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16. Abstract

Urban corridors, as they exist today, often include a freeway with a supporting network of arterial, collector, and local streets. In some cases, a freeway may not be present, leaving arterial streets as the main carriers of long distance trips. The importance of the arterial street is also shown by the fact that large urban freeway building programs are no longer being carried on in many of the major U.S. cities, thus placing an even greater load on existing and future arterials.

Experience indicates that the definition of, and boundaries for, quality of service and capacity for freeways as set forth in the 1965 Highway Capacity Manual are deemed adequate for the highway design engineer's use. There appears, however, to be less confidence and uniformity evident in the use of similar criteria for arterial streets. For this reason a portion of this research study dealt with the capacity and quality of service on arterial streets.

The research study reported herein recognized the importance of the signalized intersection as an element in determining overall corridor level of service, and to this end a part of the research was set aside to review the state-of-the-art and recommend methods of determining levels of service at signalized intersections. The study examined a number of aspects of intersection capacity and provided determinations.

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SUMMARY

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The research study reported herein recognized the importance of the signalized intersection as an element in determining overall corridor level of service, and to this end a part of the research was set aside to review the state-of-the-art and recommend methods of determining levels of service at signalized intersections. The study examined a number of aspects of intersection capacity and its principal findings were as follows:

a. A saturation flow approach somewhat similar to the Australian, British, or critical lane analysis technique is recommended.

b. A study of saturation flows conducted at 16 intersections in Austin, College Station, and Houston found no significant difference in saturation flows for lanes 10, 11, and 12 feet wide. Preliminary

saturation flow data shown in Table A are recommended for suburban arterial streets.

TABLE A. SATURATION FLOW DATA

Lane Width	9'	10' to 12'
Lane Type	Saturation Flows	Through Car Units/hr.
Through & Through Right	1600	1750
Through Left		1550
Left		1700

c. To convert vehicle counts to through car units (T.C.U.'s), the conversion factors shown in Table B are recommended.

TABLE B. THROUGH CAR UNIT (T.C.U.) FACTORS

One truck or bus	=	2.0 passenger cars (p.c.)
One left turn (p.c.)	=	3.0 T.C.U.'s
One right turn (p.c.)	=	1.25 T.C.U.'s
One through (p.c.)	=	1.0 T.C.U.'s

d. Except for the cases when intersections were operating under pressure, the duration of saturation flows varied across the traffic lanes. Consequently, the saturation flows across all lanes should be taken as 90% of the summation of anticipated flow rates for each lane in computing delays and probabilities of clearing queues.

e. Delays should be held to a reasonable minimum by the selection of cycle lengths within the range of 85-125 percent of those calculated using Webster's method. Phase lengths should preferably be apportioned using Webster's method. Frequently signal system considerations and the need to satisfy minimum pedestrian phase lengths will result in some deviation for these criteria. Delays should be estimated for each approach and for the intersection as a whole using Webster's method.

f. The probability of clearing queues and Volume/capacity ratio are recommended as descriptors of level of service for operations and design, respectively. The recommended boundary values for the various levels of service are given in Table C.

g. The sum of the ratios of demand volume (in T.C.U.'s) to saturation flows for conflicting phases which has been termed the "Y" value provides a useful general descriptor for rapid evaluation of alternative designs. The following approximate limits of Y values are suggested:

• Two-phase operation	$Y < 0.70$
• Three-phase operation	$Y < 0.66$
• Four-phase operation	$Y < 0.63$
• Diamond interchange, four-phase overlap, with total overlap less than 16 seconds	$Y < 0.75$
• Diamond interchange, four-phase overlap, with total overlap equal to 16 seconds	$Y < 0.80$
• Diamond interchange, four-phase overlap, with total overlap greater than lost time	$Y < 0.85$

TABLE C

RECOMMENDED LEVELS OF SERVICE
FOR OPERATIONS AND DESIGN CONSIDERATIONS

LEVEL OF SERVICE	FLOW DESCRIPTION	RECOMMENDED VALUES PROBABILITY OF CLEARING QUEUES	RECOMMENDED VALUES VOLUME/CAPACITY RATIO
A	Free Flow	$P_0 > 0.95$	≤ 0.60
B	Satisfactory Operation. Vehicles Wait For Second cycle < 15% of time	$0.90 < P_0 \leq 0.95$	≤ 0.70
C	Satisfactory Operation. Defines the lower limit of satisfactory Operation	$0.75 < P_0 \leq 0.90$	$\leq .80$
D	Potential Instability, Unsatisfactory Operation; Vehicles frequently waiting two or more cycles	$0.50 < P_0 \leq 0.75$	$\leq .90$
E	Unstable Flow; Unsatisfactory Operations; extensive queues formed	$P_0 \leq 0.50$	≤ 1.0

h. The following framework is suggested for determination of levels of service at signalized intersection:

1. Prepare a sketch plan of the approach under study.
2. Record the signal phasing and phase lengths (where these are not known it is suggested that trial cycle lengths and phase lengths be determined using Webster's (5) method.
3. Obtain design volumes including percentage of trucks. If these are hourly volumes, convert them to equivalent peak quarter hourly volumes by division by the appropriate peak hour factor.
4. Convert volumes to equivalent through car equivalents using the following factors:

1 Bus or truck	=	2 cars
1 Right turn vehicle	=	1.25 through car equivalents
1 Left turn vehicle (Separate turn lane and phase)	=	1 through car equivalent
1 Left turn vehicle	=	E_{LT} (Calculated using Miller' expression)

Where the signal phasing is not known, E_{LT} may be approximated as 3 for the purpose of initial evaluations.

5. Calculate y values for all approaches and phases. Sum the maximum y values for each phase to obtain Y and check against suggested limits.
6. If cycle length is not fixed, calculate Webster's optimum cycle length and phase length. Adjust them to satisfy minimum requirements for phase lengths.
7. Calculate average delay on the approach (Note: For this calculation, the volume q should be in vehicles per second.).

8. Calculate the probability of clearing queues and compare to values in Table C.

IMPLEMENTATION

The results of this research should supplement current methods of analyzing the capacity and quality of service at intersections. The saturation flow model, as suggested in this report, promises to be a more rational approach to calculating intersection capacity. Further calibration of the saturation flow model will, however, be needed before it is fully functional.

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INTRODUCTION

Urban corridors, as they exist today, often include a freeway with a supporting network of arterial, collector, and local streets. In some cases, a freeway may not be present leaving arterial streets as the main carriers of long distance trips. The importance of the arterial street is also shown by the fact that large urban freeway building programs are no longer being carried on in many of the major U.S. cities, thus placing an even greater load on existing and future arterials.

Experience indicates that the definition and boundaries for quality of service and capacity for freeways as set forth in the 1965 Highway Capacity Manual are deemed adequate for the highway design engineer's use. There appears, however, to be less confidence and uniformity evident in the use of similar criteria for arterial streets. For this reason a portion of this research study deals with the capacity and quality of service on arterial streets.

The 1973 edition of the AASHTO publication "A Policy on the Design of Urban Highways and Arterial Streets" introduces the section on intersections at grade with the following paragraph:

"Capacity, speed and safety on most urban arterial highways depend upon the number, type, and spacing of intersecting streets. The higher types of arterial streets have occasional grade separations or interchanges where they cross heavily traveled arteries but for the most part, the layout and traffic control devices for the intersections at grade are the key elements for the safe and efficient operation of the arterial highway. Since intersections at grade are such a vital part of the urban transportation system, a great challenge lies in their design and operation."

The text continues to state: "Capacity analysis is one of the most important considerations in the design of intersections."

The term "capacity of an intersection" may be used rather loosely to describe the capacity of the intersection as a whole to handle traffic or, more specifically, the capacity of an individual approach to a signalized intersection. The latter definition is perhaps most widely used (2) and is the form adopted in this discussion.

Signalization of an intersection has as its objectives (3):

- The provision for the orderly movement of traffic
- When used in construction with proper physical layouts and control measures, to increase the capacity of an intersection
- Reduction in the frequency of certain types of accidents, especially the right angle type
- Under favorable conditions signals can be coordinated to provide for continuous or nearly continuous movement of traffic at a definite speed along a given route
- They may be used to interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross.

Whereas the directional freeway interchange achieves the removal of conflicts by separation of traffic movements in space, the signalized intersection is designed to achieve similar objectives by separation of conflicting traffic movements in time. This is achieved by apportioning separate time intervals to the movement of different traffic streams.

Under these circumstances it is entirely appropriate that attempts to measure the level of service afforded by signalized intersections have concentrated on measures relating to the effectiveness with which time is apportioned to different movements.

American practice has generally placed emphasis on measures relating to the probability of clearing queues (2, 4). On the other hand, British

(5) and Australian (6) practice has tended to place more emphasis on delay as a measure of effectiveness. However, the Australian guide (6) does include a formula and nomograph for determining the probability of clearing queues.

Miller (7) has shown that signal settings selected on the basis of minimizing delay will, in general, result in shorter cycles than settings based on a high probability of clearing queues.

Thus the traffic engineer is thrown into some immediate conflict in determining the level of service provided on an arterial street. Should he aim to minimize the average delay to all vehicles and thus minimize the time units of costs of travel; or should he attempt to satisfy the individual driver more who is concerned with clearing the intersection on the first green signal indication (preferably without stopping)?

AREAS OF STUDY

This study was concerned with exploring those conflicting demands, identifying suitable models for measures of effectiveness and establishing some boundary conditions to assist design and operation engineers in analyzing the level of service afforded at signalized intersections.

The following broad approach was adopted in conducting this study:

1. A review of the literature was undertaken to ascertain the most reliable models for determination of measures of effectiveness. These models were then examined for compatibility with existing approaches used by the Texas Highway Department.
2. A representative view of people with expertise in the field of intersection performance was obtained by inviting a sample of 120 traffic engineers within the state of Texas to complete questionnaires relating to levels of service.
3. A series of field studies were undertaken to establish a basic framework of saturation flow data. These data were then used to compare alternative methods of calculating the probability of clearing queues with the results of the field studies.
4. The theoretical models were studied in conjunction with the field results to ascertain suitable boundary conditions for different levels of service.

A REVIEW OF EXISTING MEASURES OF EFFECTIVENESS

Approach Capacity

With the exception of special cases where reference is made to other publications which are noted, the following notation has been adopted in this report.

C = Cycle length (seconds)

d = Average delay to vehicles on an approach (seconds/vehicle)

g = Effective green time on an approach (seconds)

G = Total green time on an approach plus yellow (seconds)

ℓ = Lost time on an approach (seconds)

s = Saturation flow rate (T.C.U.'s/sec)

q = Average arrival rate (T.C.U.'s/sec)

T.C.U. = Vehicular volumes expressed in equivalent through car units

$y = \frac{q}{s}$ = The ratio of average arrival rate to saturation flow

$x = \frac{C}{sg}$ = The ratio of the average number of arrivals per cycle to the maximum number of departures/cycle or the saturation ratio

$\lambda = \frac{g}{C}$ = The proportion of a cycle that is effectively green

Miller (7) describes the capacity of an approach as "The maximum sustainable rate at which vehicles can pass through an intersection from the approach under the prevailing conditions." The capacity is a function of the saturation flow rates (s) plus the proportion of time the signal is effectively green.

$$\text{Capacity (Q)} = \frac{gs \times 3600}{C}$$

Thus the capacity may be increased either by increasing the saturation flow rate (addition of lanes, etc.) or by increasing the proportion of green time allocated to that approach (g/C).

Units of Measurement--The Highway Capacity Manual (2) measures flow in terms of vehicles passing through the intersection. The capacity of an approach corresponds to flow at a load factor of 1.0 and a peak hour factor of 1.0 and is considered to be a function of approach width, vehicle mix, movement mix, and environment. Thus the basic expression for capacity can be written as:

$$Q = (\text{Basic Capacity}) \times f(\%\ \text{Right Turns}) \times f(\%\ \text{Left Turns}) \times f(\text{Environment}) \times f(\text{Peak Hour Factor}) \times f(\%\ \text{Trucks \& Buses}) \times f(\text{Local Bus Factor}) \times f(\tfrac{g}{c})$$

The Australian Capacity Guide (6) considers the capacity to be a function of the intersection geometry, environment, and traffic controls. Thus:

$$Q = f(\text{Geometry}) \times f(\text{Environment}) \times f(\text{Parking}) \times f(\tfrac{g}{c})$$

In this context, capacity is expressed in equivalent through car units (T.C.U.'s) and may be considered independent of the composition of the traffic stream. Calculation of the cycle length and proportions of green time, delay, and probability of clearing queues is then carried out by converting approach volumes to through car units.

This study adopted the Australian methodology because: (1) It facilitates comparison of performance data between different intersections with similar geometry; (2) Considering "capacity" to be a function of the intersection design and signal timing appeared logical and facilitates analysis of the effects of differing design or operational flows (a.m. and p.m. peaks and off-peak) for the same intersection.

Approach Width vs. Number of Lanes--Saturation flows may be measured in relation to either the width of the approach roadway or in the number, type, and width of approach lanes. The Highway Capacity Manual (2) and English practice (5) consider approach width to have a more significant relationship to capacity.

The critical lane analysis procedure developed by TTI as reported and summarized by Drew, Pinnell, Capelle, and others (4, 19, 20), considers the number of approach lanes as the primary indication of capacity. The Australian Road Capacity Guide (6) relates capacity to lane type and lane width.

There appears to be compelling movements to recognize the effect of lane markings and, hence, lane width on capacity. This approach tends to simplify calculations and provides a basis for an improved understanding of the operation of signalized intersections. Consequently, a limited study of saturation flow data was undertaken to assess the effect of lane width on saturation flows in Texas.

A Missouri Evaluation of the Highway Capacity Manual--Pignatario (21) summarized the results of a Missouri State Highway Department survey which compared service volumes predicted by the Highway Capacity Manual with data from actual intersections as follows:

"1. Estimates of intersection approach service volumes for levels of service A, B, and C by the 1965 Highway Capacity Manual are unreliable for Missouri intersections, and tend to be higher than actually occur. At level of service D, estimates were more accurate, and were most accurate for capacity (level E).

2. Traffic does not appear to operate in the manner prescribed by the Capacity Manual until forced to by the pressure of heavy traffic conditions.

3. At intersections where exit lanes are not sufficient to carry the volume indicated by calculations, on very wide streets, on one-way streets, and where left turns are made against traffic from a through lane, actual service volumes may be less than computed.
4. At intersections where the exit legs are much wider than the approach leg, or where traffic tends to be heavily loaded or under "pressure" conditions for part of the peak hour, the actual service volume may be higher than the computed value.
5. It appears that service volume is not as closely related to approach width as indicated in the manual. When service volumes for all approaches in the 0.3 to 1.0 load factor group were recomputed using a standard lane width of 10 ft. (parking lanes, 8 ft.), the range of errors (STD) was reduced from 26.1 to 21.1 percent.
6. City size appears to have less influence on service volume than indicated in the Capacity Manual.

A Recent Identification of Research Needs--Highway Capacity Committee - Identification of Research Needs, May (22), summarized the results of an April, 1974 questionnaire that was distributed to all Highway Capacity Committee members and selected Highway Capacity Manual users. The survey listed the following areas of research in descending order of priority:

1. Width of approach vs. number of lanes
2. Effect of left turning movements
3. Load factor vs. delay evaluation
4. Overall urban arterial capacity
5. Influence of signal timing
6. Effect of special turn lanes and/or phases
7. Total intersection approach to capacity

8. Influence of parking
9. Effect of pedestrians
10. Saturation flow studies

The results of this survey were not available to the research agency until August, 1974. However, they tend to support the first phase of this study which was concerned with item 1 and, hence, item 10.

Measures of Quality of Service

General--The measures of effectiveness most frequently used in analyzing signalized intersections are:

- Delay
- Load factor
- Probability of Clearing Queues

There are numerous other measures that may be considered. For example, flexibility to adapt to varying land uses, cost, user cost, acceleration noise, accident rate, etc. However, the principal measures listed above are generally susceptible to measurement and can be related either directly or by implication to other measures. For example, low load factors and delays and high probability of clearing queues tends to infer:

- Lower travel times
- Reduced traffic interruption
- Improved safety
- Improved driving comfort and convenience
- Improved freedom to maneuver
- Lower vehicle operating costs

It is generally more convenient to use the principal determinants (delay, load factors, and probability of clearing queues) in an analysis of intersection performance than to evaluate the latter factors, many of which involve trade-offs between noncomparable variables or the use of subjective judgments.

The characteristics required of a model relate directly to its proposed use. Ideally it may be desirable to obtain a model that predicts intersection performance precisely. However, in practice, due to the dynamic nature of the traffic demand which fluctuates rapidly throughout the hour, a high level of precision in describing performance is unlikely to be achieved.

Under these circumstances, the absolute precision of a model may be less important and the reliability of the model in predicting trends in performance may be more important. Indeed it is exceedingly difficult to visualize a relatively simple model that could be used to assess intersection performance over a significant period.

Delay Models--Extensive work carried out in England (5), USA (8), and Australia (6) has resulted in the development of a number of delay models for the approach to signalized intersections. The models can be conveniently classified by the distribution of arrivals.

Expressions for delay with regular arrivals while having some theoretical interest lack the realism required in a deterministic study of intersection behavior (particularly for the cases where the saturation ratio exceeds 0.5).

Winsten (9), Dunne (10), and Potts (11) have presented results for delays occurring and results of binomial arrivals.

Probably the most widely adopted delay model is that derived by Webster (5) for the case of Poisson Arrivals:

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65\left(\frac{C}{q^2}\right)^{1/3} x^{(2+5\lambda)}$$

The model was originally derived from the results of a computer simulatio by Webster with the assistance of Welding wno suggested the second term.

Webster showed that the estimates of delay given by the model agree closely with actual delays observed at a number of sites.

Webster found that the third term of the expression represented about 10 to 15 percent of the total so that, for practical purposes, the relationship can be simplified to:

$$d = 0.9 \left(\frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)}\right)$$

Miller (7) presented another model for the Poisson Arrival case:

$$d = \frac{1-\lambda}{2(1-y)} \left(C(1-\lambda) + \frac{\exp\left(-1.33(\lambda Cs)^{1/2}\,(1-x)/x\right)}{q\,(1-x)}\right)$$

The above expression was modified to the form adopted by the Australian Road Research Board:

$$d = \frac{C-g}{2C(1-y)} \left[\frac{2}{q} E_Z + (C-g)\right]$$

where $E_Z = \dfrac{e^{-1.33\phi}}{2(1-x)}$ and

$$\phi = \frac{1-x}{x} \sqrt{sg}$$

In this case, E_Z represents the average number of vehicles left over when the light turns red.

Newell (8) and Miller (7) have developed expressions for the more general arrival models. Newell's model is as follows:

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{IH(\mu)x}{2q(1-x)} + \frac{I(1-\lambda)}{2s(1-\lambda x)^2}$$

where $I = \frac{\text{Variance of the Number of Arrivals per Cycle}}{\text{Mean Number of Arrivals per Cycle}}$

and $\mu = \frac{(sg-qC)}{Isg}$

μ is a dimensionless measure of the space capacity of an approach and $H(\mu)$ is a function obtained by numerical integration that falls from 1 at $\mu = 0$ to about 0.25 at $\mu = 1$ and decreases closer to zero as μ expands to infinity.

Miller's model is as follows:

$$d = \frac{(1-\lambda)}{2(1-\lambda x)} \quad C(1-\lambda) + \frac{(2x-1)I}{q(1-x)} + \frac{I + \lambda x-1}{s}$$

where I is as above. The value of I=1 corresponds to the case of Poisson Arrivals.

Hutchinson (12), in a comparison of alternative models, shows that, for the case of I=1 (Poisson Arrivals), the models proposed by Miller, Newell, and Webster generally agree within ±10% for practical values of C and λ.

Webster's delay model has been selected for use in this report for the following reasons:

- Alsop (13) has shown that the first two terms correspond to the theoretical values for delay in the uniform arrival case plus the theoretical correction due to the overflow effects with random (Poisson) Arrivals. This report will be concerned primarialy with the Poisson Arrival case.
- The model has been compared with observations at verious traffic flow sites in London and San Francisco. The results agreed with observed delays of between 10 and 15 percent.
- The model has been widely adopted in both the USA and England.
- The model is currently used by the Texas Highway Department (23).

Load Factor Models--For the general case of signalized intersections, delay generally decreases as cycle length is reduced until a minimum value of delay is achieved. Thereafter, delay escalates rapidly with further reduction in cycle length. In contrast, the probability of clearing queues decreases with decrease in cycle length.

The Highway Capacity Manual (2) adopts load factor as its measure of service. A phase is considered to be loaded when:

- There are vehicles ready to enter the intersection in all lanes when the signal turns green.
- They continue to be available to enter the intersection from all lanes during the entire phase with no unused time or exceedingly long spacings between vehicles at any time due to lack of traffic.

The Highway Capacity Manual imputes some significance to delay by using terms involving slight, acceptable, tolerable, and intolerable delay in defining the boundaries between various levels of service.

Load factor can be considered to equal:

1 - (Probability of Clearing the Queue) +

(Probability of Exactly Clearing the Queue)

The Capacity Manual utilizes a series of charts relating load factor to approach width, λ, roadway type, traffic stream composition, and environment. These charts are based on field observation of a large number of intersections and, unfortunately, this research effort has been unable to locate:

- Any theoretical basis for the preparation of the charts.
- Any regression models used in preparing the charts.

It is considered desirable to adopt a model that can theoretically describe load factor. Such a model could:

- Be amenable to mathematical evaluation and, hence, applicable in a computer program.

• Be independent of environment and roadway type.

Noting the relationship between load factor and the probability of clearing queues, Miller (14) fitted an expression for load factor to simulation results obtained by May (15).

The resultant expression can be written as:

$$L.F. = e^{-1.3\phi}$$

where

$$\phi = \frac{1-x}{x} \sqrt{sg}$$

The model makes allowance for overflow effects due to randomness in arrivals and appears to provide a reasonable estimator of load factor.

Probability of Clearing Queue Models--Possibly the most widely used method of calculating the probability of clearing queues is the application of the classic Poisson probability model (4).

Let the probability of clearing queues

= Probability (sg or less arrivals per cycle)

$$= \sum_{i=0}^{sg} \frac{(qC)^i \; e^{-qC}}{i!}$$

This model has been used by Drew (4) and others to calculate probabilities of failure:

$$P_{(failure)} = \sum_{i=sg+1}^{\infty} \frac{(qC)^i \; e^{-qC}}{i!}$$

$$= 1 - \sum_{i=0}^{sg} \frac{(qC)^i \; e^{-qC}}{i!}$$

The model by definition assumes that arrivals are Poisson and that no vehicles are left over from the previous phase. The failure to account for spillover effects results in significant discrepancies between the model results and field observations at moderate to high saturation ratios.

Miller (16), using Bailey's method, has shown that, for the case of random arrivals, the probability of the queue (P_o) being exhausted is

$$P_o = e^{-qC} \quad (sg-qC) \prod_{C=1}^{sg-1} \quad \xi i/(1-\xi_1)$$

where the ξ's are the imaginary roots of $\xi^{sg} = e^{qC}$ $^{(\xi-1)}$ which lie within the circle $|\xi| = 1$.

From an analogy to results obtained by Gould in simulating the filling and emptying of reserviors, Miller developed the following expression for P_o:

$$P_o = 1 - e^{-1.58\phi}$$

where $\phi = \frac{1-x}{x}\sqrt{sg}$

It can be seen by comparison of the two Miller models that load factor is approximately equal to one minus the probability of clearing queues. It is therefore superfluous to utilize both load factor and the probability of clearing queues as measures of effectiveness.

The advantage of load factor is that it has been adopted in the Highway Capacity Manual and that most practicing traffic engineers in this country are familiar with it.

The advantage of adopting the probability of clearing queues is that, being more precisely defined and self-explanatory, it may be somewhat more rapidly measured in the field. Since it is also a measure of success as distinct from failure it may have some conceptual appeal.

Volume/Capacity Ratios--In examining the case of uninterrupted flow, the Highway Capacity Manual utilizes the volume/capacity ratio as a descriptor of boundary conditions for various levels of service. The use of such a ratio is analogous to the concept of utilization adopted in queueing theory where

$$\text{Utilization} = \frac{\text{Average Arrival Rate}}{\text{Average Service Rate}}.$$

In considering an approach to a signalized intersection, the ratio of the average arrival rate (q) to the maximum sustained departure rate (saturation flows) may be written as:

$$y = \frac{q}{s}$$

This ratio has been used (6) to describe upper limits for Σy in the design of signalized intersections.

Typical relationships for the cycle lengths that minimize delay take the form

$$C = f(\frac{1}{1-Y})$$

where: $Y = \Sigma y$ (summed over all conflicting approach phases)

For example, Webster's (5) expression for an optimum cycle length is written:

$$C = \frac{1.5L + 5}{1-Y}$$

$L = \Sigma l_i$ where l_i is the last time on a specific phase.

For two phase signalization and values of Y greater than 0.70, these expressions yield excessive cycle lengths and the intersection may be considered underdesigned.

To obtain a volume/capacity ratio that is more directly related to intersection performance, it is necessary to consider the average number of arrivals per cycle (qc) in ratio to the maximum number of departures per cycle (sg). Webster has adopted the symbol "x" to represent this ratio which he termed the saturation ratio.

$$x = \frac{qC}{sg}$$

Values of x between 0.80-0.85 indicate that, for fixed time equipment, queues will be cleared between 60-95 percent of the time. For values of x greater than this range, delays are likely to be excessive. While this ratio provides a useful general descriptor of likely operation, the actual probability of clearing queues is also a function of the phase length and saturation flow.

In summary, both the Y value and the degree of saturation provide useful, general descriptors of service that can be used by designers to make a rapid assessment of a design. However, is desirable to describe potential operations in more precise terms in establishing potential levels of service.

A Questionnaire Survey of Traffic Engineers in the State of Texas

Procedure--To assist in selection of appropriate measures of effectiveness, questionnaires (Figure 1) were sent to 120 practicing traffic engineers in the state of Texas.

The questionnaire was intended to

- Assist in identifying suitable measures of effectiveness at signalized intersections.
- Provide a ranking of alternative measures.
- Provide some quantitative assessment of the relative importance

1. Measures of Signalized Intersection Operation

Please rank the following measures of the quality of service in descending order (1 = most important, 2 = second most important, etc.)

/_/ Probability of Clearing Queues

/_/ Load Factor

/_/ Cycle Length

/_/ Delay

/_/ Approach Volume/Capacity Ratio

/_/ Other (nominate if you so desire)

2. The major physical features that affect the quality of service at an intersection are geometric features, signal system and access control. To determine the relative importance of these we are asking that you assign a numerical weight to each of the features according to their importance. Please apportion your total of weighting points to 15 (for example, 7 + 5 + 3 = 15).

/_/ Access Control

/_/ Geometric Features

/_/ Signal System

3. Access control near the intersection is considered a factor in the level of service provided by an intersection. Please rate the following characteristics of access control according to how you feel they influence the quality of service provided by the intersection. Use a rating scale of 10 = excellent, 1 = intolerable.

/_/ No Access within 200 Feet of Intersection

/_/ No Major Access Points within 200 Feet of Intersection (1 or 2 driveways to low traffic generators permissible between 100 and 200 feet of intersection)

/_/ Driveways to Commercial Establishments (gas stations, small stores within 50-100 feet of intersection)

/_/ No Access Control, Driveways Permitted in the Immediate Vicinity of Intersections, Extensive Length of Curb Cuts on Intersection Approach.

4. Geometric features are considered to be a major factor in the level of service provided by an intersection. Please rate the following typical geometric conditions according to how you feel they influence the quality of service provided by the intersection. Use a rating scale of 10 = excellent, 1 = intolerable.

/_/ Divided Roadway with Separate Left Turn Bays and Channelized Right Turn Lanes

/_/ Divided Roadway with Separate Left Turn Bays

/_/ Divided Roadway without Left and Right Turn Lanes

/_/ Undivided Roadway with Separate Left Turn Lanes (includes painted channelization and/or two-way left turn lanes)

/_/ Undivided Roadways

Figure 1. Attitude Questionnaire

attached to various aspects of intersection design and control.

Completed questionnaires were separated into the following categories:

- Engineers employed by cities or counties.
- Engineers employed by the Texas Highway Department or the Federal Highway Administration.
- Engineers in private practice or employed by private consulting firms.
- Engineers employed by research agencies and universities.
- Graduate students in the Traffic and Transportation program at Texas A&M University.

The responses were analyzed to obtain the mean and standard deviation for each subcategory.

Results--The results of this survey are shown in Tables 1 through 4. Contrary to expectations, there appeared to be widespread unanimity between different employment groups.

In the case of Question 1, 61.5% of the respondents considered delay a more important measure of effectiveness than the probability of clearing queues; 78.3% of the respondents considered the probability of clearing queues a more important measure of effectiveness than load factor. Load factor and cycle length were the least preferred measures of effectiveness. (See Table 1.)

The second question was intended to obtain some assessment of the degree of importance traffic engineers attached to geometric design, signal system considerations and access control as features affecting the quality of service at signalized intersections. The results obtained for this question are summarized in Table 2. Access control was considered to have the least effect on levels of service. Some 62.9% of the respondents considered geometry more important than the signal system in affecting level of

TABLE 1. RESPONSES TO QUESTION #1

MEAN RANKING OF QUALITY OF SERVICE MEASURES

Respondents Employed By		QUALITY OF SERVICE MEASURES									
		Probability of Clearing Queues		Load Factor		Cycle Length		Delay		Volume/Capacity Ratio	
	#	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
Cities	34	2.56	1.34	4.03	1.06	3.50	1.30	1.97	1.12	3.28	1.55
THD-FHWA	31	2.63	1.31	3.70	1.03	3.96	1.22	2.22	1.15	2.37	1.39
Consultants	11	2.64	1.29	4.09	1.22	4.00	1.48	1.82	0.87	2.82	1.25
Research	7	2.43	1.62	4.00	1.62	4.00	1.29	1.71	1.11	3.29	1.25
Grad Stud.	6	2.50	1.38	3.33	1.03	4.33	1.21	2.00	1.55	2.83	1.17
TOTAL	89	2.58	1.32	3.88	1.09	3.87	1.29	2.01	1.12	2.89	1.44

TABLE 2. RESPONSES TO QUESTION #2

NUMERICAL WEIGHTING OF IMPORTANCE OF MAJOR PHYSICAL FEATURES

Respondents Employed By	MAJOR PHYSICAL FEATURES					
	Access Control		Geometric Features		Signal System	
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
Cities	3.26	1.66	6.38	2.23	5.35	1.30
THD-FHWA	3.61	1.41	6.35	1.76	4.74	1.44
Consultants	2.91	1.04	7.00	2.19	5.09	2.17
Researchers	3.86	2.19	5.43	4.16	4.43	1.99
Grad Students	3.33	1.51	6.50	1.38	5.17	2.04
TOTAL	3.39	1.53	6.38	2.21	5.02	1.57

TABLE 3. RESPONSES TO QUESTION #3

MEAN RATING OF ACCESS CONTROL CHARACTERISTICS (Scale 10 to 1)

Respondents Employed By	No Access < 200 ft.		Minor Driveways Between 100 and 200 feet		Driveways 50 to 100 feet From Intersection		No Access Control	
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
Cities	9.42	1.54	7.88	1.56	4.52	1.54	2.48	2.67
THD-FHWA	9.52	1.52	7.81	1.08	4.61	1.56	1.94	1.29
Consultants	9.00	1.89	7.80	1.75	4.10	2.18	1.80	1.55
Researchers	9.29	0.95	7.57	1.27	4.00	1.29	2.00	0.58
Grad Students	9.50	0.84	6.17	1.72	3.67	1.75	1.67	1.21
TOTAL	9.40	1.48	7.70	1.45	4.40	1.62	2.11	1.92

TABLE 4. RESPONSES TO QUESTION #4

MEAN RATING OF GEOMETRIC CONDITION EFFECTS ON QUALITY OF SERVICE (Scale 10 to 1)

Respondents Employed By	Divided With Right & Left Turn Lanes		Divided With Left Turn Lane		Divided		Undivided With Left Turn Lane		Undivided	
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
Cities	9.61	1.60	8.27	1.55	4.61	2.01	6.82	1.61	3.00	1.82
THD-FHWA	9.63	0.67	8.33	0.84	4.27	1.72	6.97	1.75	3.43	1.63
Consultants	9.82	0.60	8.36	1.50	4.09	1.92	6.45	1.57	2.27	1.10
Researchers	9.71	0.76	7.86	1.07	4.43	1.62	6.57	0.98	2.71	1.25
Grad Students	10.00	0.00	8.00	0.63	5.00	1.90	6.17	0.75	2.83	0.75
TOTAL	9.68	1.09	8.25	1.23	4.44	1.84	6.76	1.56	3.02	1.60

service.

Questions 3 and 4 were included to obtain some value judgment of the effects of different levels of access control and different design features respectively. The results obtained for these questions are summarized in Tables 3 and 4.

Discussion--Models for determination of average delay or the probability of clearing queues by implication encompass the geometric design, and the volume/capacity ratio at an intersection. The average delay per vehicle is fairly difficult to measure in the field. While there is widespread support for the use of delay as a measure of effectiveness, there is considerable difficulty in drawing boundary conditions between different levels of effectiveness. For example, the difference between delays of 20 and 30 seconds may not be very significant to the driver; whereas the difference between delays of 20 and 80 seconds, in which case drivers may be stopped through two red phases is likely to be far more important. Similarly, if the signal timing selected for a given geometric design is such that delay is "optimized" over all approaches, it is no more than the optimal arrangement for the given intersection. The resulting delays may be quite different to those obtained for another arrangement or at different intersections.

For a given set of demand volumes and a given geometric arrangement, computed delays will not vary significantly if the cycle length is within 0.85 - 1.25 times that calculated by the use of Webster's or Miller's expression and the durations of the green plus amber phases are apportioned according to Webster's method.

Thus we are of the opinion that, in evaluating the geometric design of a signalized intersection, delay should be taken into account by adopting the above general restrictions.

Within the above constraints the performance at the individual approaches to the intersection can be evaluated in terms which relate to the probability of clearing the signal on the first green indication.

INTERSECTION PERFORMANCE STUDIES

General

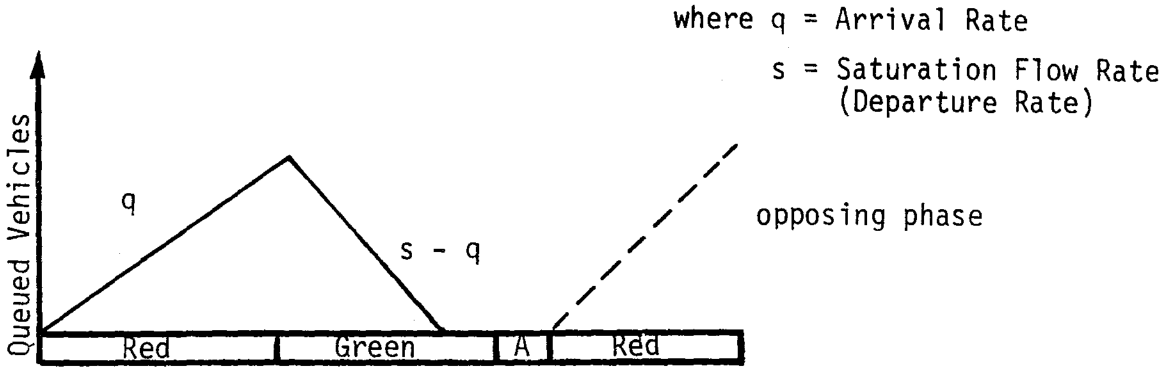
Based upon the number of varying capacity and quality of service models, and recognizing their shortcomings, an effort was made to adapt those methods which seemed to be the most promising for implementation.

Saturation Flow Studies at Signalized Intersections in Texas

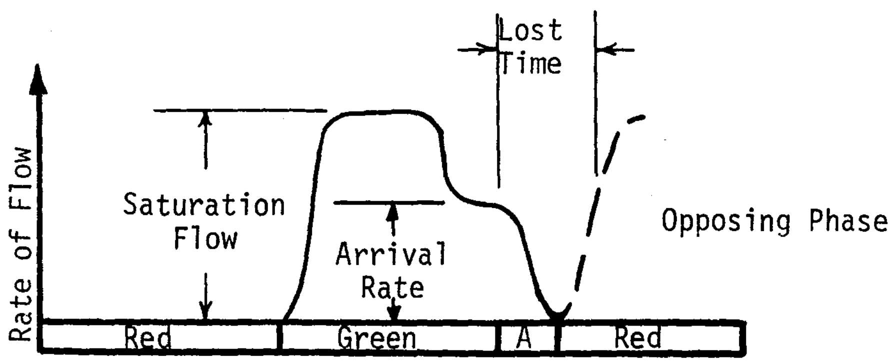
Definition and Theory--As an alternative to present intersection capacity calculation methods as set forth in the 1965 Highway Capacity Manual, a method of capacity calculations based on saturation flows was explored by the use of field studies. To understand the rationale behind this method, consider an approach to a signalized intersection. For the time duration that the signal is red, traffic accumulates on the approaches (Figure 2). When the signal turns green, flow through the intersection increases to a reasonably uniform rate (s) until the queue and vehicles which arrive during the queue discharge phase are dissipated. Flow then continues at the arrival rate (q). This process may be interrupted at any stage by the return of the signal to the red indication.

The model adopted to describe this operation appears to have first been described by Clayton (17) and has subsequently been used by Greenshields (18), Webster (5), Capelle and Pinnell (19, 20), Miller (6), and Drew (4).

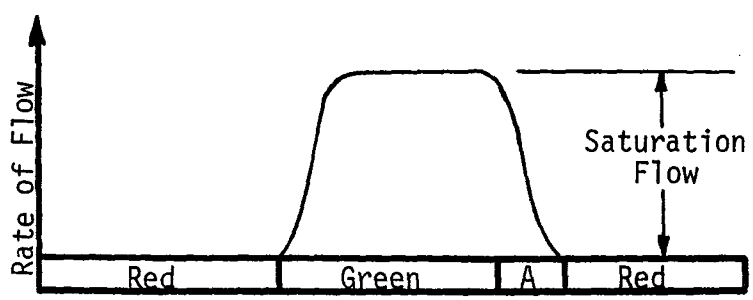
Data obtained by Pinnell and Capelle illustrated that the rate of discharge varies due to starting delays for the first 4-6 vehicles and thereafter is maintained at a fairly uniform rate. In the analysis of intersection performance, it is convenient to replace the individual differences in starting headways of the first few vehicles with an allowance for the starting delay



QUEUE BUILD-UP AND DISCHARGE



BASIC MODEL FOR SIGNAL OPERATION



QUEUE DISCHARGE UNDER NORMAL OPERATIONS

Figure 2. Typical Traffic Signal Operation

and consider the discharge to occur at a uniform rate for the remainder of the effective green time (g). Drew, et al., have adopted this approach and utilize headways between vehicles to describe the saturation flow. Webster, Miller, and others have preferred to use the inverse of this approach and describe departures in terms of flow rates. The Texas Highway Department currently uses both approaches in the analysis of signalized intersections.

While both methods are essentially equivalent, this study preferred to adopt the methods of describing departures by flow rate for the following reasons:

- This approach is consistent with that adopted in most methods of calculating measures of effectiveness (delay and probability of clearing queues).
- The differences between flow rate for differing design configurations appear to be more meaningfully described in terms of flow rates than in terms of headways.
- The approach is consistent with one of the options currently offered in the Texas Highway Department's analytical program.

The model presented above implies either single lane flow or that the saturation flow characteristics occur uniformly over all lanes. Observation of field operation and study of the field results suggested that, except in some cases with queue lengths exceeding the number of departures, this may not necessarily be the case. Consider an approach with a number of traffic lanes. For various reasons drivers may tend to favor one or more of the lanes in which case the discharge characteristics take on the form shown in Figure 3.

The Australian Road Capacity Manual makes some allowance for this operation by adopting a utilization of 60% for curb lanes. The studies conducted for this

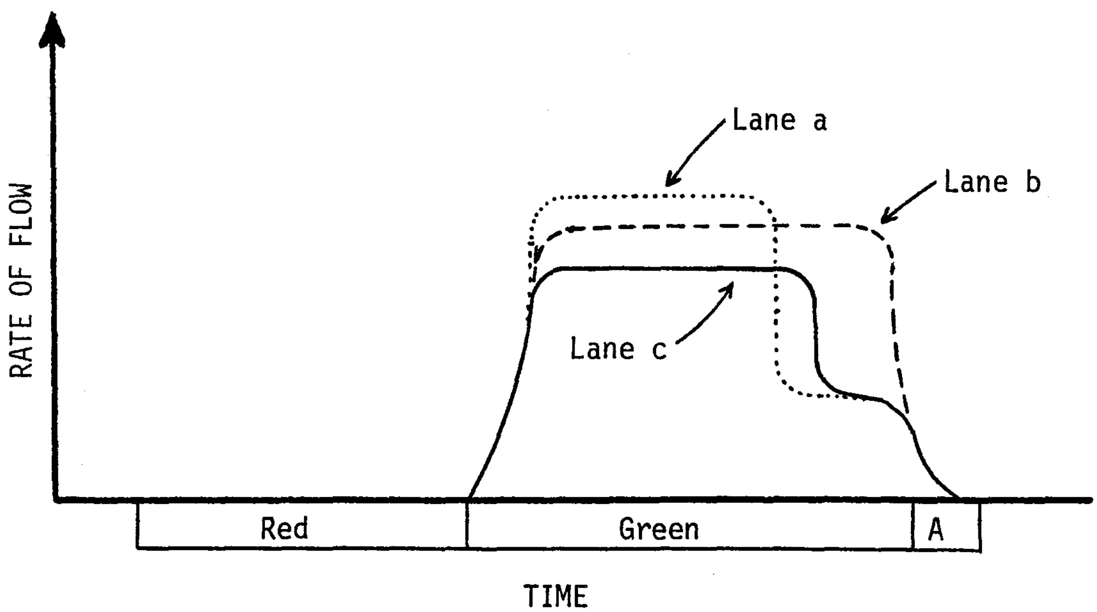


Figure 3. Utilization of Individual Lanes.

project failed to find evidence of similar behavior. The field observations indicated some clear curb lane preferences at intersections of undivided roads with significant left turn percentages, and at intersections with a high percentage of right turns. Low curb lane usage was noted at one intersection with a flared approach which included one additional curb lane at the intersection. In general, it was noted that, unless there were competing turning movements or obvious geometric biases, the preferred lane tended to change during the period of study. For this reason the researchers elected to replace the saturation flow with an equivalent flow taken over all lanes. This was done by adoption of a utilization rate for all lanes.

Method of Study--Studies were conducted at 16 intersection approaches in Houston, Austin, and Bryan-College Station. (A number of intersections were counted twice thus providing 22 data sites.) The intersections selected were characterized as being located on arterial streets in suburban areas. Individual intersections were selected on the basis of traffic volume, and to provide a sample of data from different cross-section types and widths.

The study method developed by Miller (6) was adopted. Three observers were located at the intersection approach. At the commencement of the green indication, Observer #1 started a stop watch and noted the last vehicle in the stationary queue. The watch was stopped when that vehicle crossed the stop line or the signal returned to red, whichever occurred first.

Observer #2 counted vehicles crossing the stop line up to the last vehicle identified by Observer #1. Observer #3 counted all vehicles crossing the stop line during the total green and yellow intervals.

Counts were made on the basis of lane distribution, movement (through, right or left turning), and vehicle type (commercial or passenger car).

To calculate saturation flow rates, the following equivalents were adopted:

One truck or bus	$\doteq$	2.0 passenger cars (p.c.)
One left turn (p.c.)	=	3.0 T.C.U.'s
One right turn (p.c.)	=	1.25 T.C.U.'s
One through (p.c.)	=	1.0 T.C.U.'s

The left turn equivalent was checked using Miller's (6) relationship:

$$E_{LT} = \frac{1.5}{f\,\frac{sg-qC}{g(s-q)} + \frac{4.5}{g}}$$

where q, s, g, and c are the values of these parameters for the opposing traffic; q and s relate only to the through vehicles from the opposite direction.

A uniform starting delay of 1.5 seconds per phase was assumed. Thus, the saturation flow is given by...

$$s = \frac{\Sigma u}{\Sigma t - 1.5n}$$

where Σu is the summation of vehicles in t.c.u.'s flowing over n phases for a total duration of Σt.

Counts were analyzed in two ways. First, flow was considered over all lanes. <u>The rate of discharge of the queue taken over all lanes was calculated using the time required to clear the longest approach queue.</u> Secondly, individual phases were examined and the saturation flow rates were determined on a lane by lane basis using only the data from the critical queue on each phase.

The utilization was then calculated as the ratio of the saturation flows for the approaches as a whole, to the summation of the saturation flows for the individual lanes, expressed as a percentage.

Lanes were classified by width and by movements as follows:

<u>Width Classification</u>

9' - 9.9'   10' - 10.9'   11' - 11.9'   > 12'

<u>Movement Classification</u>

L   Lanes exclusively containing left turn movements accommodated by a separate phase

T   Lanes exclusively accommodating through movements

TL   Lanes accommodating left turns plus through movements

TR   Lanes accommodating through and right turning movements

In general, counts were conducted over approximately one peak period duration. This provided 500-1200 seconds of data per site.

Counts were normally conducted in morning or evening peak periods. Whenever possible, the counting period was selected to coincide with a constant cycle time on the signal controller.

One study was carried out in College Station using a larger number of operators and a 20-pen recorder; the manpower requirements and tedium of producing this type of data precluded use of this technique at the remaining sites.

Signal timing, geometric layouts, plans, and photographs were obtained of all intersections and are held by the research agency.

Results--The results of the studies are summarized in Table 5. The mean results classified by lane width and type are summarized in Table 6.

TABLE 5

SUMMARY OF SATURATION FLOW STUDIES

Approach and Intersecting Streets	Lane		Saturation Flow T.C.U.'s/hour		Utilization (percent)
	Type	Width (ft)	By Lane	By Approach	
Long Point	TR	11	1721	3048	96
Bingle	TL	11	1461		
Long Point	TR	11	1824	3004	92
Bingle	TL	11	1441		
Buffalo Speedway &	T	12	1864	3700	100
Bissonet	TR	12	1826		
Westhiemer	T	12	1774	2190	95
Weslayan	TR	12	1584		
Westheimer	T	11	1733	2912	87
Kettering	TR	11	1601		
San Felipe &	T	13	1819	3740	96
Chimney Rock	TR	11	1769		
San Felipe &	T	12.5	1576	2986	90
Post Oak	TR	12	1737		
Beechnut &	T	12	1862	3717	98
S. Rice	TR	12	1940		
San Felipe &	TL	10	1643	3020	94
Claremont	TR	10	1800		
San Felipe & Voss	L	10	1620	--	--
Texas &	L		1760	--	--
University	T		1510		
	TR		1620		
Texas &	L		1670	--	--
University	T		1870		
	TR		1650		

CONTINUED

TABLE 5, CONTINUED

SUMMARY OF SATURATION FLOW STUDIES

Approach and Intersecting Streets	Lane		Saturation Flow T.C.U.'s/hour		Utilization (percent)
	Type	Width (ft)	By Lane	By Approach	
Long Point &	T	8.9	1370	2459	86
Bingle Modified	TR	9.0	1478		
Burnett &	TR	10	1715	3075	92
Koenig	TR	10	1627		
Guadalupe &	T	11	1727	3105	87
24th	TR	19	1836		
Burnett &	TR	10	1833	3328	92
Koenig	TR	10	1782		
South First &	LT	11.5	1749	3016	88
Oltorf	TR	10.5	1663		
Koenig &	T	9.5	1580	2777	90
Airport	T	11.5	1510		
Koenig &	T	9.5	1796	3245	90
Airport	T	11.5	1795		
Ben White &	T	11	1711	3960	79
Manchaca	T	11	1703		
	TR	12.5	1613		
Koenig &	T	9.5	1774	3245	95
Airport	T	11.5	1705		
Long Point &					
Antoine	LT	11	1510	--	--

TABLE 6

MEAN SATURATION FLOW DATA

Lane Type	Lane Width				
	9-9.9	10-10.9	11-12	≥ 12	≥ 10
T	1574	--	1729	1736	1733
TL	--	1643	1550	--	--
TR	1478	1701	1755	1742	1737
L	--	1620	1715	--	1683

No significant difference was found between T and TR lanes 10 ft. or greater in width. The mean saturation flows for these lanes was found to be 1735 T.C.U./hr. with a standard deviation of 103 T.C.U./hr, (Table 6).

The large variation in flow results between individual intersections is the result of a number of factors. In general, there was no parking of vehicles on the intersection approaches. However, flows did appear to be affected by the combination of signal phasing and site layout. For example, at the intersection of Texas Avenue and University Drive in College Station, a short left turn lane combined with a separate left turn phase appear to reduce the saturation flow rate in the adjacent lane. Flow rates also appeared to vary somewhat with demand. No data were available to support this hypothesis; however, it is suggested that future studies include this variable.

Pending further expansion of these studies by the sponsor, the saturation flow values as shown in Table 7 are suggested as representing the results of this study.

TABLE 7

SUGGESTED SATURATION FLOW VALUES

Lane Width	9' - 9.9'	≥ 10'
Lane Type	Sat. Flows	tcu's/hr
T and TR	1600	1750
TL		1550
L		1700

The values presented above are based on data which were obtained largely on arterial streets in suburban commercial environments. It is probable that different values might have been obtained in differing environments.

Pending any further studies that the Department may undertake, it is suggested that saturation flows be increased by 50 T.C.U.'s/hr. for residential environments and reduced by 50 T.C.U.'s/hr. for intersections in central business districts.

Studies Relating to the Probability of Clearing Queues

Background--The Texas Highway Department currently uses the Poisson probability model (21) and/or the Highway Capacity Manual charts to assist in assessing the level of service at signalized intersections.

The literature survey conducted as part of the project suggested that Miller's model has a number of advantages over these methods. Consequently, part of this study involved a limited test of the reliability of alternative methods of predicting queue clearance and establishing boundary conditions that describe a satisfactory level of service in terms of the probability of clearing queues.

The study was carried out in conjunction with the saturation flow study phase of the project.

The probability of clearing queues was then defined as the proportion of cycles in which the queue was cleared and flow returned to normal prior to the amber indication, as recorded by Observer #2 in the saturation flow studies.

The probability of clearing queues was then calculated using two methods:

(a) Miller's model described previously

$$P1 = 1 - e^{-1.58\phi}$$

(b) Poisson model

$$Pp = \sum_{i=o}^{SG} \frac{(qC)^i e^{-qC}}{i!}$$

Determination of Satisfactory Boundaries--Examination of the results of the field studies and the performance characteristics described by the Miller models lead to suggested boundaries for different levels of service.

Table 8 presents the results of the field studies compared with those predicted using Miller's and the Poisson model. Unfortunately, the sample data are biased in that, at the majority of intersections studied, queues were cleared on more than 80% of the cycles. In which case, both models yield fairly similar results.

At lower probabilities Miller's model appears to be a more reliable predictor (as was expected) due to its accounting for the spillover effects from previous cycles.

The judgment of the research staff was that the majority of intersections were operating satisfactorily when queues were cleared more than approximately 80% of the time. Service was regarded as just tolerable at Westheimer-Weslayan in Houston (observed P_0 = 0.62) and Bingle-Long Point (modified) (observed P_0 = 0.70). Queues were extensive and vehicles were forced to wait several cycles at Bingle and Long Point in the evening peak period when the observed P_0 was 0.20 and 0.44.

Figure 4 presents a relationship of Probability of Clearing Queues and Saturation Flow which may be used as measures of effectiveness for operations and design respectively. To make this comparison, a maximum realistic signal timing is assumed: sg = 15. For a time headway of 2.0 seconds in the critical lane, this value would represent an effective green time of 30 seconds, a realistic upper limit for normal operations. Other values (sg = 10 and sg = 20) are possible but would not greatly alter the quidelines set forth herein.

Across the lower scale, it is noted that recommended levels of service are related to the probability of clearing queues, and along the right hand scale are recommended levels of service for design as related to saturation ratios. It should be noted that the recommended levels of service for operations and design nominally agree, but contrast significantly with levels of service based on Load Factors presented in the Highway Capacity Manual (page 323, Table 10, 13) these are shown across the top of the graph, Figure 4.

By combining the results of the field studies and examination of the form of these relationships, the following boundaries of levels of service for operations and design are recommended.

Recommended Descriptors of Intersection Performance

General Descriptors

1. Demand/Capacity Relationships. To assist the design staff in obtaining a rapid assessment of a potential design, the following general descriptors are proposed:

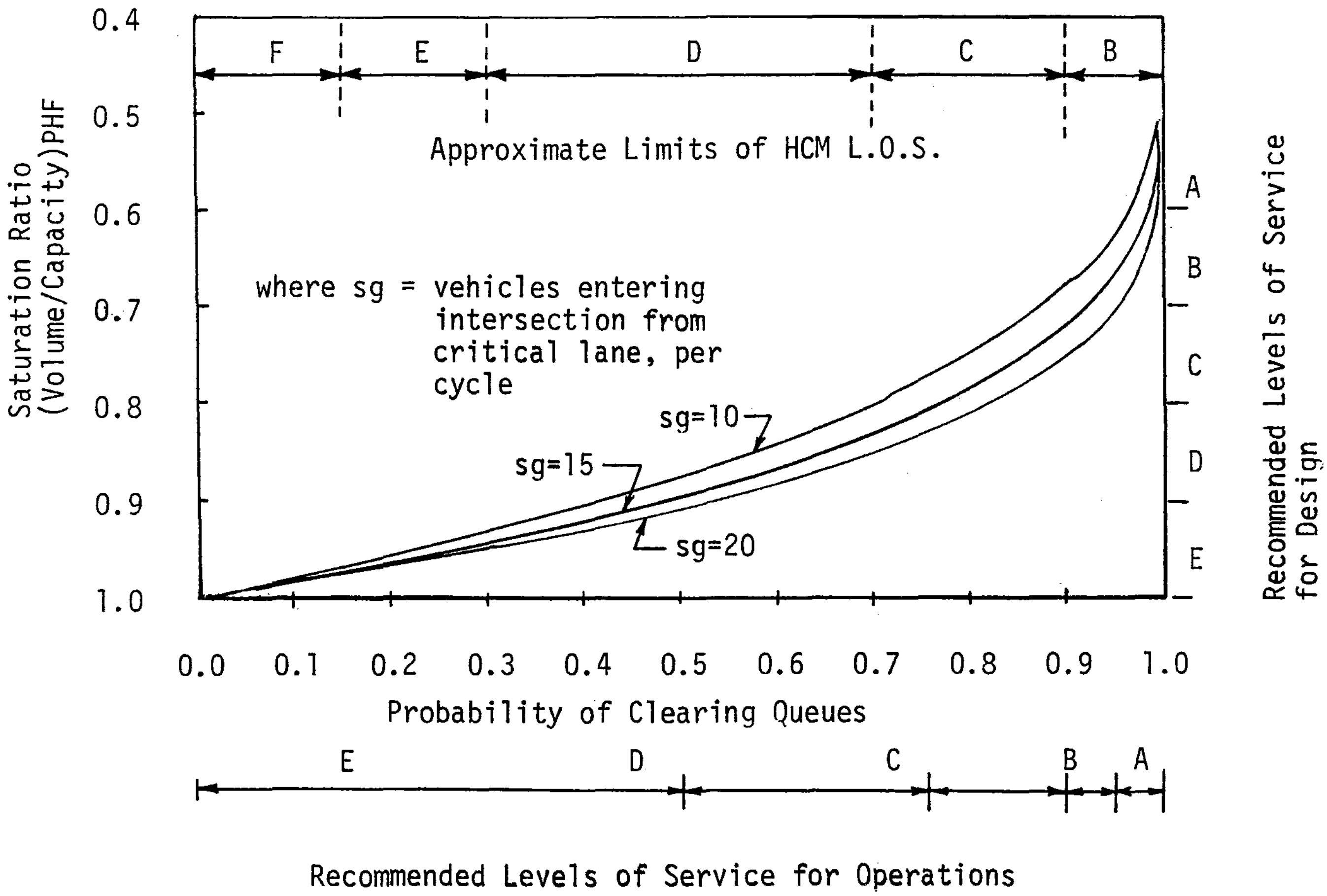
   The sum of the maximum "y" values for simultaneous movements at signalized intersections should preferably fall below the following limits:

   - Two-phase operation — $Y < 0.70$
   - Three-phase operation — $Y < 0.66$
   - Four-phase operation — $Y < 0.63$
   - Diamond interchange, four-phase overlap with total overlap less than the lost time — $Y < 0.75$

TABLE 8

COMPARISON OF ALTERNATIVE METHODS FOR ESTIMATING THE PROBABILITY OF CLEARING QUEUES BASED ON THE RESULTS OF FIELD STUDIES

Intersection	Observed P.	$P_o = 1 - e^{-1.58\phi}$ (Miller)	$P = \sum_{i=1}^{sg} \frac{(qC)^i e^{-qc}}{i}$ (Poisson)
Long Point & Bingle	0.88	0.98	0.98
Long Point & Bingle	0.77	0.83	0.89
Long Point & Bingle	0.20	0.27	0.60
Long Point & Bingle	0.44	0.38	0.60
Long Point & Bingle (Modified)	0.70	0.86	0.90
Westhiemer & Weslayan	0.62	0.75	0.84
College Avenue & Sulphur Springs	0.89	0.95	0.95
San Felipe & Voss	0.77	0.66	0.72
Burnett & Koenig	1.0	0.62	0.82
Burnett & Koenig	0.98	0.66	0.63
Guadalupe & 24th	1.0	1.0	1.0
Koenig & Airport	0.86	0.69	0.73
Koenig & Airport	0.90	0.97	0.97
S. First & Oltorf	0.98	0.67	0.72
Ben White & Manchaca	0.91	0.93	0.92
Beechnut & Rice	0.88	0.96	0.94



"A" Excellent service. Queues cleared 95% or more of the time.

"B" Satisfactory service. Queues cleared 90-95% of cycles.

"C" Limits of stable operation. Queues cleared only 75-90% of cycles.

"D" Potential stability. Vehicles frequently waiting two or more cycles arrival queues cleared 50-75% of cycles.

"E" Unstable operation. Extensive queues formed.

Figure 4. Recommended Levels of Service Based on Values of Probability of Clearing Queues and Volume/Capacity Ratios.

- Diamond interchange, four-phase overlap with total overlap equal to lost time — $Y < 0.80$

- Diamond interchange, four-phase overlap with total overlap greater than the lost time — $Y < 0.85$

These values assume that cycle lengths will generally be limited to between 40 and 80 seconds. The saturation ratio "x" will be limited to approximately 0.80 and green splits will conform to those calculated using Webster's method.

2. Cycle Lengths. Cycle lengths should be limited to 85% and 125% of the optimum predicted using Webster's expression.

   - Two-phase intersections 40-70 seconds
   - Multi-phase intersections 45-80 seconds

Performance Descriptors

1. Delay. Delay is recommended as the primary performance descriptor for level of service. Signal settings should be calculated to hold delay to a reasonable minimum. To this end cycle lengths and the durations of green plus amber phases should be as near as practicable to those calculated using Webster's method.

2. Probability of Clearing Queues. There is considerable difficulty in conducting field measurements of delay and in ascribing levels of importance to variations in calculated delays. Consequently, the probability of clearing queues should be used as the measurable performance descriptor at signalized intersections.

   Broad boundary conditions for probability of clearing queues are suggested that describe different levels of service and presented in Table 9.

TABLE 9

RECOMMENDED LEVELS OF SERVICE
FOR OPERATIONS AND DESIGN CONSIDERATIONS

LEVEL OF SERVICE	FLOW DESCRIPTION	RECOMMENDED VALUES PROBABILITY OF CLEARING QUEUES	RECOMMENDED VALUES VOLUME/CAPACITY RATIO
A	Free Flow	$P_0 > 0.95$	≤ 0.60
B	Satisfactory Operation. Vehicles Wait For Second cycle < 15% of time	$0.90 < P_0 \leq 0.95$	≤ 0.70
C	Satisfactory Operation. Defines the lower limit of satisfactory Operation	$0.75 < P_0 \leq 0.90$	$\leq .80$
D	Potential Instability, Unsatisfactory Operation; Vehicles frequently waiting two or more cycles	$0.50 < P_0 \leq 0.75$	$\leq .90$
E	Unstable Flow; Unsatisfactory Operations; extensive queues formed	$P_0 \leq 0.50$	≤ 1.0

A Framework for Determination of Levels of Service at Signalized Intersections

Procedure--The following procedure is recommended in determining levels of service at an approach to a signal and intersection.

1. Prepare a sketch plan of the approach under study.
2. Record the signal phasing and phase lengths (where these are not known, trial cycle lengths and phase lengths should be determined using Webster's (5) method.
3. Obtain design volumes including percentage of trucks. If these are hourly volumes, convert them to equivalent peak quarter hourly volumes by applying the appropriate peak hour factor.
4. Convert volumes to equivalent through car equivalents. Using the following factors:

1 Bus or truck	=	2 cars
1 Right turn vehicle	=	1.25 through car equivalents
1 Left turn vehicle (Separate turn lane and phase)	=	1 through car equivalent
1 Left turn vehicle	=	E_{LT} (Calculated using Miller's expression)

   Where the signal phasing is not known, E_{LT} may be approximated as 3 for the purpose of initial evaluations.

5. Calculate y-values for all approaches and phases. Sum the maximum y-values for each phase to obtain Y and check against suggested limits.
6. If cycle length is not fixed, calculate Webster's optimum cycle length and phase length. Adjust them to satisfy minimum requirements for phase lengths.
7. Calculate average delay on the approach (Note: For this calculation, the volume q should be in vehicles per second.).

8. Calculate the probability of clearing queues and compare recommended values in Table 9.

Commentary--Delays and probabilities of clearing queues may be estimated by hand or machine calculations or by a computer. There are a number of nomographs available that simplify calculations. To assist in these calculations, a series of tables have been incorporated in this report.

The use of these tables is illustrated in Appendix A which includes typical calculations in determining the level of service at a hypothetical two-phase intersection. Appendix B illustrates a potential application of the Australian Road Capacity Guide method to the treatment of the case of a separate left turn lane without a separate left turn phase.

SUMMARY OF RESULTS AND RECOMMENDATIONS

- Within the limits of the field studies conducted in this phase of the research, intersection capacity is better described by the number of approach lanes.
- The study found no significant difference in saturation flows for lanes 10-12 ft. wide carrying through or mixed through and right-turning vehicles.
- Unless the intersection is operating under congested conditions, the effective saturation flow taken over all approach lanes is approximately 90% of that which might be expected by summation of the capacity of individual lanes.
- While a questionnaire survey indicated a preference for the use of delay as an indicator of level of service, this study was unable to establish well-defined boundaries to describe level of service utilizing delay as a measure.
- It is recommended that delays be kept to a reasonable minimum by adopting cycle lengths in the range of 85% to 125% of that calculated using Webster's method. Subject to the requirements to satisfy minimum phase lengths, the effective green time should be apportioned using Webster's method.
- A relationship is established between Probability of Clearing Queues and Saturation ratio or Volume/Capacity ratio as respective measures of level of service for operations and design.
- To assist in the rapid, early evaluations of trial designs, suggested limits on Y values are included in the body of the report.

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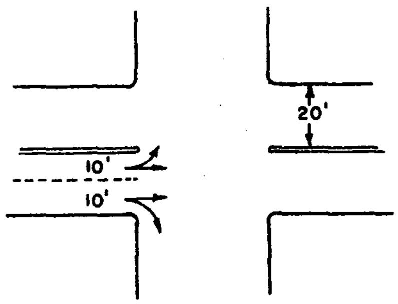
APPENDIX A
EXAMPLE CALCULATION #1

Example Calculation #1

Two Phase Intersection



Given

- 10 Ft. Lanes
- No Separate Turn Lanes
- Cycle Length 60 sec.

  Green Phase 30 sec.

  Amber Phase 3 sec.
- Volumes

  → 86

  → 749

  → 81

  ← 300

  7% Trucks and Buses

1. Convert Volumes to T.C.U.'s

   (a) Calculate E_{LT}

$$E_{LT} = \frac{1.5}{f \times \frac{sg - qc}{g(s - q)} + \frac{4.5}{g}}$$

$s = (1750 + 1550)0.90 = 2970$ T.C.U./hr.

$q = 300 \times 1.07 = 321$ T.C.U./hr.

$f = 0.73$ ref. Figure (A-1)

$g = 33 - 4 = 29$ sec.

$$E_{LT} = \frac{1.5}{\frac{0.73\ (29 \times 2970 - 321 \times 60)}{29\ (2970 - 321)} + \frac{4.5}{29}}$$

$E_{LT} = 1.90$

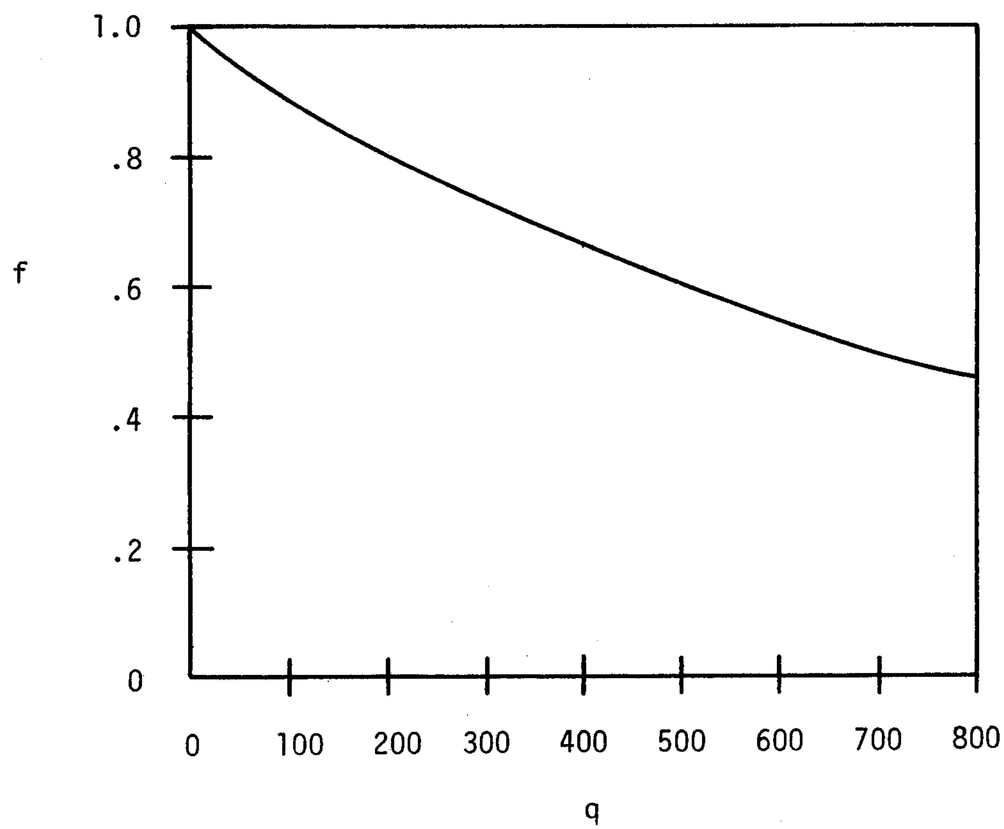


Figure A-1

Value of f for Calculation of E_{LT}

Volumes in T.C.U.'s = 86 x 1.07 x 1.90 = 175

749 x 1.07 = 801

$\frac{81}{916 \text{ vehicles}}$ x 1.07 x 1.25 = $\frac{108}{1084}$ = q in T.C.U.'s

(b) Calculate y, $\frac{g}{c}$, X

Then $y = \frac{q}{s} = \frac{1084}{2970} = 0.365$

$$\frac{g}{c} = \frac{29}{60} = 0.483$$

$$x = \frac{y}{\frac{g}{c}} = \frac{0.365}{0.483} = 0.756$$

(c) Estimate Probability of Clearing Queues (P_o) using Table A-1

Enter Table A-1 with values of sg and x.

Where $sg = \frac{2970 \times 29}{3600 \frac{\text{sec.}}{\text{hr.}}} = 24$ and x (as calculated above) = 0.756

The value from the table then is $\underline{P_o = 0.92}$

(d) Estimate Delay Using Tables A-2 and A-3.

Enter Table A-2 with y = 0.365 (from above) and $\frac{g}{c}$ = 0.483 (from abo

The corresponding value is about 0.17. Multiple this by 60 second

First Term = 60 x 0.17 = 10.2 seconds. Enter Table A-3 with x = 0. (from above)

The corresponding value is 3645. Divide this by 916, the total num

of <u>vehicles</u> (not T.C.U.'s). Second Term = $\frac{3645}{916}$ = 4.0 seconds.

Add First and Second Terms. <u>Estimated Delay = 10.2 + 4.0 = 14.2 $\frac{\text{se}}{\text{ve}}$</u>

(e) Alternatively P_o + Delay May be Calculated from the relevant expressions

$$P_o = 1 - e^{-1.58\phi}$$

$$\phi = \frac{1-x}{x} \quad sg$$

	X: 0.500	0.600	0.650	0.700	0.750	0.800	0.850	0.900	0.950	0.975	1.000
SO											
5	0.971	0.905	0.851	0.780	0.692	0.587	0.464	0.325	0.170	0.087	0.000
10	0.993	0.964	0.932	0.882	0.811	0.713	0.586	0.426	0.231	0.120	0.000
15	0.998	0.983	0.963	0.927	0.870	0.783	0.660	0.493	0.275	0.145	0.000
20	0.999	0.991	0.978	0.952	0.905	0.829	0.713	0.544	0.311	0.166	0.000
25	1.000	0.995	0.986	0.966	0.928	0.861	0.752	0.584	0.340	0.183	0.000
30	1.000	0.997	0.991	0.975	0.944	0.885	0.783	0.618	0.366	0.199	0.000
35	1.000	0.998	0.993	0.982	0.956	0.903	0.808	0.646	0.389	0.213	0.000
40	1.000	0.999	0.995	0.986	0.964	0.918	0.829	0.671	0.409	0.226	0.000
45	1.000	0.999	0.997	0.989	0.971	0.929	0.846	0.692	0.428	0.238	0.000
50	1.000	0.999	0.998	0.992	0.976	0.939	0.861	0.711	0.445	0.249	0.000
55	1.000	1.000	0.998	0.993	0.980	0.947	0.874	0.728	0.460	0.260	0.000
60	1.000	1.000	0.999	0.995	0.983	0.953	0.885	0.743	0.475	0.269	0.000
65	1.000	1.000	0.999	0.996	0.986	0.959	0.894	0.757	0.489	0.279	0.000
70	1.000	1.000	0.999	0.997	0.988	0.963	0.903	0.770	0.501	0.287	0.000
75	1.000	1.000	0.999	0.997	0.990	0.967	0.911	0.781	0.513	0.296	0.000
80	1.000	1.000	1.000	0.998	0.991	0.971	0.917	0.792	0.525	0.304	0.000
85	1.000	1.000	1.000	0.998	0.992	0.974	0.924	0.802	0.535	0.312	0.000
90	1.000	1.000	1.000	0.998	0.993	0.976	0.929	0.811	0.546	0.319	0.000
[illegible]											

PROBABILITY OF CLEARING QUEUES (MILLER)

G/C	0.200	0.250	0.300	0.350	0.400	0.450	0.500	0.550	0.600	0.650	0.700
Y											
0.050	0.303	0.266	0.232	0.200	0.171	0.143	0.118	0.096	0.076	0.058	0.043
0.100	0.320	0.281	0.245	0.211	0.180	0.151	0.125	0.101	0.080	0.061	0.045
0.150	0.339	0.298	0.259	0.224	0.191	0.160	0.132	0.107	0.085	0.065	0.048
0.200	0.360	0.316	0.276	0.238	0.203	0.170	0.141	0.114	0.090	0.069	0.051
0.250	0.384	0.337	0.294	0.253	0.216	0.181	0.150	0.121	0.096	0.073	0.054
0.300	0.411	0.362	0.315	0.272	0.231	0.194	0.161	0.130	0.103	0.079	0.058
0.350	0.443	0.389	0.339	0.293	0.249	0.209	0.173	0.140	0.111	0.085	0.062
0.400	0.480	0.422	0.368	0.317	0.270	0.227	0.188	0.152	0.120	0.092	0.067
0.450	0.524	0.460	0.401	0.346	0.295	0.248	0.205	0.166	0.131	0.100	0.074
0.500	0.576	0.506	0.441	0.380	0.324	0.272	0.225	0.182	0.144	0.110	0.081
0.550	0.640	0.562	0.490	0.422	0.360	0.302	0.250	0.202	0.160	0.122	0.090
0.600	0.720	0.633	0.551	0.475	0.405	0.340	0.281	0.228	0.180	0.138	0.101
0.650	0.823	0.723	0.630	0.543	0.463	0.389	0.321	0.260	0.206	0.157	0.116
0.700	0.960	0.844	0.735	0.634	0.540	0.454	0.375	0.304	0.240	0.184	0.135
0.750	1.152	1.012	0.882	0.760	0.648	0.544	0.450	0.364	0.288	0.220	0.162
0.800	1.440	1.266	1.102	0.951	0.810	0.681	0.562	0.456	0.360	0.276	0.202
0.850	1.920	1.687	1.470	1.267	1.080	0.907	0.750	0.607	0.480	0.367	0.270
0.900	2.880	2.531	2.205	1.901	1.620	1.361	1.125	0.911	0.720	0.551	0.405
Y											
G/C	0.200	0.250	0.300	0.350	0.400	0.450	0.500	0.550	0.600	0.650	0.700

COEFFICIENT FOR THE FIRST TERM IN WEBSTERS DELAY

Table A-2

X=	0.025	0.050	0.075	0.100	0.125	0.150
T=	1.038	4.263	9.851	18.000	28.929	42.882
X=	0.175	0.200	0.225	0.250	0.275	0.300
T=	60.136	81.000	105.823	135.000	168.983	208.235
X=	0.325	0.350	0.375	0.400	0.425	0.450
T=	253.500	305.308	364.500	432.000	508.891	596.454
X=	0.475	0.500	0.525	0.550	0.575	0.600
T=	696.214	810.000	940.025	1088.999	1260.264	1458.000
X=	0.625	0.650	0.675	0.700	0.725	0.750
T=	1687.500	1955.571	2271.114	2645.999	3096.408	3645.000
X=	0.775	0.800	0.825	0.850	0.875	0.900
T=	4324.496	5183.996	6300.637	7802.996	9922.496	13121.990

COEFFICIENT (T) FOR SECOND TERM IN WEBSTERS DELAY (VOLUMES IN V.P.H.)

Table A-3

$$= \frac{0.244}{0.756} \sqrt{\frac{2970}{3600}} \times 29 = 1.578$$

$$\underline{P_o = 1\text{-}e^{-1.58\emptyset} = 0.917}$$

And Delay Using Webster's Simplified relationship is:

$$d = 0.90 \left(\frac{c(1-\frac{g}{c})^2}{2(1-y)} + \frac{x^2}{2q}(1-x) \right)$$

$$= 0.90 \left(\frac{60(1-0.483)^2}{2(0.635)} + \frac{0.756^2 \times 3600}{2x916(0.244)} \right)$$

$$= 0.90\ (12.62 + 4.60)$$

$$\underline{= 15.5 \text{ seconds}}$$

APPENDIX B
EXAMPLE CALCULATION #2

Example #2

2 Phase Intersection With a separate Left Turn Lane

1. Calculations for the through and the through right turn lane are carried out in the same manner as above (excluding the left turns).

2. In the case of the left turns we are now interested in whether the left turns can be cleared by a combination of filtering through the opposing traffic and clearing at the end of the green phase. The method outlined for this calculation is based on the model due to Miller.

   •Using the volumes obtained above the expected number of left turns/cycle

$$= \frac{86 \times 1.07 \times 60}{3600} = 1.53$$

$$F = \frac{s^*g}{3600E_{RT}} \qquad \text{*s taken for opposing lanes}$$

$$= \frac{2970 \times 29}{3600 \times 1.9} = 12.59$$

$\underline{12.59 > 1.53 \therefore \text{ OK}}$