | 39 | 29 |
| | | | 10 | 28 | 18 |
| | | | 20 | 28 | 18 |
| .85 | 320 | 480 | 1 | 121 | 106 |
| | | | 5 | 90 | 82 |
| | | | 10 | 56 | 45 |
| | | | 15 | 39 | 32 |
| | | | 20 | 35 | 30 |
| .95 | 360 | 540 | 1 | 137 | 117 |
| | | | 5 | 100 | 83 |
| | | | 10 | 94 | 57 |
| | | | 20 | 81 | 35 |

TABLE 15

SIMULATED AVERAGE DELAY PER VEHICLE PER MOVEMENT WHERE GREEN TIMES PROPORTIONED TO YIELD UNIFORM DEMAND TO CAPACITY RATIOS FOR A 60-SECOND CYCLE LENGTH WITH A LAGGING LEFT TURN

| Volume to Capacity, X | Left Turn Demand, VPH | Through Demand, VPH | Left Turn Bay Length, Vehicles | Left Turn Delay, Sec./Veh. | Through Delay, Sec./Veh. |
|---|---|---|---|---|---|
| .21 | 80 | 120 | 1 | 22 | 16 |
| | | | 5 | 21 | 16 |
| | | | 10 | 21 | 16 |
| | | | 20 | 21 | 16 |
| .42 | 160 | 240 | 1 | 33 | 31 |
| | | | 5 | 23 | 17 |
| | | | 10 | 23 | 17 |
| | | | 20 | 22 | 17 |
| .64 | 240 | 360 | 1 | 125 | 121 |
| | | | 5 | 34 | 27 |
| | | | 10 | 28 | 18 |
| | | | 20 | 28 | 18 |
| .85 | 320 | 480 | 1 | 114 | 112 |
| | | | 5 | 85 | 81 |
| | | | 10 | 54 | 42 |
| | | | 15 | 38 | 31 |
| | | | 20 | 35 | 30 |
| .95 | 360 | 540 | 1 | 133 | 130 |
| | | | 5 | 91 | 86 |
| | | | 10 | 90 | 50 |
| | | | 20 | 80 | 32 |

TABLE 16

SIMULATED AVERAGE DELAY PER VEHICLE PER MOVEMENT WHERE GREEN TIMES PROPORTIONED TO YIELD UNIFORM DEMAND TO CAPACITY RATIOS FOR AN 80-SECOND CYLCE LENGTH WITH A LEADING LEFT TURN

| Volume to Capacity, X | Left Turn Demand, VPH | Through Demand, VPH | Left Turn Bay Length, Vehicles | Left Turn Delay, Sec./Veh. | Through Delay, Sec./Veh. |
|---|---|---|---|---|---|
| .21 | 80 | 120 | 1 | 27 | 20 |
| | | | 5 | 27 | 20 |
| | | | 10 | 26 | 20 |
| | | | 20 | 27 | 20 |
| .42 | 160 | 240 | 1 | 49 | 34 |
| | | | 5 | 30 | 22 |
| | | | 10 | 30 | 22 |
| | | | 20 | 30 | 21 |
| .64 | 240 | 360 | 1 | 150 | 127 |
| | | | 5 | 67 | 49 |
| | | | 10 | 34 | 24 |
| | | | 20 | 35 | 24 |
| .85 | 320 | 480 | 1 | 136 | 116 |
| | | | 5 | 103 | 89 |
| | | | 10 | 64 | 52 |
| | | | 15 | 42 | 35 |
| | | | 20 | 40 | 34 |
| .95 | 360 | 540 | 1 | 164 | 152 |
| | | | 5 | 146 | 125 |
| | | | 10 | 115 | 94 |
| | | | 20 | 95 | 44 |

TABLE 17

SIMULATED AVERAGE DELAY PER VEHICLE PER MOVEMENT WHERE GREEN TIMES PROPORTIONED TO YIELD UNIFORM DEMAND TO CAPACITY RATIOS FOR AN 80-SECOND CYCLE LENGTH WITH A LAGGING LEFT TURN

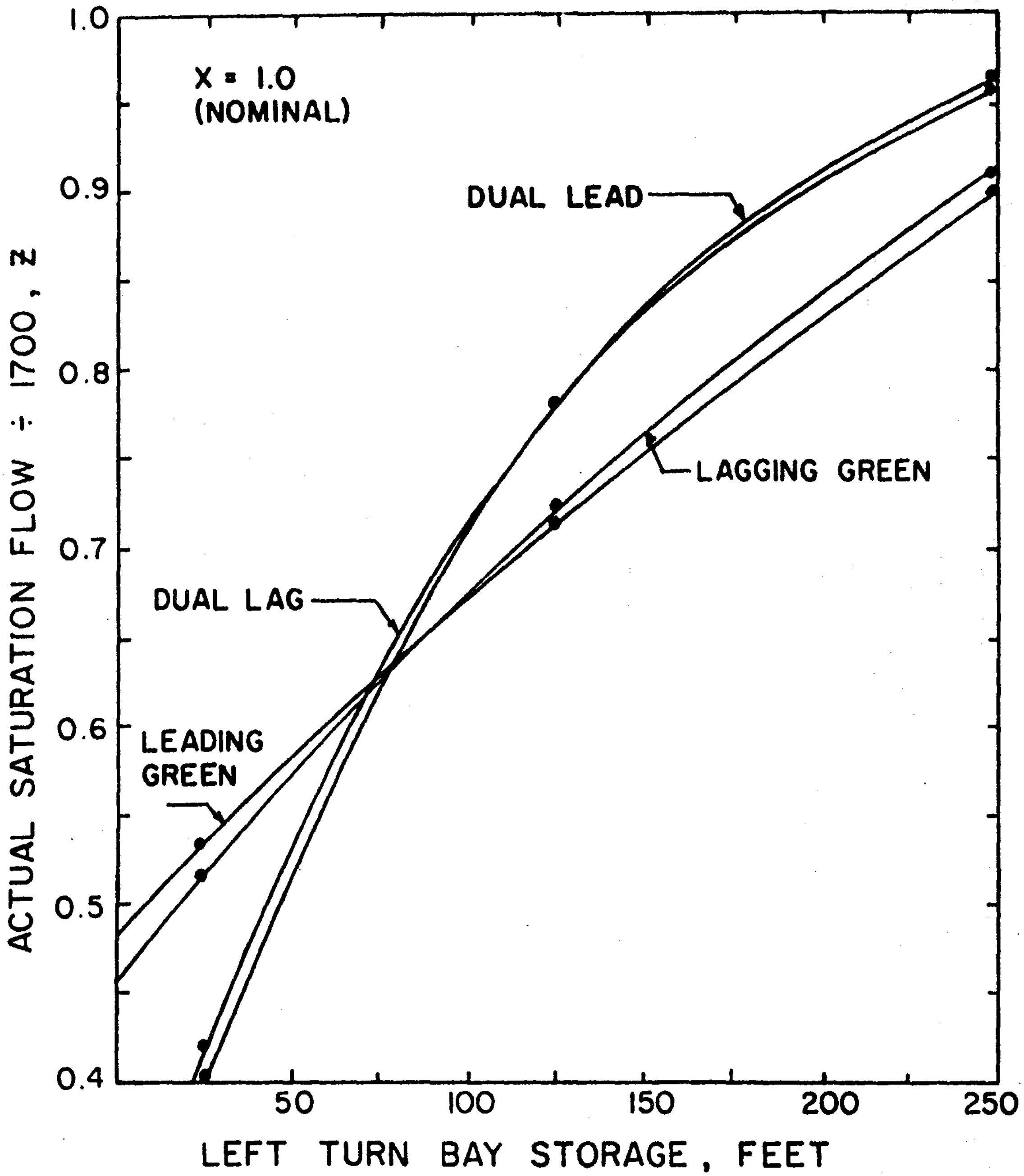
| Volume to Capacity, X | Left Turn Demand, VPH | Through Demand, VPH | Left Turn Bay Length, Vehicles | Left Turn Delay, Sec./Veh. | Through Delay, Sec./Veh. |
|---|---|---|---|---|---|
| .21 | 80 | 120 | 1 | 27 | 23 |
| | | | 5 | 26 | 20 |
| | | | 10 | 26 | 20 |
| | | | 20 | 26 | 20 |
| .42 | 160 | 240 | 1 | 48 | 43 |
| | | | 5 | 29 | 22 |
| | | | 10 | 29 | 21 |
| | | | 20 | 29 | 21 |
| .64 | 240 | 360 | 1 | 138 | 135 |
| | | | 5 | 57 | 50 |
| | | | 10 | 35 | 24 |
| | | | 20 | 34 | 23 |
| .85 | 320 | 480 | 1 | 130 | 127 |
| | | | 5 | 96 | 94 |
| | | | 10 | 60 | 51 |
| | | | 15 | 42 | 35 |
| | | | 20 | 40 | 35 |
| .95 | 360 | 540 | 1 | 143 | 140 |
| | | | 5 | 103 | 98 |
| | | | 10 | 97 | 73 |
| | | | 20 | 95 | 43 |

lengths must be considerably higher than the nominal value, and that the saturation flow (and capacity) must be correspondingly less than 1700 CPHG.

*Left Turn Capacity* - Left turn capacity and saturation flow studies were conducted in view of the previous findings. Most of these subsequent simulation runs were made at nominal volume-to-capacity ratios of about 1.0. During these capacity studies, two additional phase sequences of left turns first (dual lefts leading) and through movements first (dual lefts lagging) were added. Average results of these simulation studies are depicted in Figure 11. For the conditions evaluated, some differences in saturation flow (capacity) were observed with lagging and leading left turn green phasing being slightly better for extremely short bay lengths; whereas, dual lefts leading or lagging performed better at bay lengths of 5 to 10 vehicles.

It is important to note, however, that all of the phasing arrangements experienced reductions in capacity for these conditions, a nominal saturation ratio of 1.0. A left turn bay length of 5 vehicles (125 feet) experienced a 20 to 30 percent reduction in capacity. General reductions in capacity were observed in most of the simulation runs made (90 were made) with greater reductions in capacity occurring at higher volume conditions. Similar reductions in capacity were experienced by the adjacent through lane. Reductions in capacity also varied with the percent of traffic turning left and the green split between the two movements in an apparently complex manner. No overall mathematical model was developed which included all the variables that were identified.

To aid design and operations engineers in estimating a reasonable capacity and saturation flow for a given left turn bay storage length, the combined simulation results of all 90 runs were pooled together from which the following




**REDUCTION IN LEFT TURN SATURATION FLOW DUE TO PHASING**

FIGURE 11

multiple regression model, having a statistical R-square value of 0.80, was developed:

$$Z = 0.98 - 0.14 \cdot V - 0.19 \cdot X_L \cdot V + 0.24 \cdot X_T \cdot V \qquad (12)$$

where:

$Z$ = Actual left turn saturation flow divided by the nominal saturation flow ($Z = S/1700$).

$X_L$ = Nominal left turn saturation ratio.

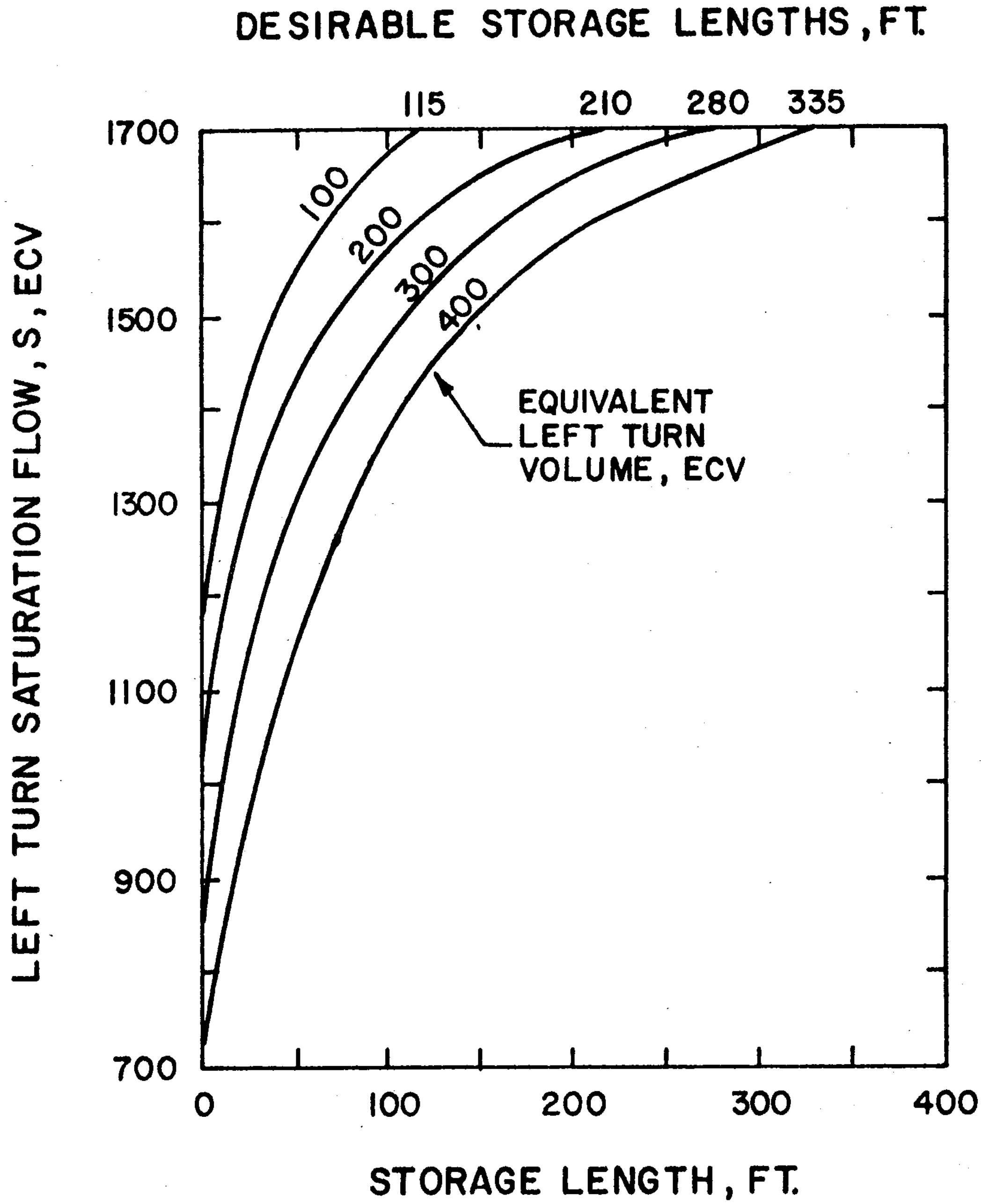
$X_T$ = Nominal through movement saturation ratio.

$V$ = $X_L \cdot X_T \cdot K$, where K is the average number of left turns arriving per cycle divided by the storage length of the bay.

This equation was used to develop saturation flow and storage design curves shown in Figure 12. Inputs selected for design were $X_L = 0.8$, $X_T = 0.8$, nominal saturation flow of 1700 CPHG, an assumed storage requirement of 25 feet per car, and a cycle length of 75 seconds. The saturation flow, S, for left turns in Figure 12 was calculated from $S = 1700 \cdot Z$. Volumes are Equivalent Car Volumes (ECV) in cars per hour.

At the top of Figure 12 are located the left turn bay storage lengths that will result in practically no reduction in capacity for the intercept left turn volume level. These storage lengths can be used as practical design storage lengths. Interpolated storage lengths can be calculated for intermediate left turn volumes. These storage lengths compared favorably as design values for 12 queue distributions of vehicle storage available from the simulation runs. Only 12 plots of queue distributions were made due to computer plotting costs.

A special set of simulation runs was made to test and illustrate the capacity results of Figure 12. An intersection was assumed to have a left turn bay of 25 feet (1 car) and a leading left turn signal phasing sequence. It was




SATURATION FLOW OF LEFT TURN PHASE AS A FUNCTION OF BAY STORAGE LENGTH AND TURNING VOLUME

FIGURE 12

assumed that the left turning volume was 320 equivalent cars per hour (ECV) and the through movement volume was 480 ECV. Corresponding (effective) green times were 14 and 20 seconds. Nominal volume-to-capacity (saturation) ratios of about 0.8 existed on both movements. According to Figure 12, however, the 25-foot bay length combined with a 320 ECV left turning volume should result in a large reduction in left turn capacity and saturation flow from 1700 CPHG to an actual flow of about 1060 CPHG. If this reduction in capacity does exist, then the given conditions are overloaded and large delays should result. The actual saturation ratios, X, would be about 1.30 on both movements.

Table 18 illustrates the consequences of the short bay and reduced capacity. The first row of Table 18 contains the initially given conditions and results. Low flows and excessively large delays occurred. As the movement green times are increased, flows climb to the volume levels being simulated while delays drop to acceptable levels. In order to compensate for the 60 percent reduction in saturation flow estimated from Figure 12 (1700/1060 = 1.60, a 60% reduction) and provide actual saturation ratios of about 0.8, similar increases in green are required. Green times of 22 seconds for the left turn and 32 seconds for the through movement provide the needed 57-60 percent increase. It would appear for this one extreme example that the reduction in capacity is slightly larger than estimated by Figure 12, although delay variations are very sensitive in the region being analyzed. However, the general trend and practical magnitude of expected left turn saturation flows given in Figure 12 are supported.

## Left Turn Bay Length - Modified Poisson Approach

The previous simulation studies of the capacity and desirable length of left turn bays were an outgrowth of an earlier project analysis of the length of left turn bays where a more simplified approach or model was considered.

TABLE 18

SIMULATION EXAMPLES OF EFFECTS OF REDUCTION
IN SATURATION FLOW DUE TO SHORT BAY LENGTH

| Green, Sec. | | Flow, ECV | | Green Increase, % | | Delay, Sec./Veh. | |
|---|---|---|---|---|---|---|---|
| Left | Through | Left+ | Through* | Left | Through | Left | Through |
| 14 | 20 | 220 | 320 | 0 | 0 | 121 | 107 |
| 18 | 26 | 265 | 402 | 28 | 30 | 87 | 74 |
| 22 | 30 | 315 | 459 | 57 | 50 | 60 | 51 |
| 22 | 32 | 316 | 467 | 57 | 60 | 53 | 44 |
| 24 | 32 | 319 | 481 | 71 | 60 | 34 | 27 |
| 26 | 34 | 317 | 480 | 85 | 70 | 23 | 18 |

+Left turn simulated volume = 320 cars/hr., ECV.
*Through simulated volume = 480 cars/hr., ECV.

This earlier approach is an extension of the Poisson procedure frequently used by practicing traffic engineers. The Poisson approach forms the basis for storage length recommendation given in the AASHO Red Book - "A Policy on Design of Urban Highways and Arterial Streets - 1973". To quote AASHO (13). "At signalized intersections, the required storage length depends on the cycle length, the signal phasing arrangement, and rate of arrivals and departures of left-turning vehicles. The storage length should be based on 1.5 to 2 times the average number of vehicles that would store per cycle, predicated on the design volume."

The modified Poisson approach to be presented subsequently provides guidance to determining the relationship between the multiplier (1.5 to 2 times) and design left turning volumes. In addition, these results will support the previously recommended storage bay lengths given in Figure 12. Other important interrelationships will be presented between design and operational variables.

Miller (8) has presented the following equation which estimates the average number of vehicles remaining in the queue at a pretimed signal at the end of the green phase:

$$A = \frac{e^{-1.3\left[\frac{1-X}{X} \cdot \sqrt{\frac{qC}{X}}\right]}}{2\,(1-X)} \tag{13}$$

where:

A = Average number of vehicles in the left turn bay at end of green.

q = Left turn flow rate, veh./sec.

C = Cycle length, sec.

X = Left turn saturation ratio, qC/gs.

g = Left turn effective green, sec.

s = Left turn saturation flow, veh./sec. green.

The number of left turn vehicles arriving during the effective red which must be stored in the left turn bay in addition to "A" is

$$B = q \cdot R \tag{14}$$

where:

B = Number of left turn vehicles arriving on red.

R = Left turn effective red time, sec.

q = Left turn flow rate, veh./sec.

After the left turn signal turns green, additional left turn vehicles are joining the rear of the stopped left turn queue for a time T (See Equation 10.) until it is time for the vehicle in queue position $N_p$ to begin moving forward. If $T_f$ is set equal to the arrival time of vehicle $N_p$ after the start of green, then

$$T_f = 2 + 1 \cdot N_p = (N_p - A - B + 2 \cdot q) / q \tag{15}$$

and $$N_p = \frac{A + B}{1 - q} \tag{16}$$

The left turn flow rate, q, should be higher than the average left turn flow rate to account for the short-term peak flows that occur cycle-by-cycle during random (Poisson) flow (which is assumed). The flow rate which was selected will not be exceeded more than once during 50 percent of the peak 15-minute periods of the design hour during the year. That is;

$$\Sigma P_q \cdot \frac{3600}{C} \cdot \frac{1}{4} = 0.50$$

where:

$\Sigma P_q$ = Cumulative Poission probability of exceeding flow rate q, (10).

C = Cycle length, sec.

Letting the design storage capacity of the bay be $N_p$, which in turn is calculated from q, then the above probability of overflow criterion can be expressed in design level of performance terms as follows: "The odds are 50/50 that the left turn storage demands will exceed capacity only once during a peak 15-minute period of the design hour." Table 19 summarizes input values used to develop modified Poisson left turn bay storage requirements from Equation 16.

TABLE 19

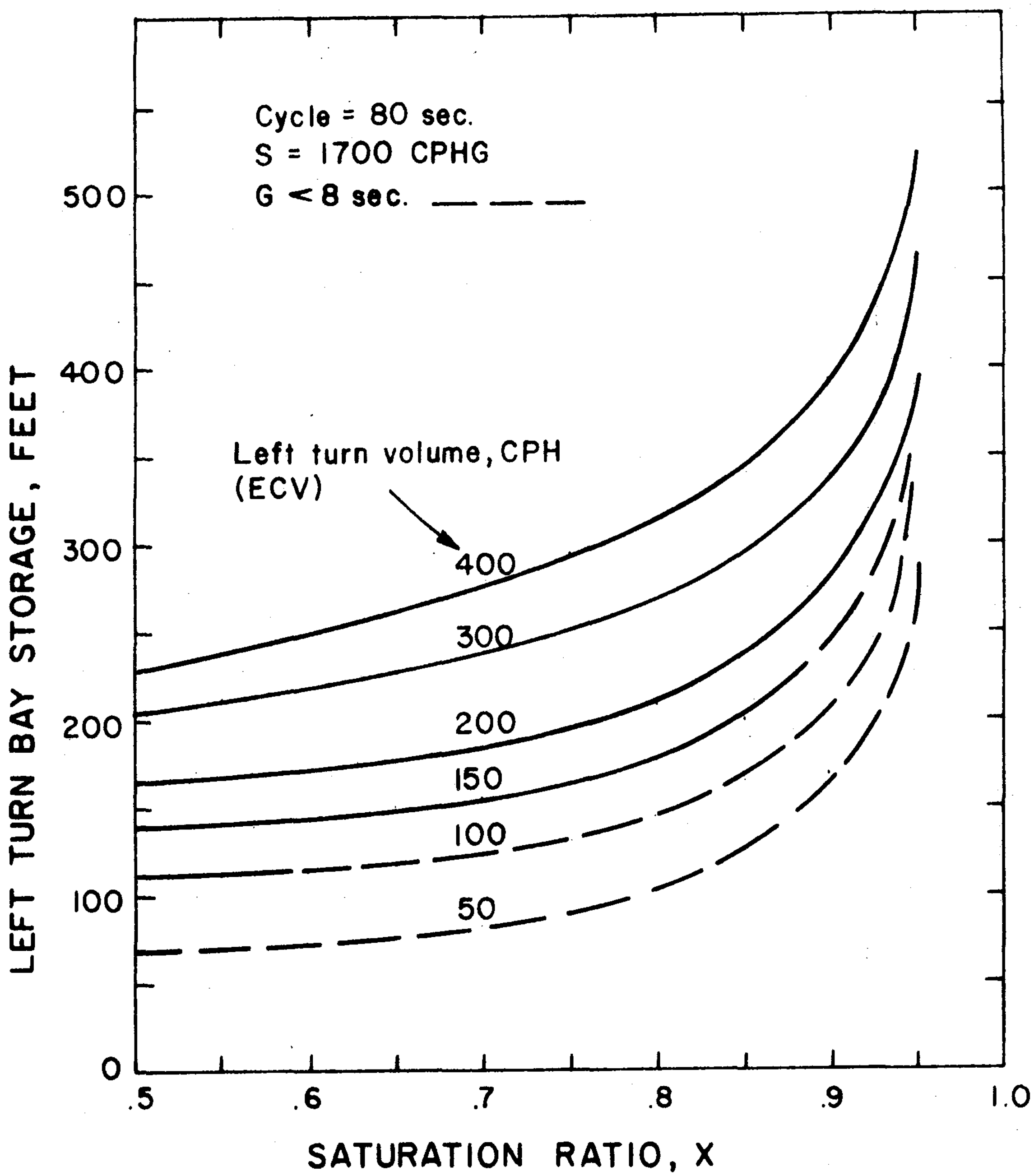
INPUT VALUES FOR MODIFIED POISSON APPROACH FOR CALCULATING LEFT TURN BAY STORAGE REQUIREMENTS

| Cycle Length C, sec. | 60 | 70 | 80 | 90 | 100 |
|---|---|---|---|---|---|
| $\Sigma P_q$ | .033 | .039 | .044 | .050 | .055 |
| Left Turn Volume During Peak 15 Min. ECV | Input Left Turn Volume, q · 3600, CPH ECV | | | | |
| 50 | 132 | 129 | 122 | 116 | 108 |
| 100 | 234 | 216 | 207 | 200 | 190 |
| 150 | 312 | 293 | 279 | 268 | 259 |
| 200 | 396 | 365 | 347 | 336 | 324 |
| 300 | 540 | 509 | 486 | 468 | 454 |
| 400 | 672 | 643 | 617 | 596 | 576 |

Results of this modified Poisson approach are presented in Figures 13, 14 and 15. Figure 13 shows that the length of storage required increases with left turn volume and with the signal phase's saturation ratio, X. This latter fact is important for several reasons. The normal Poisson approach to left turn bay storage design (13) does not account for the signal's operating saturation ratio. If the saturation ratio exceeds 0.85, the length of storage needed to reduce the likelihood of interactions and blockages increases dramatically. As was shown in the earlier section on simulation of left turns, blockages cause a reduction in saturation flow (capacity), further compounding the problem. A maximum saturation ratio of 0.8 would appear practical for use in design.

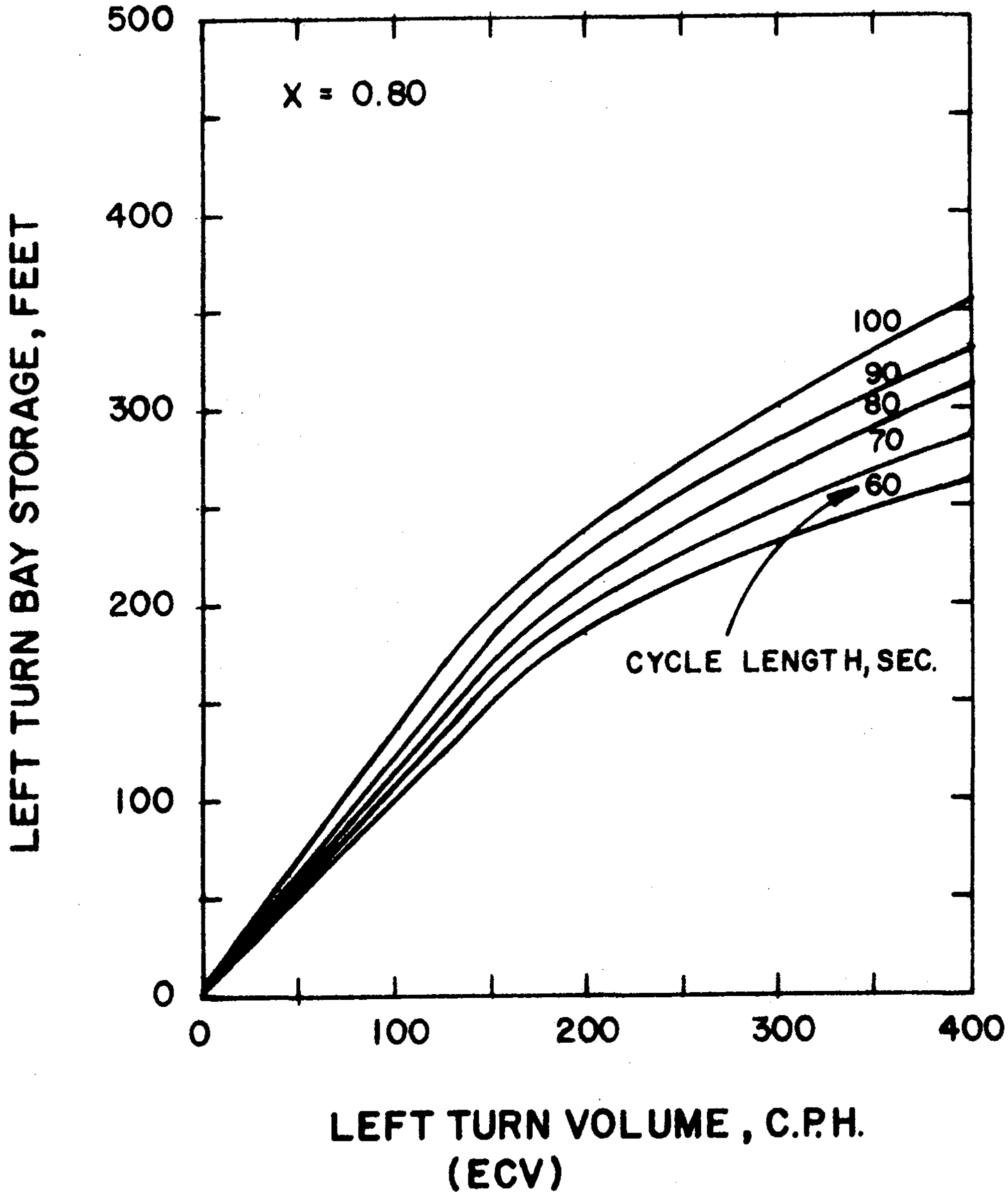
Figure 14 presents the length of storage required as a function of cycle length and left turning volume for the assumed design saturation ratio of 0.8. The storage length increases with increasing cycle length, but the rate of increase is only about 40 percent as large as suggested by the normal Poisson approach. This is due to the fact that while longer cycle lengths require more vehicles to be stored per cycle, there are fewer cycles that have the opportunity to "fail" during the peak 15 minute period of the design hour. This reduction is not accounted for in the normal Poisson approach.

Figure 15 presents comparative results between the 1973 AASHO Red Book design guidelines (13) previously noted in this section and results obtained from the modified Poisson approach using a saturation ratio of 0.8 and a cycle length of 75 seconds. The variable "m" in Figure 15 is the normal Poisson parameter "average number of left turns per cycle". The Red Book guidelines "1.5 - 2 m" bound the modified Poisson curve up to left turn volumes of 350 vehicles per hour. The length of left turn bay required in Figure 15



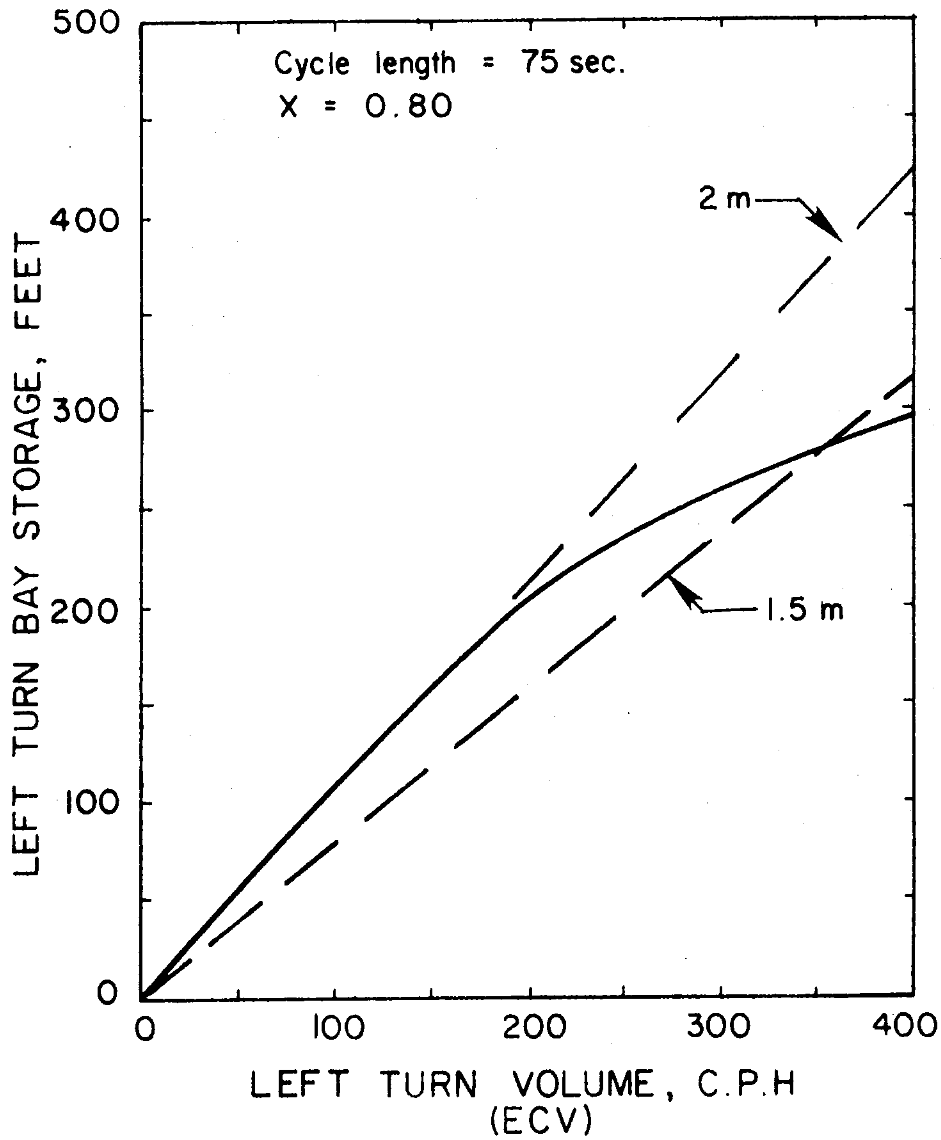

**LEFT TURN BAY STORAGE**
**VERSUS SATURATION RATIO**
**(MODIFIED POISSON)**

FIGURE 13




LEFT TURN BAY STORAGE VERSUS TURNING VOLUME FOR VARIOUS CYCLE LENGTHS (MODIFIED POISSON)

FIGURE 14




**LEFT TURN BAY STORAGE VERSUS TURNING VOLUME**

(MODIFIED POISSON)

FIGURE 15

is within ten percent of those storage lengths shown at the top of Figure 12 which were developed from the simulation analyses. In general, cycle lengths in excess of 80 seconds in Figure 14 result in slightly longer storage requirements than those given in Figure 12.

On the basis of supporting results of two different approaches, it is recommended that the storage requirements for left turns be determined from either Figure 12 or Figure 14. Figure 14 should only be utilized if the cycle length used will result in longer storage requirements than those given in Figure 12.

## FIELD EVALUATION OF SIGNAL OPERATIONS

Sometimes it is desired to evaluate the operations of an existing signal system. Extensive field procedures have frequently been used to measure delays on the approaches to the intersection and other operational measures of effectiveness. The following procedure is presented to assist in evaluating operating conditions with a minimum of field personnel and to provide a basis for evaluating the level of service at pretimed signalized intersections.

### Level of Service Measures

Table 20 presents the measures of effectiveness which are evaluated together with previously published level of service criteria for each. Different measures may yield slightly different levels of service when evaluated for the same approach, particularly when comparing delay with the other measures.

TABLE 20

LEVEL OF SERVICE CRITERIA FOR OPERATIONAL MEASURES OF EFFECTIVENESS ON SIGNALIZED MOVEMENTS

| OPERATIONAL MEASURE | LEVEL OF SERVICE A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| Saturation Ratio, $X^+$ | $\leq .6$ | $\leq .7$ | $\leq .8$ | $\leq .85^y$ | $\leq 1.0$ | $>1.0$ |
| Probability of Clearing Queues, $P_c^*$ | $\geq .95$ | $\geq .90$ | $\geq .75$ | $\geq .50$ | $<.50$ | |
| Average Approach$^x$ Delay, d, sec./veh. | $\leq 15$ | $\leq 30$ | $\leq 45$ | $\leq 60$ | $> 60$ | |

Source: $^+$ Reference (15), $^*$ Reference (5), $^x$ Reference (16), $^y$ Value modified in this project.

## Field Data Collection

One person would normally be required to collect the necessary field data for each approach studied at the same time; however, a skilled observer might be able to study more than one. The following data are required for the time period being evaluated:

1. Cycle length, C, in seconds.
2. Green time of movement, G, in seconds. (No yellow).
3. Calculate the effective red time, R, from C minus G.
4. Measure the time it takes each queue to clear its approach after the start of green (visually average the lanes).
5. Record each time the phase clears the queue.
6. Calculate the average queue clearance time, T, in seconds.

Table 21 illustrates field data recorded for an approach during moderate rush hour traffic flow. Two cycles failed to clear the queues during this study.

## Estimated Probability of Clearing Queues and Delay

Estimates of the probability of clearing queues using Miller's model and average vehicle delay in seconds per vehicle using Webster's approximation model (17) are obtained by applying the following procedure to Figure 16. These procedures will be illustrated for an example recorded data set presented in Table 21.

It is first necessary to calculate the average saturation ratio, X, existing on the approach during the study period from:

$$X = \frac{T - 2.0}{G} \cdot \frac{C}{R + T - 2.0} \qquad (17)$$

where the variables are defined in Table 21. Thus, the saturation ratio is

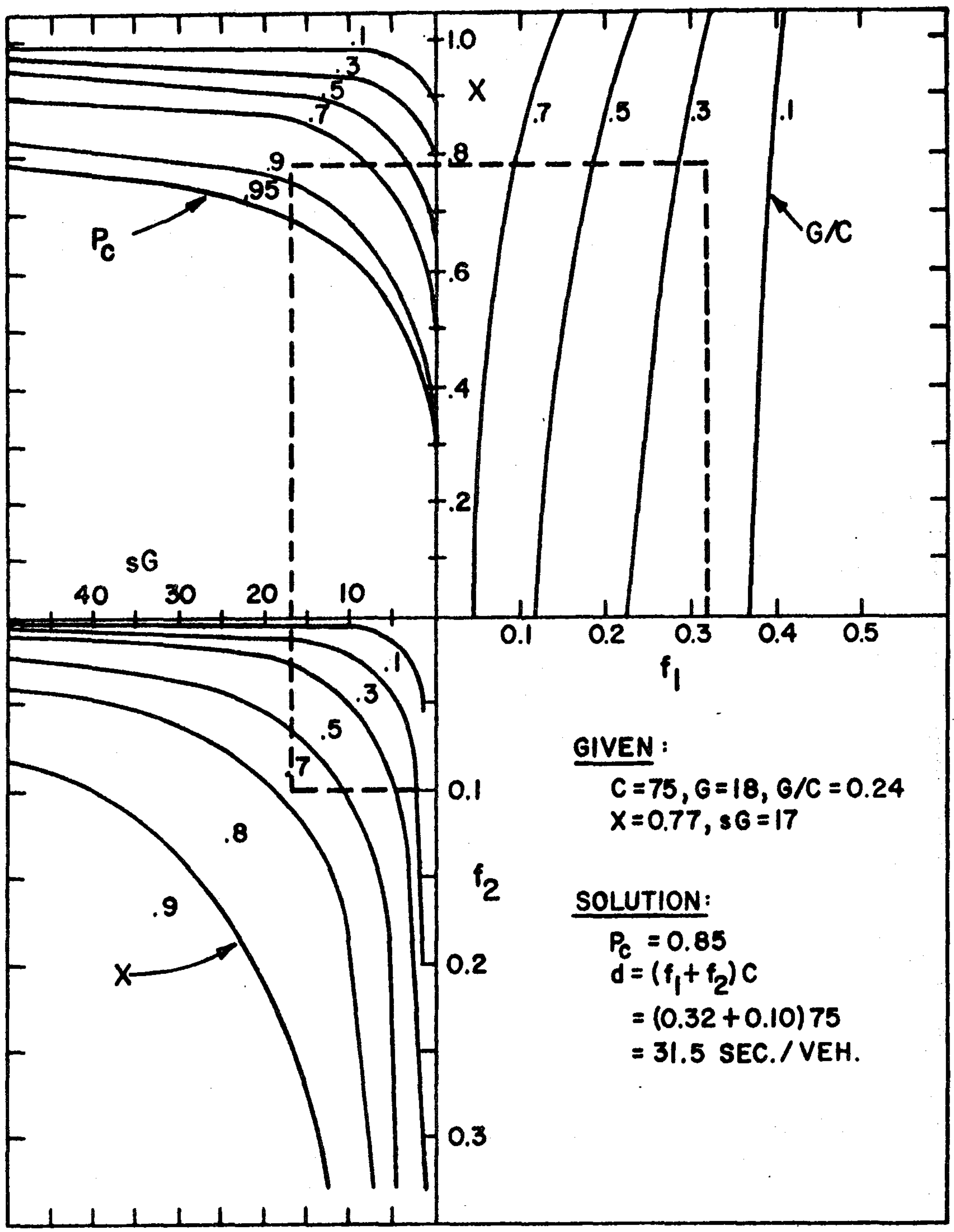
$$X = \frac{15.0 - 2.0}{18} \cdot \frac{75}{57 + 15.0 - 2.0} = 0.77$$

TABLE 21

FIELD DATA COLLECTED FOR EVALUATING
OPERATING CONDITIONS AT A PRETIMED SIGNALIZED
INTERSECTION APPROACH

LOCATION: Southbound Texas Avenue at University
ESTIMATED SATURATION FLOW, S = 3400 VPHG
C = 75 sec., G = 18 sec., R = C - G = 57 sec.

| Time of Day | Time to Clear Queue (sec.) | Queue Cleared? | Average Queue Clearance, T |
|---|---|---|---|
| 5:00 p.m. | 14 | Yes | |
| | 13 | Yes | |
| | 7 | Yes | |
| | 15 | Yes | |
| | 17 | Yes | |
| | 9 | Yes | |
| | 15 | Yes | |
| | 15 | Yes | |
| | 21 | No | |
| | 19 | Yes | |
| | 21 | No | |
| | 14 | Yes | 15.0 sec. |
| 5:15 p.m. | | | |




METHOD FOR ESTIMATING DELAY AND PROBABILITY OF CLEARING QUEUES

FIGURE 16

Next, calculate the quantity: sG

$$sG = \frac{S \cdot G}{3600} = \frac{3400 \cdot 18}{3600} = 17.0$$

Draw a horizontal line across the upper central vertical scale at the calculated X (saturation) ratio value in Figure 16 (0.77). Extend this horizontal line to the right until the existing G/C ratio is reached. (The existing G/C ratio is 18/75 or 0.24.) Extend a line vertically downward and note the intercept value of $f_1$ (0.32). This value will be used later in the delay calculation.

Draw a vertical line across the left central horizontal scale, the sG scale, at the calculated value for sG (17.0). Extend this line first vertically until it intersects the previously drawn horizontal line for X. The intersection point is Miller's estimate for probability of clearing queues. Thus,

- Estimated $P_c$ = 0.85 (85%)

This compares with an observed value of 10/12 = 0.83. Field comparisons of the probability of clearing queues made from one day's study may not always compare closely to theoretical values due to the binary nature of queue clearances and the assumption made of Poisson arrival flow on the approach.

Next, extend the vertical line (sG = 17.0) downward in Figure 16 until the appropriate X (saturation) ratio curve is reached (0.77). Then extend a horizontal line from this point to intercept the vertical $f_2$ scale. Note the value of $f_2$ (0.10).

Calculate the value of delay as illustrated in Figure 16. The calculated value for the delay on the approach studied from 5:00 to 5:15 p.m. is:

$$d = (f_1 + f_2) \cdot C$$

$$d = (0.32 + 0.10) \cdot 75$$

$$d = 31.5 \text{ sec./ veh.}$$

Level of Service Summary

Comparing the calculated values for the X (saturation) ratio (0.77), probability of clearing queues (0.85) and delay (31.5 sec./veh.) with those level of service limits given in Table 20 indicates that traffic flow conditions were at Level of Service "C". Field observations would confirm these indications.

It is envisioned that this procedure will provide an efficient but consistent field evaluation procedure for pretimed signalized intersections.

Figure 16 was developed by combining Miller's probability of clearing queues equation (18) with Webster's simplified delay equation (17). Repeating Miller's probability of clearing queues equation

$$P_c = 1 - e^{-1.58\left(\frac{1 - X}{X}\right)\sqrt{s \cdot g}}$$

where:

$P_c$ = Decimal fraction of cycles which clear the queues on green.

s = Saturation flow rate of movement, veh./sec. green.

g = Movement effective green, sec.

X = Movement saturation ratio.

Webster's simplified equation for delay, d, (sec./veh.) is

$$d = 0.9 \left[ \frac{C(1 - \frac{g}{C})^2}{2(1 - X \cdot \frac{g}{C})} + \frac{X^2}{2q(1 - X)} \right] \qquad (18)$$

Since the saturation ratio, X, can be calculated from

$$X = \frac{q \cdot C}{g \cdot s} \qquad (19)$$

where q is the movement flow rate (veh./sec.) and C is the cycle length (sec.), then

$$q = \frac{X \cdot g \cdot s}{C} \tag{20}$$

Substituting for q in Webster's simplified delay equation (Eq. 18) results in

$$d = 0.9 \left[ \frac{C(1 - \frac{g}{C})^2}{2(1 - X \cdot \frac{g}{C})} + \frac{X \cdot C}{2g \cdot s(1 - X)} \right] \tag{21}$$

Rearranging terms slightly yields a delay equation of

$$d = C \left[ \frac{0.45(1 - \frac{g}{C})^2}{1 - X \cdot \frac{g}{C}} + \frac{0.45 \cdot X}{g \cdot s(1 - X)} \right] \tag{22}$$

or, as in Figure 16,

$$d = C \left[ f_1 + f_2 \right] \tag{23}$$

Note in Equation 22 that the independent variables g, C and s can be estimated or measured very easily. A more difficult variable to evaluate is the saturation ratio, X. This is also the case for Miller's probability of clearing queues equation which is also determined in Figure 16.

Figure 17 will be used to develop an equation used to estimate the saturation ratio, X, from queue clearance times manually recorded in the field. Referring to Figure 17, the number of vehicles queued on the approach at the beginning of the effective green is

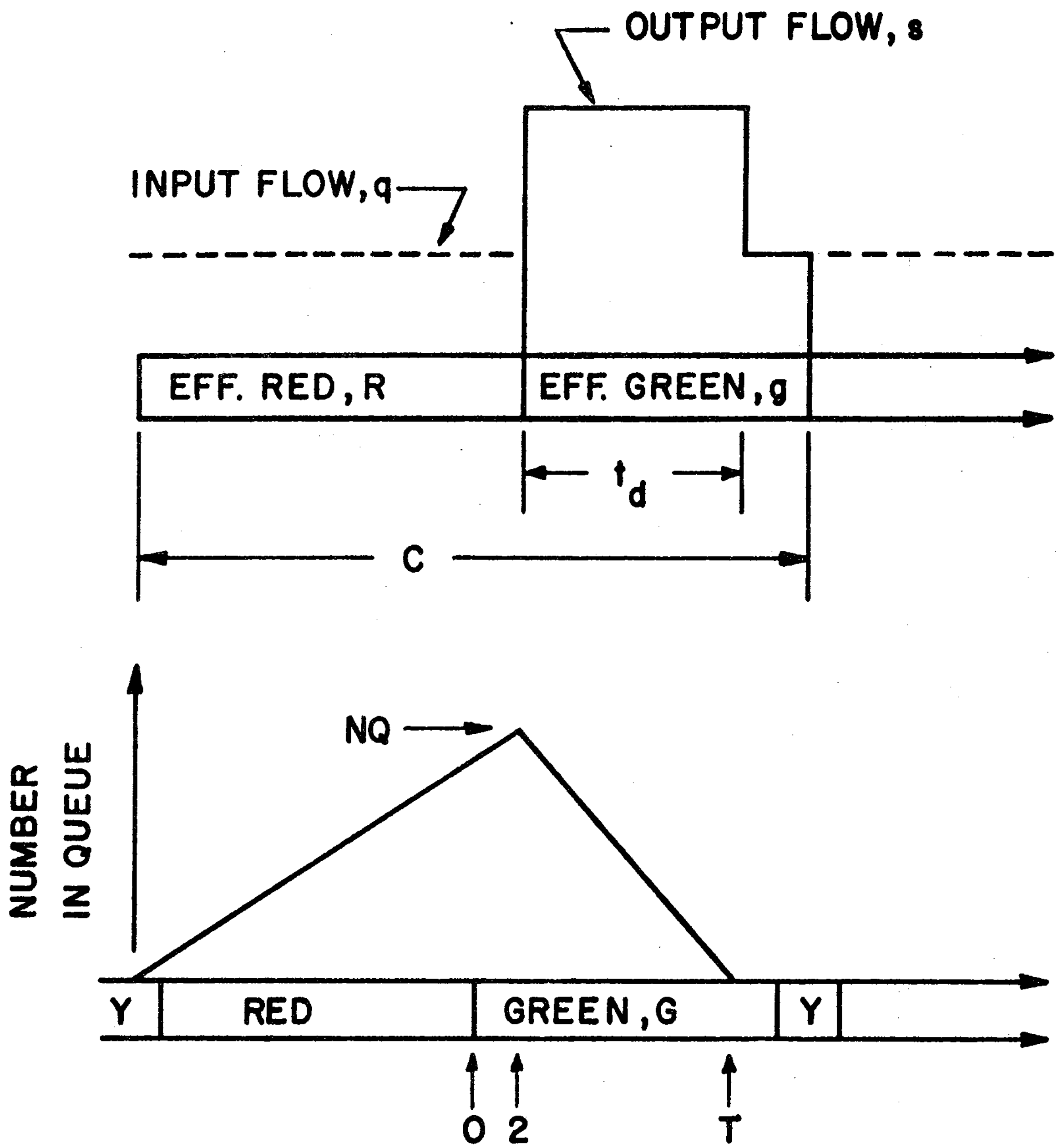
$$NQ = q \cdot R$$

where:

NQ = Number of vehicles in queue on the movement at the start of effective green, veh.

q = Average flow rate of the movement, veh./sec.

R = Effective red on movement, sec.




QUEUE AND SIGNAL CONDITIONS ASSUMED FOR FIELD EVALUATION

FIGURE 17

The average flow rate on the movement, $q$, can be estimated using the queueing input-output equation. Let $t_d$ be the time after the start of the effective green when the queue dissipates, or is reduced to zero. Thus, at time $t_d$,

$$0 = NQ + q \cdot t_d - s \cdot t_d$$

or

$$0 = q \cdot R + q \cdot t_d - s \cdot t_d$$

where s is the saturation flow rate (veh./sec.) on the movement. Solving for the average flow rate, q, yields

$$q = \frac{s \cdot t_d}{R + t_d}$$

The saturation ratio, X, can now be calculated as follows:

$$X = \frac{q \cdot C}{g \cdot s} = \frac{s \cdot t_d \cdot C}{(R + t_d) \cdot g \cdot s}$$

or

$$X = \frac{C \cdot t_d}{(R + t_d) \cdot g}$$

A slight adjustment is necessary to the previous equation to account for the way $t_d$ will be measured in the field. The variable $t_d$ is the elapsed time since the start of effective green when the queue dissipates. However, the elapsed time measured in the field will be from the start of the actual green. For practical purposes, it can be assumed that the queue "lost time" is 2.0 seconds (see Figure 17) and, therefore,

$$t_d = T - 2.0$$

where T is the average of the queue dissipation times measured from the start of the actual green. (See Table 21). Thus,

$$X = \frac{C \cdot (T - 2.0)}{(R + T - 2.0) \cdot g}$$

Since the actual green time, G, is approximately the same length as the effective green, g, the average saturation ratio can be estimated from

$$X = \frac{T - 2.0}{G} \cdot \frac{C}{R + T - 2.0} \tag{24}$$

which is Equation 17. The saturation ratio can then be used to calculate the probability of clearing queues and delay as shown in Figure 16.

## PASSER-II EXTENSIONS

Efforts were continued during this research project on improving the effectiveness of the Department's PASSER-II signal progression program which was developed and reported in an earlier research project (2). Some refinements have been made to the existing program reflecting user experiences with it. The revisions were internal improvements to the program and did not affect input or output formats. Several cities besides the State Department of Highways and Public Transportation (SDHPT) of Texas have successfully used PASSER-II on the remote computer terminals of SDHPT during the spring and summer of 1975.

A major task was undertaken to add an arterial signal system evaluation routine to the basic PASSER-II program. An economic approach was desired both from the research viewpoint and also in the operational usage of the program. Only straightforward, deterministic, non-iterative approaches were considered feasible and within the scope of the research effort.

The PASSER-II computer program (2) provided an initial set of outputs as shown in Figure 18. Sufficient outputs were provided to implement a complete signal timing plan along an arterial which would maximize arterial progression. The lower section in Figure 18 summarizes those outputs which have been added to PASSER-II as a result of this current research.

Primary research emphasis was directed toward characterizing the movement of progressive platoons along an arterial and toward developing a mathematical model for estimating delay on the arterial through movements where progression is provided. Webster's delay equation (17) was modified to estimate the delay on a movement where progression is provided, as described in subsequent sections.

**INPUT VARIABLES**

Movement Volumes

Movement Saturation Flows

Link Speeds

Allowable Signal Phasings

Minimum Greens

**INITIAL OUTPUTS**

Movement Green Times

Movement Saturation Ratios

Optimal Progression

Optimal Offsets

Optimal Phases

Time-Space Diagram

**ADDED OUTPUTS**

Movement Delay and
Probability of Clearing Queues

Level of Service Analysis

Intersection and Arterial Delay

INITIAL PASSER-II AND EXTENSIONS

FIGURE 18

## Development of Webster's Modified Delay Equation

Probably the most extensively used expression for delay at signalized intersections with Poisson arrivals is that developed by Webster (17):

$$d = \frac{C(1 - \frac{g}{C})^2}{2(1 - \frac{q}{s})} + \frac{X^2}{2q(1 - X)} - 0.65(\frac{C}{q^2})^{1/3} X^{2+5(\frac{g}{C})} \qquad (25)$$

where:

$d$ = Average delay, sec./veh.

$g$ = Length of effective green phase, sec.

$C$ = Cycle length, sec.

$q$ = Arrival rate, veh./sec.

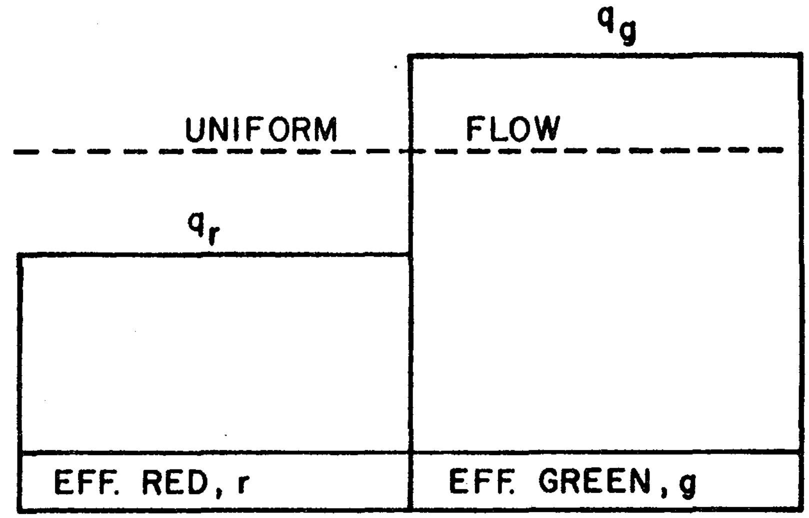
$s$ = Saturation flow, veh./sec.

$X$ = Saturation ratio, $q \cdot C/g \cdot s$.

Webster's first term is an expression for the average delay when arrivals are uniform; the other two terms estimate the added delay due to randomness. This extra delay results mainly from queue overflow from one cycle into the next during high volume conditions.

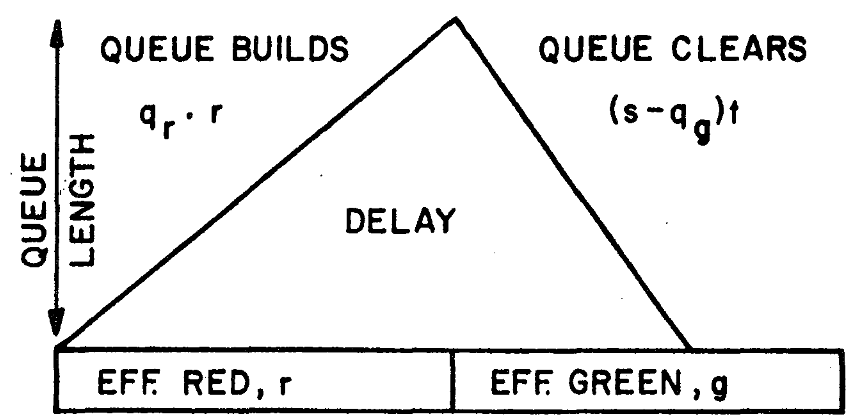
An interconnected signal system can result in non-uniform flow rates during individual cycles. If progression between signals is good, most of the traffic will arrive at the downstream intersection during the green phase of the signal. This results in the average arrival rate in the green portion of the cycle being greater than the average arrival rate during the red phase as shown in Figure 19-a. Poor progression could result in a greater arrival rate on the red than on the green.

To account for this phenomenon, the first term in Webster's delay equation was modified as described in subsequent sections. A list of previously defined terms used in the development includes:




ASSUMED EFFECT OF PROGRESSION ON TRAFFIC FLOW DURING THE CYCLE

FIGURE 19 - a.




TOTAL DELAY AT AN INTERSECTION

FIGURE 19 - b.

ASSUMED EFFECT OF PROGRESSION ON DELAY AT AN INTERSECTION

FIGURE 19

$NQ$ = Number in queue, veh.

$q_r$ = Arrival rate during the red, veh./sec.

$r$ = Length of the effective red phase, sec.

$q_g$ = Arrival rate during the green, veh./sec.

$g_o$ = Time at which NQ equals zero, sec.

$D$ = Total delay per cycle, veh.-sec.

In Figure 19-b, the number of vehicles in the queue at end of red is:

$$NQ = q_r \cdot r$$

Number in queue at time t during the green is:

$$r < t < g_o$$

$$NQ(t) = (q_r \cdot r) + (q_g \cdot t) - (s \cdot t)$$

The queue is emptied at time $g_o$. Therefore,

$$0 = (q_r \cdot r) + (q_g \cdot g_o) - (s \cdot g_o)$$

$$g_o = \frac{q_r \cdot r}{(s - q_g)} \tag{26}$$

The uniform component of delay is determined from the area of the number of vehicles in the queue given in Figure 19-b and is calculated in the following manner:

The total vehicle delay per cycle is:

$$D = (\tfrac{1}{2} r)\ (q_r \cdot r) + (\tfrac{1}{2} g_o)\ (q_r \cdot r)$$

Substituting Equation 26 and rearranging terms yields:

$$D = \frac{q_r \cdot r^2}{2} + \frac{q_r{}^2 \cdot r^2}{2(s - q_g)}$$

Factoring and rearranging terms gives:

$$D = \frac{q_r \cdot r^2}{2} \left[ \frac{s - q_g + q_r}{s - q_g} \right]$$

Thus, the average delay per cycle is given by:

$$d = \frac{D}{(q_r \cdot r + q_g \cdot g)}$$

or

$$d = \frac{q_r \cdot r^2}{2(q_r \cdot r + q_g \cdot g)} \left[ \frac{s - q_g + q_r}{s - q_g} \right]$$

Making this equation more consistent with the style of the first term in Webster's delay equation:

$$d = \frac{C(1 - \frac{g}{C})^2}{2} \left[ \frac{C \cdot q_r}{(q_r \cdot r + q_g \cdot g)} \right] \left[ \frac{s - q_g + q_r}{s - q_g} \right] \qquad (27)$$

To be able to use this equation, average arrival rates on both the green and red phases of the cycle must be known. In Webster's normal delay equation, the percent of the cycle that is green is the same as the percent of the volume that arrives on green; in other words:

$$\frac{PVG}{PTG} = 1$$

where:

PVG = Percent of the volume that arrives on the green.

PTG = Percent of the cycle that is green

Based on this, the flow during the green phase of the cycle can be calculated from:

$$q_g = \frac{PVG}{PTG} \cdot q$$

and the flow during the red phase is as follows:

$$q_r = \frac{PVR}{PTR} \cdot q = \frac{(1 - PVG)}{(1 - PTG)} \cdot q$$

When there is no progression (PVG is equal to PTG), Equation 27 will reduce to the first term in Webster's normal delay equation as shown below: Substituting for $q_r$ and $q_g$ in Equation 27:

$$d = \frac{C(1 - \frac{g}{C})^2}{2} \left[ \frac{C \cdot \frac{1 - PVG}{1 - PTG} \cdot q}{\frac{1 - PVG}{1 - PTG} \cdot q \cdot r + \frac{PVG}{PTG} \cdot q \cdot g} \right] \left[ \frac{s - \frac{PVG}{PTG} \cdot q + \frac{1 - PGV}{1 - PTG} \cdot q}{s - \frac{PVG}{PTG} \cdot q} \right]$$

Recalling that PVG/PTG = 1 and (1 - PVG)/(1 - PTG) = 1, it follows that

$$d = \frac{C(1 - \frac{g}{C})^2}{2} \left[ \frac{C \cdot q}{q \cdot r + q \cdot g} \right] \left[ \frac{s - q + q}{s - q} \right]$$

$$d = \frac{C(1 - \frac{g}{C})^2}{2 \frac{(s - q)}{s}}$$

$$d = \frac{C(1 - \frac{g}{C})^2}{2(1 - \frac{q}{s})}$$

as desired to illustrate in equation 25.

A periodic scan, computer simulation approach was chosen to study the effect of progression on delay at fixed-time traffic signals and to test the accuracy of the modified Webster's delay equation. Guidelines selected for this study were to develop a computer simulation program which would yield results within ten percent of the values predicted by Webster's normal formula. Traffic flow was simulated for one through lane approach to the intersection. Vehicles arrived at the intersection in a random (Poisson) manner and departed at a predetermined maximum rate. Individual vehicle delay on the approach was calculated as an operational measure of effectiveness.

In statement format, the simulation model can best be described in the following brief outline:

1. The simulation scans the system every second in the periodic scan mode, updating all program variables that should be changed.
2. Vehicles are assumed to arrive at the intersection in a random manner according to the Poisson distribution.
3. The average arrival rate during the green portion of the cycle may be different than the average arrival rate during the red phase.
4. More than one vehicle may arrive at the intersection during any scanning period.
5. Queued vehicles begin to leave the intersection two seconds after the start of the (effective) green phase at a headway of two seconds.
6. Vehicles which arrive at the intersection on the red phase of the cycle or before the queue has dissipated are delayed one second each scanning period until they clear the intersection.

The following variables are inputs to the intersection approach simulation program developed in this portion of the research:

1. Cycle length, sec.
2. Percent of the cycle that is green, PTG.
3. Percent of the total approach volume that arrives on the green phase of the signal, PVG.
4. Total approach volume, veh./hr.

The simulation program developed in this study calculates the following information:

1. Vehicle arrivals on green.
2. Vehicle arrivals on red.
3. Average vehicle delay, sec./veh.

To test the accuracy of the delays predicted by the computer simulation program, runs were made in which the average flow was the same during both the green and red phases of the cycle. In this way, simulated results could be compared directly to those values for delay predicted by Webster's normal equation. Figure 20 and Table 22 illustrate the results of this part of the study. Based on these results, it was concluded that the simulation program did indeed yield results within ten percent of those values for delay predicted by Webster's normal delay equation.

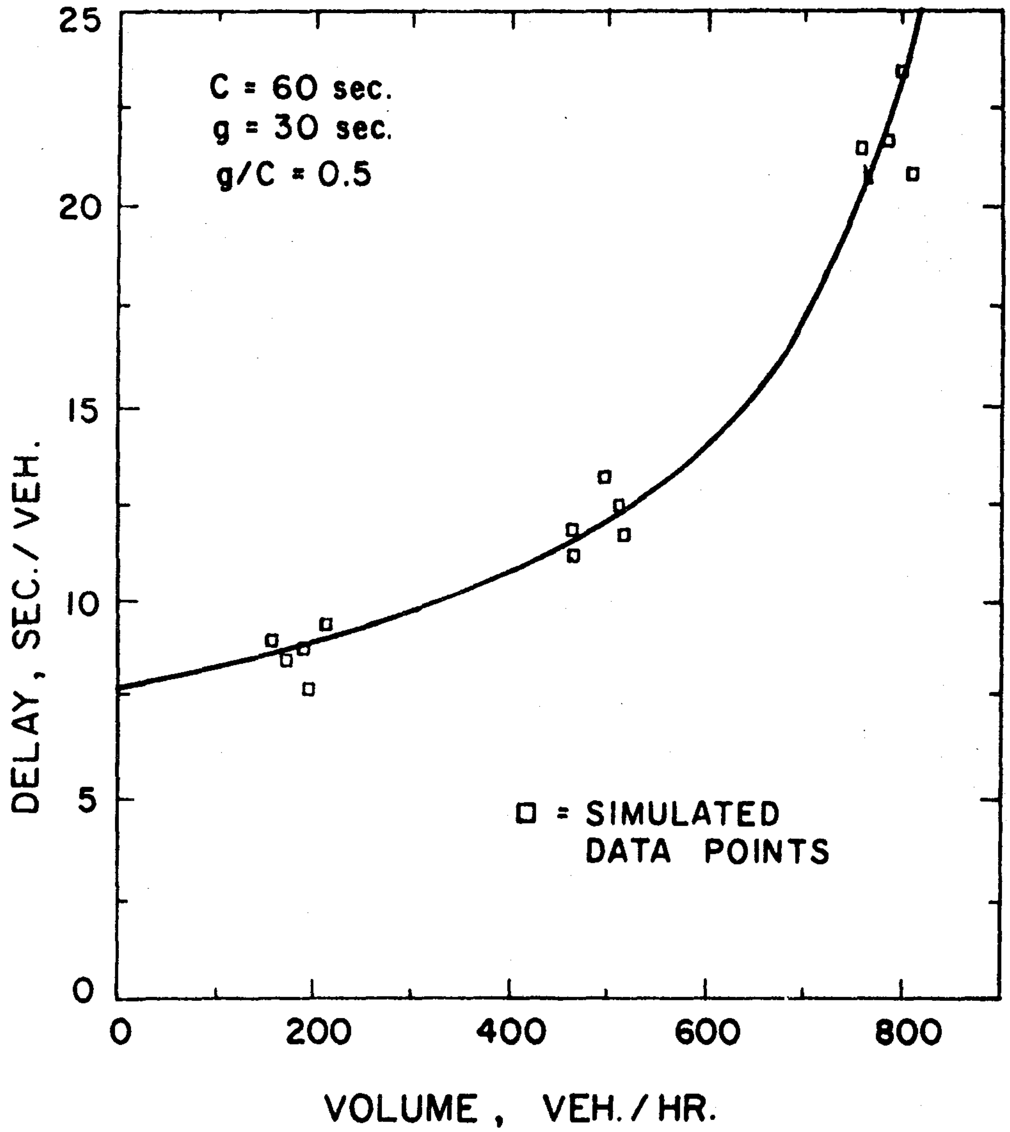
The expected effect of progression on delay at signalized intersections is to decrease the average delay per vehicle. Figure 21 illustrates this effect based on Webster's normal and modified equations. To verify these results, similation runs were made in which the percent of traffic arriving on the green phase was varied while the length of the green phase was held constant. The values of delay decreased as the percent traffic arriving on green increased as shown in Figure 22. In order to further verify Webster's modified equation, extra simulation runs were made in which each of the input variables was varied. Results of this study are illustrated in Table 23.

To use Webster's modified delay equation to estimate the effects of progression on delay, the ratio of the percent of the traffic which arrives on the green phase (PVG) to the percent of the cycle that is green (PTG) must be determined. This ratio, the progression interconnect (I), has the following characteristics and effects on delay:

- When more vehicles arrive on green than "normal", then

$$I = \frac{PVG}{PTG} > 1$$

and progression is effective and delays are less than those predicted by Webster's normal equation.




COMPARISON OF WEBSTER'S DELAY EQUATION TO RESULTS FROM COMPUTER SIMULATION PROGRAM

FIGURE 20

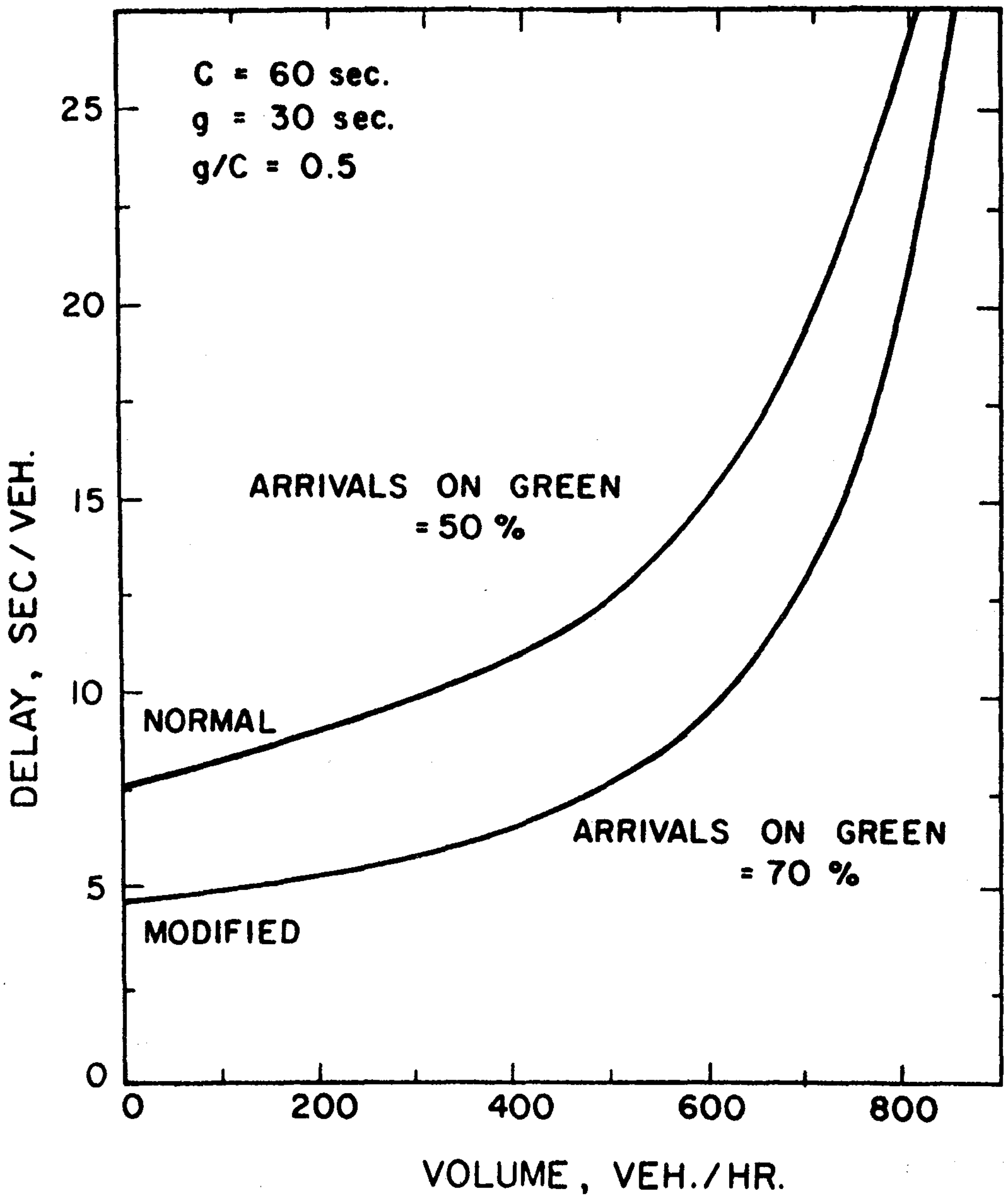
TABLE 22

DELAY AS PREDICTED BY COMPUTER SIMULATION

PROGRAM AND WEBSTER'S EQUATION

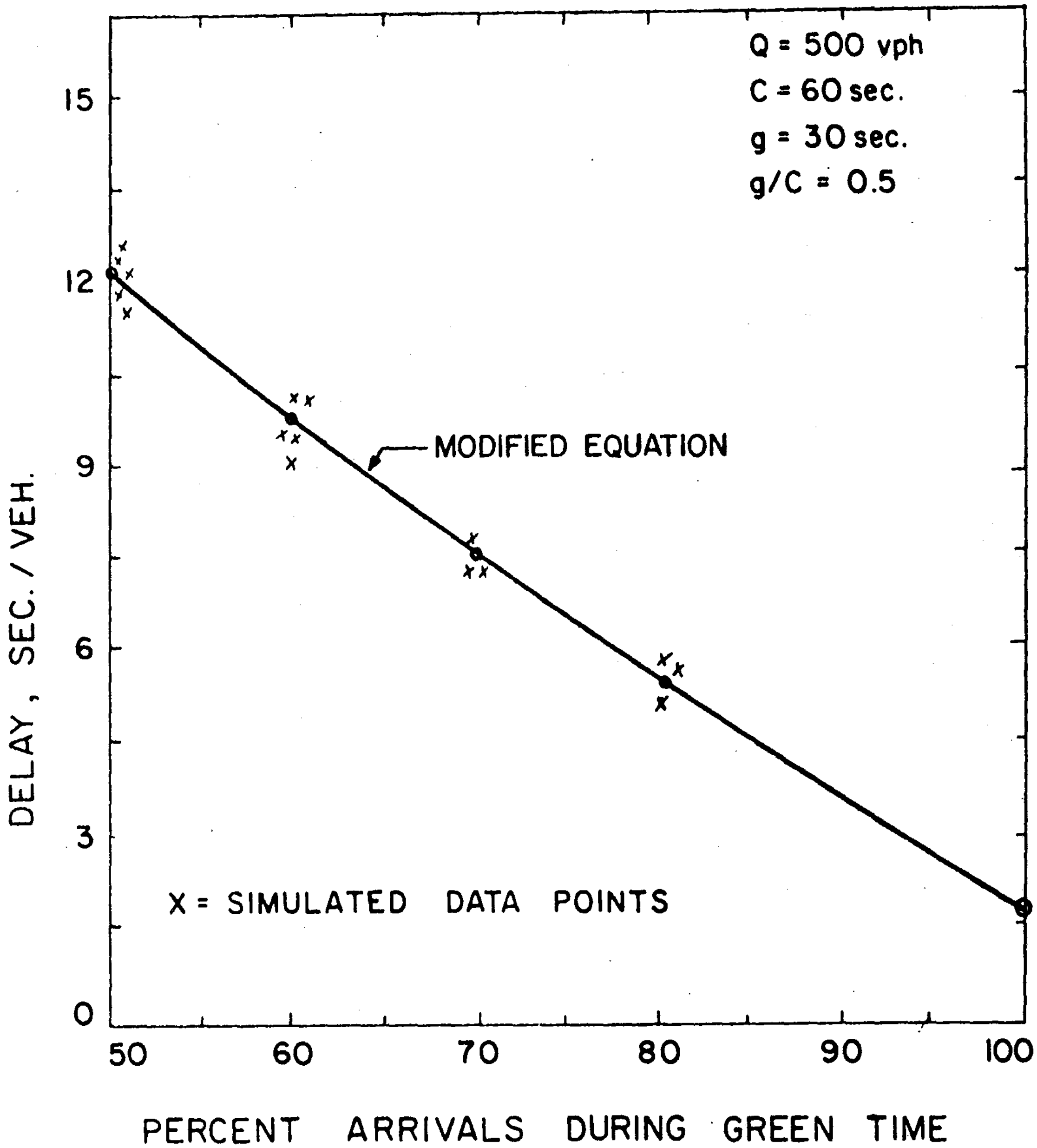
| Cycle Length | Percent Cycle Green | Percent Arrivals on Green | Simulated Arrivals* | Simulated Delay* sec./veh. | Webster's Normal Delay sec./veh. |
|---|---|---|---|---|---|
| 60 | 50 | 50 | 196 | 9.47 | 8.96 |
| 60 | 50 | 50 | 496 | 11.67 | 12.16 |
| 60 | 50 | 50 | 803 | 24.86 | 25.95 |
| 80 | 70 | 70 | 199 | 4.38 | 4.31 |
| 80 | 70 | 70 | 498 | 5.64 | 5.84 |
| 80 | 70 | 70 | 812 | 7.88 | 8.47 |

*Average of ten hours of simulated data.




**COMPARISON OF WEBSTER'S NORMAL AND MODIFIED DELAY EQUATIONS**

FIGURE 21




EFFECTS OF PROGRESSION ON INDIVIDUAL VEHICLE DELAY

FIGURE 22.

TABLE 23

DELAY AS PREDICTED BY COMPUTER SIMULATION

AND WEBSTER'S MODIFIED EQUATION

| Cycle Length | Percent Cycle Green | Percent Arrivals on Green | Simulated Arrivals* | Simulated Delay* sec./veh. | Webster's Modified Delay sec./veh. |
|---|---|---|---|---|---|
| 60 | 50 | 60 | 507 | 9.26 | 9.91 |
| 60 | 50 | 70 | 194 | 6.04 | 5.37 |
| 60 | 50 | 70 | 500 | 7.35 | 7.55 |
| 60 | 50 | 70 | 487 | 7.72 | 7.42 |
| 60 | 50 | 70 | 793 | 16.96 | 18.47 |
| 60 | 50 | 80 | 467 | 5.06 | 5.17 |
| 80 | 30 | 70 | 478 | 28.87 | 29.58 |
| 80 | 50 | 70 | 507 | 9.38 | 9.47 |
| 80 | 50 | 70 | 491 | 8.74 | 9.29 |
| 90 | 40 | 50 | 492 | 21.17 | 20.94 |
| 90 | 40 | 60 | 488 | 17.01 | 16.68 |
| 90 | 40 | 70 | 498 | 13.38 | 13.90 |
| 90 | 60 | 70 | 200 | 6.27 | 6.29 |
| 90 | 60 | 70 | 494 | 7.95 | 8.22 |
| 90 | 60 | 70 | 791 | 11.45 | 11.92 |

*Average of ten hours of simulated data.

- When vehicle arrivals are "normal", then

$$I = \frac{PVG}{PTG} = 1$$

and there is no progression between signals. Vehicle delays are the same as those predicted by Webster's normal equation.

- When fewer vehicles arrive on green than "normal", then

$$I = \frac{PVG}{PTG} < 1$$

and progression is bad and delays are greater than those predicted by Webster's normal equation.

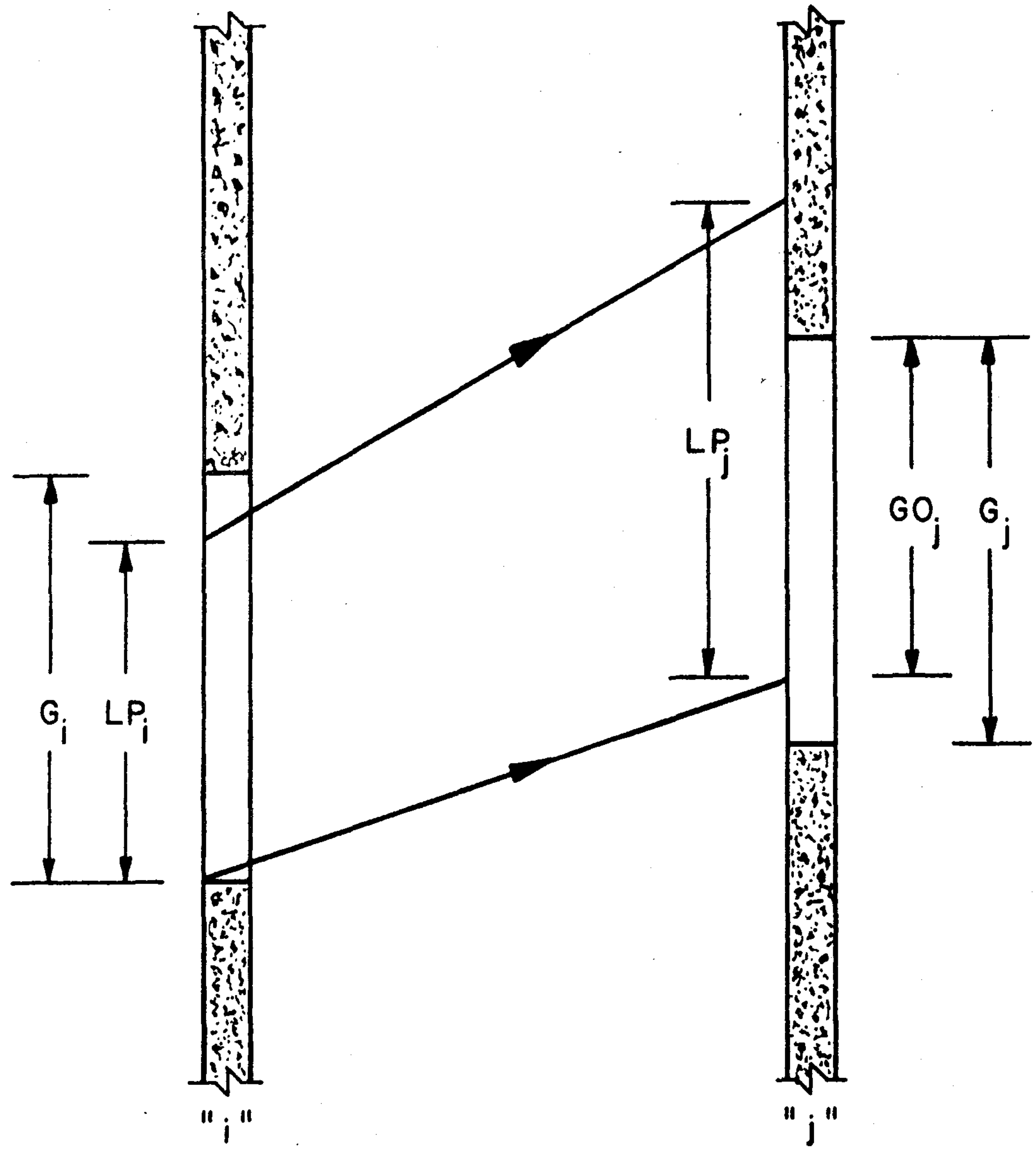
The following section of this report describes how the progression interconnect I is calculated in PASSER-II for use in Webster's modified delay equation. Primary interest will be focused on estimating the percent of the approach's through volume arriving on the through green (PVG) since the percent of the cycle that is green (PTG) can be calculated easily.

<u>Percent Volume Progressed</u>, $PVG_j$

The percent of an approach's through traffic which has come from the through traffic movement of an adjacent upstream intersection and which arrives during the through green at the intersection ("j"), $PVG_j$, depends on several factors. Three principal factors considered in this model are: 1) the percent of the total approach traffic which is in the progression platoon, 2) the size and rate of platoon dispersion, and 3) the quality of platoon progression between the two intersections. Figure 23 summarizes the progression interconnect model.

The percent volume progressed is calculated from

$$PVG_j = PTT_j \cdot \frac{GO_j}{LP_j} \qquad (28)$$




MODEL OF PROGRESSION PLATOON MOVEMENT FROM INTERSECTION I TO J

FIGURE 23

where:

$PVG_j$ = Percent of the approach's total through traffic at intersection "j" arriving during green, $G_j$, due to progression.

$PTT_j$ = Percent of the approach's total through traffic at intersection "j" arriving in the platoon of length $LP_j$.

$LP_j$ = Estimated platoon length when it arrives at intersection "j" from intersection "i", sec.

$GO_j$ = Length of time platoon arrivals at intersection "j" overlap the approach's through green time, $G_j$, sec. (See Figure 23.)

The percent of the total through traffic on the approach being considered at intersection "j" that is assumed to be in the progression platoon, $PTT_j$, is calculated from

$$PTT_j = \frac{0.9 \cdot Q_{i4A}}{\left[0.9 \cdot Q_{i4A} + Q_{i3B} + 0.1 \cdot Q_{i4B}\right] \geq \left[Q_{j1A} + Q_{j4A}\right]} \qquad (29)$$

where the volume $Q_{i4A}$ is movement 4 turning volume (See Figure 24.) at intersection "i", etc. A ten percent right turning volume was assumed at intersection "i". This is not a critical assumption since a downstream check of total approach volumes is made at "j".

The length of the platoon, when it arrives at intersection "j" from intersection "i", is calculated from

$$LP_j = LP_i \cdot PD_{ij} + 0.8 \cdot J_t \qquad (30)$$

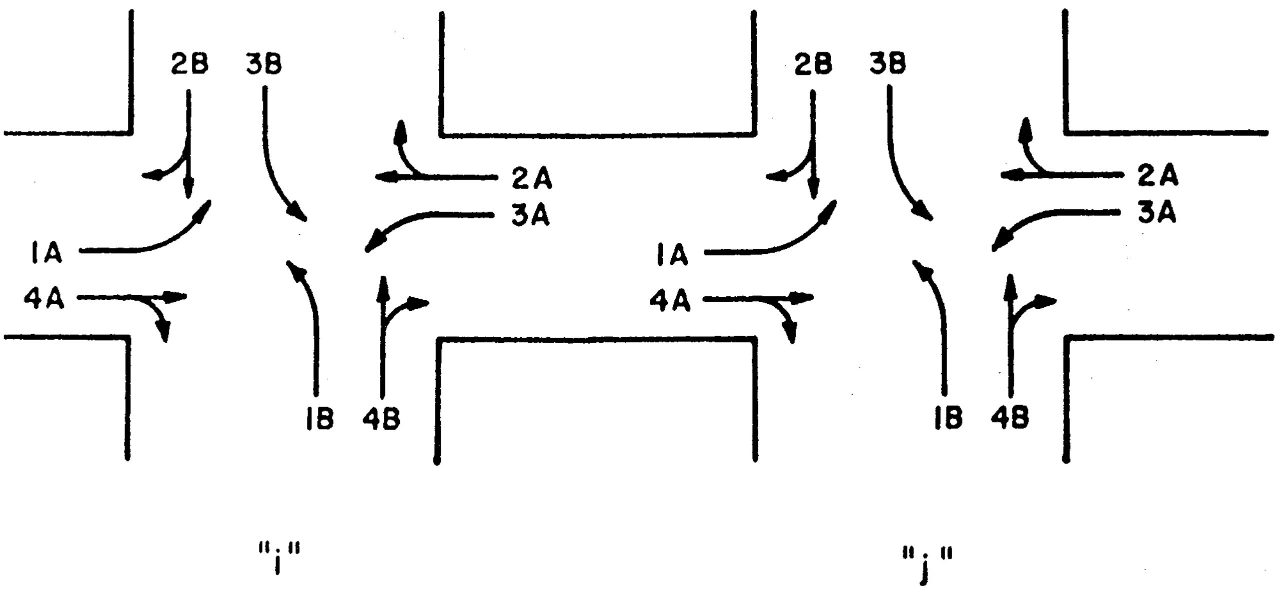
where:

$LP_j$ = Length of platoon arriving at intersection "j", sec.

$LP_i$ = Length of platoon when leaving "i", sec. (Equation 31.)

$PD_{ij}$ = Platoon dispersion factor from "i" to "j". (Equation 33.)

$J_t$ = Standard deviation of arrival times at "j". (Equation 35.)




DEFINITION OF TRAFFIC MOVEMENTS
AT INTERSECTIONS I AND J

FIGURE 24

The length of the progression platoon initially leaving intersection "i" bound for intersection "j" is calculated from a rather complicated formula

$$LP_i = G_o \left(PVR + PVG \cdot \frac{G_o}{G}\right) + PVG \; (G - G_o) \qquad (31)$$

where:

$LP_i$ = Length of platoon leaving intersection "i", sec.

PVR = Percent of approach through volume at "i" arriving at "i" on red.

PVG = Percent of approach through volume at "i" arriving at "i" on green.

G = Green time for the through movement at "i", sec.

$G_o$ = Time saturation flow ceases on through movement at "i", sec.

where:

$$G_o = \frac{PVR \cdot X}{(1 - PVG \cdot X)} \cdot G \leq G \qquad (32)$$

where X is the average saturation (volume-to-capacity) ratio for the through movement, $X = QC \div GS$.

The equation for $LP_i$ is somewhat complex. Boundary condition values for $LP_i$ will be presented to illustrate the equation's output. The boundary condition results at intersection "i" are:

1. When there is no through volume, $X = 0$, $G_o = 0$, and $LP_i = PVG \cdot G$,
2. When the approach is fully loaded to capacity, $X = 1$, $G_o = G$, and $LP_i = G$,
3. When PVG = 0, (bad progression), $G_o = X \cdot G$, and $LP_i = X \cdot G$,
4. When PVG = 1.0 (perfect progression), $G_o = G$, and $LP_i = G$.

The latter two conditions illustrate an important point which the model describes. That is, bad progression (low PVG) at intersection "i" makes it easier to provide good progression downstream for the next intersection. On the other hand, good progression (high PVG) at intersection "i" makes it more difficult to provide progression to the next downstream intersection.

Platoon Dispersion, $PD_{ij}$

The platoon dispersion factor, $PD_{ij}$, which gives the dispersion of a platoon as it travels between intersection "i" and "j", was developed from field studies conducted during 1974-75. After some initial pilot testing in Dallas and Houston, extensive platoon dispersion studies were conducted in College Station. A total of 349 platoons were measured along Texas Avenue and 55 on University Drive. Speed limits were 45 MPH and 40 MPH, respectively. The earlier pilot studies indicated that a high vantage point was necessary to manually collect accurate field data. As a consequence, all platoon dispersion data in College Station were collected from the 15th story of the Oceanography-Meteorology Building at Texas A&M University.

Data were collected by timing platooned vehicles between pre-measured checkpoints on the street as they departed from a traffic signal. This was accomplished by assigning an observer to each checkpoint and as each of the vehicles passed his checkpoint the observer would activate one of the pens on a 20-pen, Esterline-Angus event recorder resulting in a series of "blips" for the platoon at each checkpoint. The first checkpoint was located about a hundred feet from the stop line to allow the platoon to begin moving before it was measured.

The number of vehicles in each platoon was also recorded. Platoon data were divided into very light, light, moderate, and heavy platoons with less than 4, 4 - 8, 9 - 16, and greater than 16 vehicles, respectively. Facilities were chosen for the study which had few points of entrance or egress within the study section. Those few vehicles, however, which did enter or leave the platoon were noted on the paper chart and later discarded during field data reduction.

In order to reduce some of the arrival time variation caused by the leading and lagging vehicles in larger platoons the first and last vehicles in the 4 - 8 and 9 - 16 vehicle platoons, and the first two and last two vehicles in the greater than 16 platoons were discarded from the arrival time samples.

As mentioned previously, vehicles were timed as they moved through the study section. The times were recorded for each vehicle in the platoon as it passed the various checkpoints. From these data platoon arrival times and platoon dispersion factors were calculated as follows:

Platoon Arrival Time: The amount of time needed for the first measured vehicle to travel between the initial checkpoint "A" and one of the successive checkpoints;

and,

Platoon Dispersion: The length of the platoon in time (sec.) at some point compared to its initial length. This value was found by dividing the time between the first and last measured vehicle at a point by the length of time between the first and last measured vehicle at the initial point.

Results of the platoon movement and dispersion studies conducted along Texas Avenue in College Station are presented in Tables 24 and 25. Average arrival times of platoons did not vary much with platoon size and neither did platoon speeds. Average mid-block (B to C) speeds ranged only from 38 to 40 miles per hour. Platoons were observed to disperse slightly faster on the average as the number of vehicles in the platoon became smaller, as presented in Table 25.

A multiple linear regression analysis was run on the data to develop an equation for platoon dispersion. The following equation resulted:

TABLE 24

AVERAGE PLATOON TRAVEL TIMES

ON TEXAS AVENUE IN COLLEGE STATION

| Date | Platoon Size | Sample Size | Platoon Arrival Time at Pt. B+ | C* | Dx |
|---|---|---|---|---|---|
| Summer, 1974 | L | 40 | 10.35 | 20.91 | 28.60 |
| | M | 92 | 10.52 | 21.49 | 29.63 |
| | H | 19 | 9.98 | 20.97 | 28.95 |
| | Avg. | | 10.41 | 21.27 | 29.29 |
| February 14, 1975 | VL | 7 | 10.26 | 20.29 | 29.00 |
| | L | 57 | 10.14 | 20.43 | 28.96 |
| | M | 36 | 10.50 | 21.00 | 30.27 |
| | Avg. | | 10.27 | 20.57 | 29.36 |
| April 3, 1975 | VL | 19 | 10.14 | 20.21 | 28.29 |
| | L | 49 | 10.28 | 20.33 | 28.43 |
| | M | 30 | 10.01 | 20.57 | 29.28 |
| | Avg. | | 10.17 | 20.41 | 28.71 |

+Distance from point A to point B is 590 feet.

*Distance from point A to point D is 1190 feet.

xDistance from point A to point D is 1640 feet.

TABLE 25

PLATOON DISPERSION DATA

ON TEXAS AVENUE IN COLLEGE STATION

| Date | Platoon Size | Average Dispersion at Pt. B+ | C* | Dx |
|---|---|---|---|---|
| Summer, 1974 | L | 1.08 | 1.28 | 1.40 |
| | M | 1.05 | 1.21 | 1.31 |
| | H | 1.06 | 1.14 | 1.25 |
| | Avg. | 1.05 | 1.20 | 1.30 |
| February 14, 1975 | VL | 1.14 | 1.50 | 1.96 |
| | L | 1.06 | 1.31 | 1.61 |
| | M | 1.06 | 1.23 | 1.38 |
| | Avg. | 1.08 | 1.30 | 1.55 |
| April 3, 1975 | VL | 0.95 | 1.33 | 1.97 |
| | L | 1.03 | 1.19 | 1.39 |
| | M | 1.11 | 1.28 | 1.43 |
| | Avg. | 1.06 | 1.23 | 1.41 |

+Distance from point A to point B is 590 feet.

*Distance from point A to point C is 1190 feet.

xDistance from point A to point D is 1640 feet.

$$PD_{ij} = 1.0 + (0.026 - 0.0014 \cdot NP)\ t_{ij} \tag{33}$$

where:

$PD_{ij}$ = Platoon dispersion factor, $LP_i \div LP_j$; $\geq 1.1$

NP = Number of vehicles in platoon, veh.

$t_{ij}$ = Running travel time from intersection "i" to intersection "j", sec.

Equation 33 is illustrated in Figure 25. Dispersion increases with increasing travel time and smaller platoons. The new PASSER-II program calculates all variables necessary to calculate platoon dispersion, including the number of vehicles in platoon from

$$NP = q_r \cdot R + q_g \cdot G_o \tag{34}$$

where:

NP = Number of vehicles in platoon, veh.

$q_r$ = Total through approach volume on red, veh./sec.

R = Red time on through movement, sec.

$q_g$ = Total through approach volume on green, veh./sec.

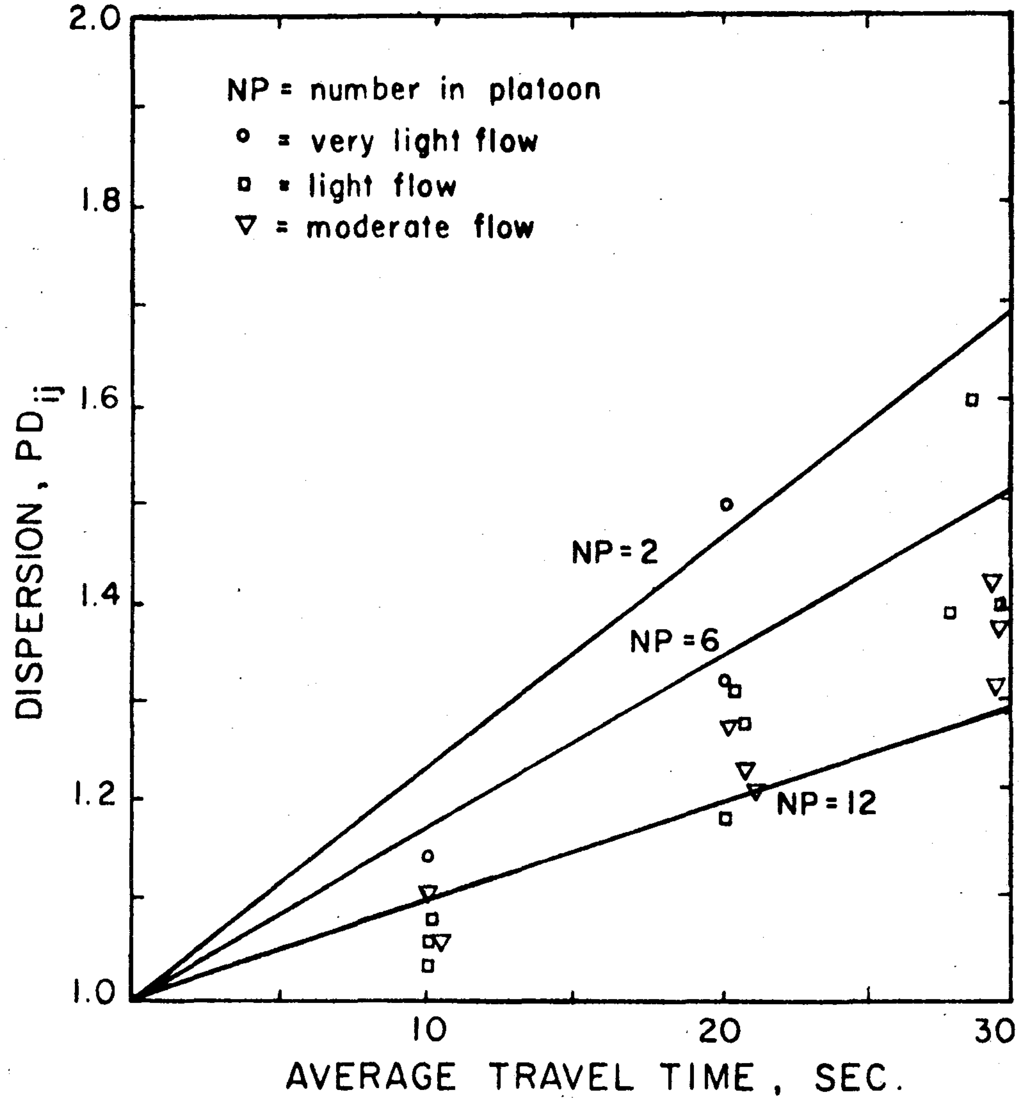
$G_o$ = Queue clearance time from Equation 32, sec.

The British developed a platoon dispersion factor which also relates platoon dispersion to travel time (19). Expressing their platoon dispersion equation in similar terms yields

$$PD_{ij} = 1.0 + 0.00667 \cdot t_{ij}$$

The two platoon dispersion equations give the same platoon dispersion factors for the same travel times, $t_{ij}$; when the number of vehicles in platoon, NP, equals 13.8 in the former equation.

The field studies also indicated that platoons do not arrive at the same location at the same time, even when progression is provided. The standard




**DISPERSION VERSUS TRAVEL TIME FOR VARIOUS PLATOON SIZES**

FIGURE 25

deviation of platoon arrival times presented in Figure 26 illustrates this fact. The longer platoons must travel between signalized intersections, the more dispersed their arrival times become. This has the effect over many cycles of dispersing platoon arrivals on the average. This platoon dispersion effect was given by 0.8 $J_t$ in Equation 30 where $J_t$ is equal to

$$J_t = 0.90 + 0.056 \cdot t_{ij} \tag{35}$$

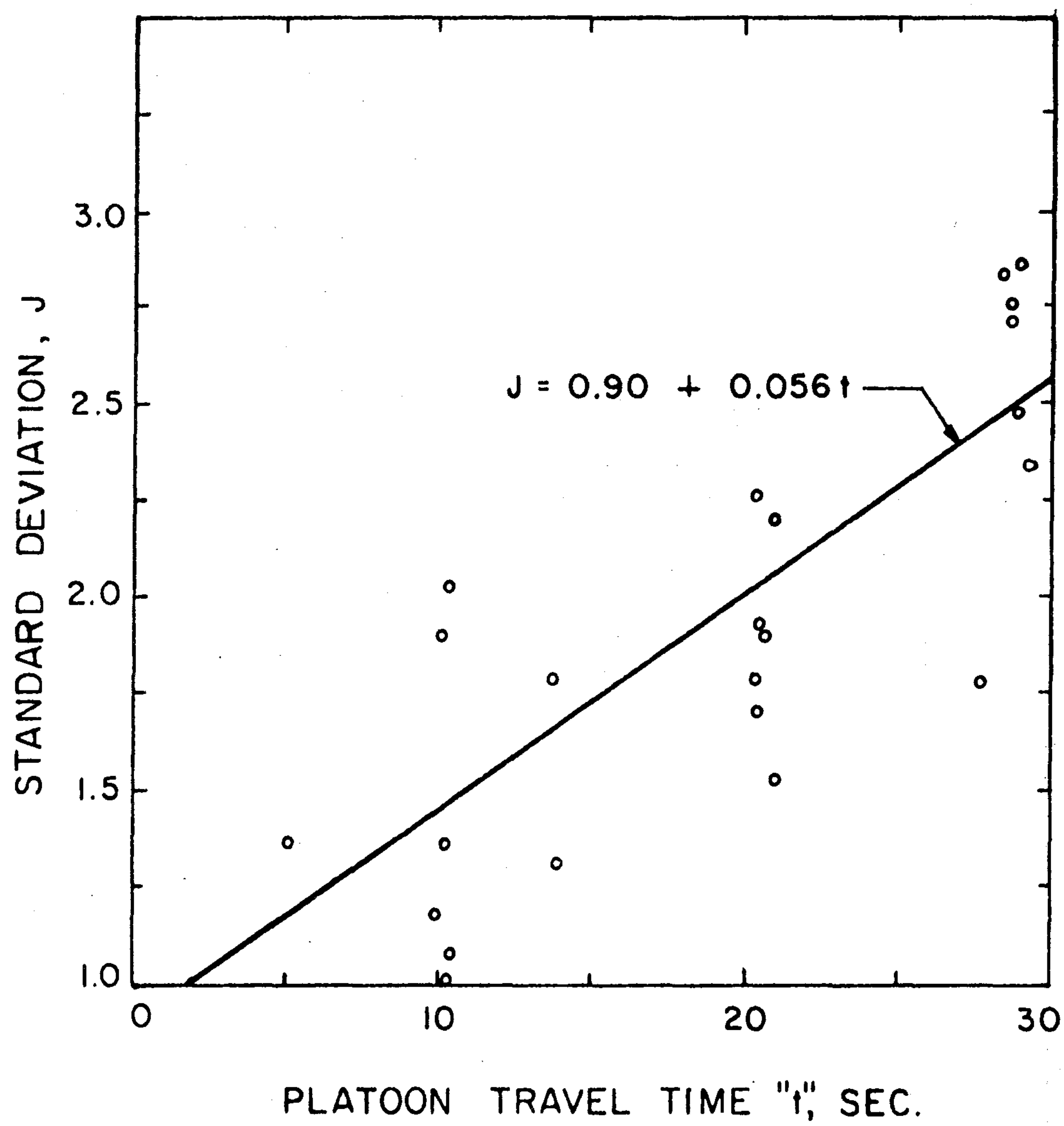
where $t_{ij}$ is the running travel time from intersection "i" to "j". The factor 0.8 in Equation 30 corrects the standard deviation of arrival times to mean deviations (20).

Green Overlap, $GO_j$

The amount of time the progressed platoon overlaps the through green at intersection "j" is denoted by $GO_j$. As illustrated in Figure 23, this amount of time depends on the length of the platoon at intersection "j", $LP_j$, the length of the through green at "j", $G_j$, and also on the quality of progression between intersections "i" and "j". The optimal arterial progression time-space diagram calculated by PASSER-II (2) is used to determine the quality of progression between the intersections. Good progression would result in a larger green overlap, $GO_j$, while bad progression might result in little or no overlap.

Summary of Progression Interconnect

The results of these previous studies and progression model developments describe indirectly some of the general characteristics that would indicate when interconnecting an arterial signal system to provide progression might be expected to reduce arterial delay. The characteristics are:




VARIATION IN PLATOON TRAVEL TIMES BETWEEN INTERSECTIONS

FIGURE 26

- Intersection spacings of about one-fourth or one-half mile.
- Moderate traffic volumes and running speeds.
- Low intersection turning volumes.
- Low mid-block turning volumes.
- Consistently large, arterial through greens.

## ACKNOWLEDGMENTS

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# APPENDIX A

## EXAMPLE CALCULATION

<u>Given</u>: Two lane approach with adequate length left turn bay.

Two-phase signal timing.

Two lanes of opposing flow.

$Q_T$ = 600 vph

C = 70 sec.

G = 28 sec.

A = 3 sec.

R = 39 sec.

<u>Assume</u>:

$L_1 + L_2$ = 4 sec.

$S_T$ = 1750 vph

$T_c$ = 4.5 sec.

H = 2.5 sec.

<u>Determine</u>:

Left turn capacity of approach in vehicles per hour.

<u>Solution</u>:

Determine lane distribution for two lane approach:

$$P = 0.55 + 0.45e^{(-0.18 \times m)}$$

and

$$m = \frac{(Q_T) \times (C)}{3600} = \frac{600 \times 70}{3600} = 11.7$$

$$P = 0.55 + 0.45e^{(-0.18 \times 11.7)} = .605$$

Calculate time for queue to clear:

$$T_Q = \frac{P \times Q_T \times (R + L_1 + L_2)}{S_T - (P \times Q_T)}$$

$$T_Q = \frac{.605 \times 600 \times (39 + 4)}{1750 - (.605 \times 600)}$$

$$T_Q = 11.25 \text{ seconds}$$

Determine the total time available for left turns:

$$T_A = G + A - (L_1 + L_2) - T_Q$$

$$T_A = 28 + 3 - 4 - 11.25$$

$$T_A = 15.75 \text{ seconds}$$

Calculate the left turn capacity of the approach across free flow, random traffic:

$$Q_{LH} = Q_T \times \frac{e^{-q_T T_c}}{1-e^{-q_T H}}$$

$$Q_{LH} = 600 \times \frac{e^{-(600/3600) \times 4.5}}{1-e^{-(600/3600) \times 2.5}}$$

$$Q_{LH} = 832 \text{ vph}$$

Determine left turn capacity of approach per hour of signal operation:

$$Q_L = \frac{Q_{LH} \times T_A}{C}$$

$$Q_L = \frac{832 \times 15.75}{70}$$

$$Q_L = 187 \text{ vph}$$

Compare to minimum left turn capacity (1.6 left turns per cycle):

$$Q_{L_{min}} = \frac{1.6 \times 3600}{C}$$

$$Q_{L_{min}} = \frac{1.6 \times 3600}{70}$$

$$Q_{L_{min}} = 82 < 187$$

Left turn capacity,

$$Q_L = \underline{187 \text{ vph}}$$