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16. Abstract

This report presents the development and findings of a research project conducted by the Texas Transportation Institute entitled "Development of a Frontage Road Level of Service Evaluation Program -- PASSER III. The research was sponsored by the Texas Department of Highways and Public Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The one year research effort was directed toward several topic areas which included: development of a practical diamond interchange computer program which could be used to evaluate various design and signalization strategies and to determine optimum configurations, development of a related operational computer program for providing frontage road progression, evaluate techniques for making smooth signal transitions, develop methods for estimating freeway and frontage road travel times, and develop design distance separation requirements for entrance and exit ramps to cross-street interchanges on urban freeways with frontage roads. A package of computer programs dealing with the design and operation of signalized diamond interchanges was programmed on SDHPT's computer system, and a user's manual was prepared.

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ANALYSIS OF DIAMOND INTERCHANGE OPERATION AND DEVELOPMENT OF A FRONTAGE ROAD LEVEL OF SERVICE EVALUATION PROGRAM - PASSER III -- FINAL REPORT

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State Department of Highways and Public Transportation
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ABSTRACT

This report presents the development and findings of a research project conducted by the Texas Transportation Institute entitled "Development of a Frontage Road Level of Service Evaluation Program -- PASSER III. The research was sponsored by the State Department of Highways and Public Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The one year research effort was directed toward several topic areas which included: development of a practical diamond interchange computer program which could be used to evaluate various design and signalization strategies and to determine optimum configurations, development of a related operational computer program for providing frontage road progression, evaluate techniques for making smooth signal transitions, develop methods for estimating freeway and frontage road travel times, and develop design distance separation requirements for entrance and exit ramps to cross-street interchanges on urban freeways with frontage roads. A package of computer programs dealing with the design and operation of signalized diamond interchanges was programmed on SDHPT's computer system, and a user's manual was prepared.

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SUMMARY

The continued demand for urban mobility requires that the highest degree of traffic service be obtained from existing and future freeway facilities including the frontage roads and related signalized diamond interchanges. Innovative solutions to selected types of urban freeway traffic problems, such as ramp metering, have been successfully implemented. To facilitate freeway flow during incident conditions, changeable message signing systems have been tested. More effective utilization of the freeway - frontage road system is needed, however, to serve existing peak hour traffic demands and freeway motorists during incident conditions. In recognition of the unique mobility needs of urban motorists, the State Department of Highways and Public Transportation sponsored a cooperative research project with the Texas Transportation Institute in cooperation with the Federal Highway Adminstration entitled "Development of a Frontage Road Level of Service Evaluation Program -- PASSER III which addressed several objectives related to improving frontage road-freeway design and operations. This report describes the project's objectives and study results.

The first major section of the report describes the theory and operational features of the PASSER III computer program. Pretimed or traffic responsive, fixed-sequence, signalized diamond interchanges can be analyzed. All basic interchange signal phasing patterns can be calculated using Webster's method and then analyzed by the delay-offset technique to determine which pattern provides the smallest overall interchange delay. Signal phasing patterns which can be analyzed include the lead-lead, lag-lead, lead-lag, and lag-lag sequences. Both interior and exterior movement delays are accounted for by the program. Interior maximum queue lengths experienced per cycle are also compared to the queue storage provided by the design. The program is structured to evaluate either proposed designs or existing facilities on an individual interchange basis. Example problems are provided to demonstrate program features.

Linked to the individual diamond interchange program, to be used when desired on an optional basis, is the frontage road progression program. This program can analyze the diamond interchange - frontage road network as if it were a signalized arterial. Optimal frontage road progression time-space diagrams can be developed and offsets calculated. A complete frontage road level of service analysis is provided including frontage road travel times.

Several previous methods for implementing various interchange and intersection

signal timing plans, and the required transition phases, are discussed. A simulation study of various transition strategies was conducted and results presented. Practical transition guidelines are provided based on the study results and existing technology and operational experience.

A detailed discussion of theoretical travel time considerations and study results are presented in a subsequent chapter. Travel time predictions for both the freeway and frontage roads are presented during normal and freeway incident conditions. A variable input-output model was developed for predicting future freeway travel times; whereas, a real-time algorithm was formulated and tested for estimating current travel time.

Extensive field studies were conducted on urban Texas freeways to observe and evaluate existing design distances provided between entrance or exit ramps and adjacent cross-street signalized diamond interchanges. It was concluded that more specific separation distance design guidelines were needed. Recommended distances were developed and provided in the report.

Implementation

This report provides the theory of operation and validation studies for the diamond interchange computer program PASSER III currently being implemented on the Department's computer system. This battery of programs will be available to the local district traffic and design engineers through the remote computer terminal facilities. A user's manual of the program is also being developed.

Additional study results on design guidelines for entrance and exit ramp separation requirements to cross-street intersection may also be included in future urban freeway design procedures.

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CHAPTER 1
PROJECT BACKGROUND

Introduction

The expansion of traffic engineering technology in recent years has resulted in innovative solutions to several traffic flow problems plaguing urban areas. For example, ramp metering techniques have been effective in reducing freeway congestion and improving the freeway level of service. To facilitate flow along urban arterials, computerized traffic signals systems have been developed which have proven to be cost-effective. Despite these advances, movement of vehicular traffic remains a major problem in urban areas. Since urban traffic demands for mobility continue to increase, new methods for improving the design and operation of traffic facilities are needed to maintain even current transportation standards. The improvement of traffic operations in urban areas, therefore, remains a continuing goal for traffic engineers.

In recognition of the mobility needs of urban motorists, the State Department of Highways and Public Transportation has sponsored a cooperative research program with the Texas Transportation Institute. During the past several years, this joint research effort has developed or refined many traffic design and operations techniques. These previous research program efforts have resulted in: (1) new design and operations procedures for signalized intersections, (2) multiphase progression analysis of arterial signal systems using PASSER II, and (3) level of service determination for intersection approaches.

This current research project extends the research effort into the area of the freeway corridor, including consideration of the freeway main lanes, frontage road ramps, and signalized diamond interchanges. The PASSER II program, designed to evaluate arterial signal systems, was not applicable to frontage road analysis due to the differences in design and operation of freeway interchanges. Thus, a frontage road signal analysis program, PASSER III, had its genesis.

To fully evaluate traffic operations along the freeway corridor, it is desirable to know what effects signalization has on vehicle operating characteristics in the corridor. Level of service analyses of frontage roads and interchange intersections should include all allowable signal phasing combinations, including 3-phase and 4-phase diamond interchange phasings. Efficient signal transitions from one phasing arrangement to another are desired to avoid excessive delay and unstable traffic flow, particularly if computer control is operative. Also, time-space diagrams obtained from optimization calculations are needed for both design and operations applications.

TTI has reported on an algorithm for predicting the time required to travel past an incident on the freeway main lanes under average flow conditions. Additional work was proposed to develop a variable input-output flow model. Also, there has been little research to determine the time required to travel along an alternate route such as a frontage road and an arterial street. The PASSER II program provided a means to predict the travel time along an alternate arterial street route. Completion of the prediction model is needed to determine whether it is best for the motorist to remain on the freeway or, directed by signing or other means, to select an alternate frontage road or city street route.

A need to consider the effects of the location of freeway entrance and exit ramps with regard to the adjacent diamond interchange also exists. Under certain traffic volume conditions at interchanges and entrance ramps, the spacing between the entrance and exit ramps and the interchange becomes critical to interchange operation. Additional research is warranted to determine if the ramps should be located farther from the interchange to obtain suitable operation.

Project Objectives

The Texas Transportation Institute initiated a one year research effort September 1, 1975, to study the problem areas previously described. During this one year period, seven specific project objectives were addressed. These were as follows:

1. Expand PASSER II Level of Service analysis to include frontage roads and frontage road (and interchange) intersections. The research will include the analysis of diamond interchange phasings generally used (3-phase and 4-phase).
2. Evaluate techniques for making smooth signal pattern transitions from one phasing arrangement to another.
3. Develop PASSER II time-space plot for progression along frontage roads. This will provide a means of designing and operating continuous frontage roads and short segments of continuous frontage roads during non-incident peak and off-peak period conditions and during main lane incident management conditions.

4. Develop a practical variable, input flow, travel time prediction model for the freeway main lanes during freeway incident conditions.
5. Develop a means to predict travel time along an alternate route such as a frontage road when an incident occurs on a freeway main lane.
6. Study the location of entrance and exit ramps with regard to their effects on the operation along the frontage road approaches to interchanges and develop (identify) criteria to determine the appropriate distance away from the interchange for locating the ramps.
7. Analyze the frontage road geometric design procedure used by the Department in regard to the design and operations findings of the research. Recommend any additions to the design and operations manual developed in HPR Project 203.

The following chapters of this report describe the research conducted toward satisfying these objectives and present study results.

CHAPTER 2

OPTIMIZATION OF PRETIMED SIGNALIZED DIAMOND INTERCHANGES AND FRONTAGE ROAD PROGRESSION USING PASSER III

Introduction

The urban signalized diamond interchange is a critical facility for providing high levels of operational performance in the urban freeway corridor, Efficient movement of traffic through the interchange is highly desired. The quality of service provided motorists depends to a large measure on the physical design and type of signalization used. However, there seem to be differences of opinion regarding the best way to signalize a diamond interchange. This may be due in part to the lack of an efficient methodology for analyzing the problem.

The goal of this chapter is to describe a computer program that can determine the design and signalization strategy which minimizes the average delay per vehicle experienced by all vehicles using a pretimed or traffic responsive fixed sequence signalized diamond interchange. This computer program, named PASSER III, is one of a series of signalization programs (1, 2) developed for the State Department of Highways and Public Transportation in Texas (SDHPT).

Munjal (3) has presented a systematic discussion of diamond interchange signalization. Subsequent diamond interchange simulation programs were reported by Munjal and Fitzgerald (4). Much discussion in the literature has addressed the relative merits of four-phase overlap signalization as compared to other types of phasing patterns. Messer, et al. (5) have contributed to this discussion. The authors hope that this study will provide results that traffic engineers can use to accurately analyze their interchange problems.

Signalization Problem

The basic problem addressed is to determine the optimal diamond interchange signalization pattern to service a given set of traffic demands using pretimed and traffic responsive fixed sequence control techniques. Assume the given set of traffic demands are those presented in Figure 2-1. Poisson arrivals are assumed for the exterior traffic flow. Two-lane approaches exist on all arterial through movements and on the frontage roads, whereas the two interior left turn volumes are serviced by one-lane left turn bays having adequate storage capacity. All geometrics and volume assumptions are arbitrary.

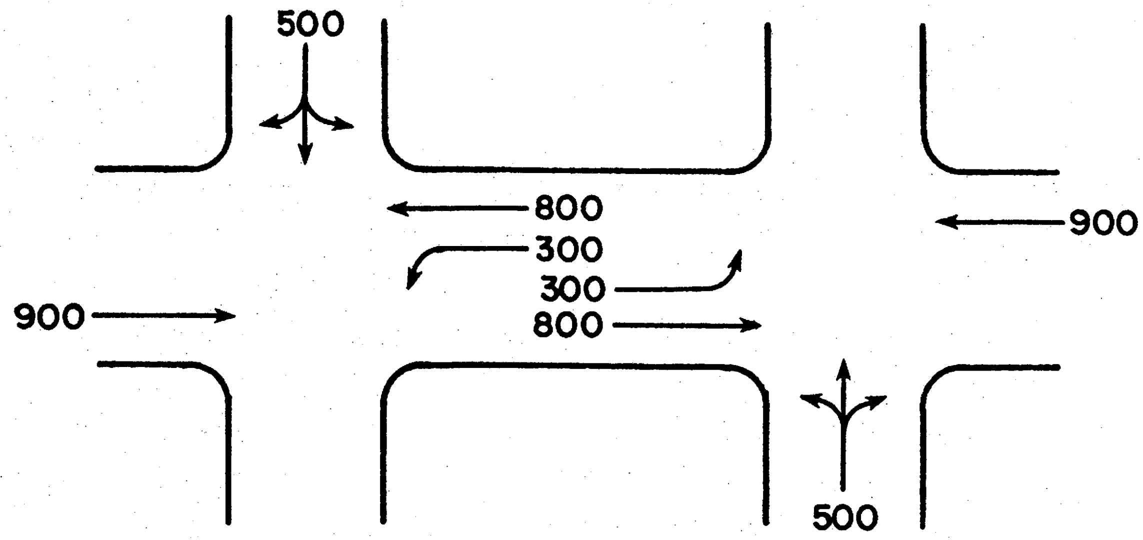


Figure 2-1. Diamond Interchange Example Signalization Problem With Approach Volumes

Signal Phasing

Consider the left-side intersection of a diamond interchange as shown in Figure 2-2 and note how many different signal phases this intersection can have where there will be no conflicts between movements. One phase at this intersection would exist when the off-ramp and the left-turn traffic from the arterial are stopped and the straight-through traffic is moving. This phase is called Phase A, as shown in Figure 2-2. Another phase results when the traffic from the off-ramp is given a green signal. To do this, all other movements at this intersection must be stopped. Call this Phase B. The other phase occurs when the outbound arterial left turn traffic is given a green signal. To obtain this, all the incoming conflicting traffic that may feed the diamond at this intersection must be stopped. Call this Phase C. There are no additional basic phases at this intersection. In addition, there are only 3 similar basic phases at the right-side intersection of the interchange; these form the basis for all possible phasing patterns. Any phases for pedestrians, as well as amber phases, have been excluded from all phasing patterns discussed. Permissive left turns (left turns legal on circular green) in conjunction with a protected left turn phase are discussed on page 2-29 of this chapter, however.

Munjal (3) has shown that the left- and right-side intersections can have either phase order ABC or ACB independently of one another. Order ABC was called leading left turns and order ACB lagging left turns. Thus, there are only four possible basic interchange phasing codes (sequences) that can be generated, as presented in Figure 2-3. Munjal's (3) equivalent descriptions are given in Table 2-1. All of the possible signal phasing patterns that an engineer might devise can be developed by using these basic phase codes and then varying the offset between the two intersections from zero to one cycle length. In this chapter, the offset is defined as the time difference in seconds between the start of left-side phase A and the end of right-side basic phase B.

An example of how an interchange signal phasing pattern results from a given interchange phasing code and offset is presented in Figure 2-4. Phase code No. 1 from Table 2-1 has been selected together with an arbitrary offset.

Signal Timing

Signal green times are usually calculated by the PASSER III computer program as if two intersections were independent of one another. The green times are also normally calculated independently of the interchange phase code selected. One exception is permitted, four-phase with overlap signalization (5).

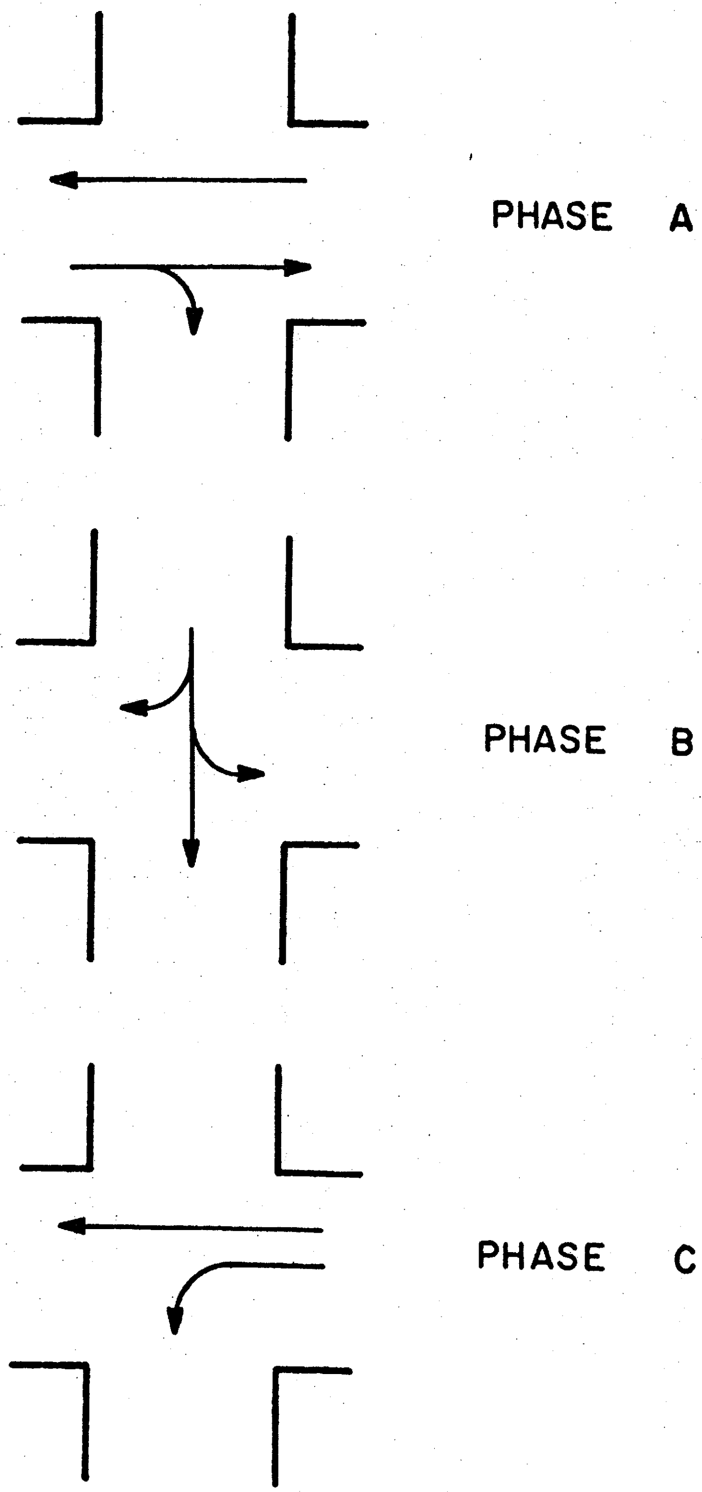


Figure 2-2. Three Basic Phases at Left-Side Intersection of Interchange

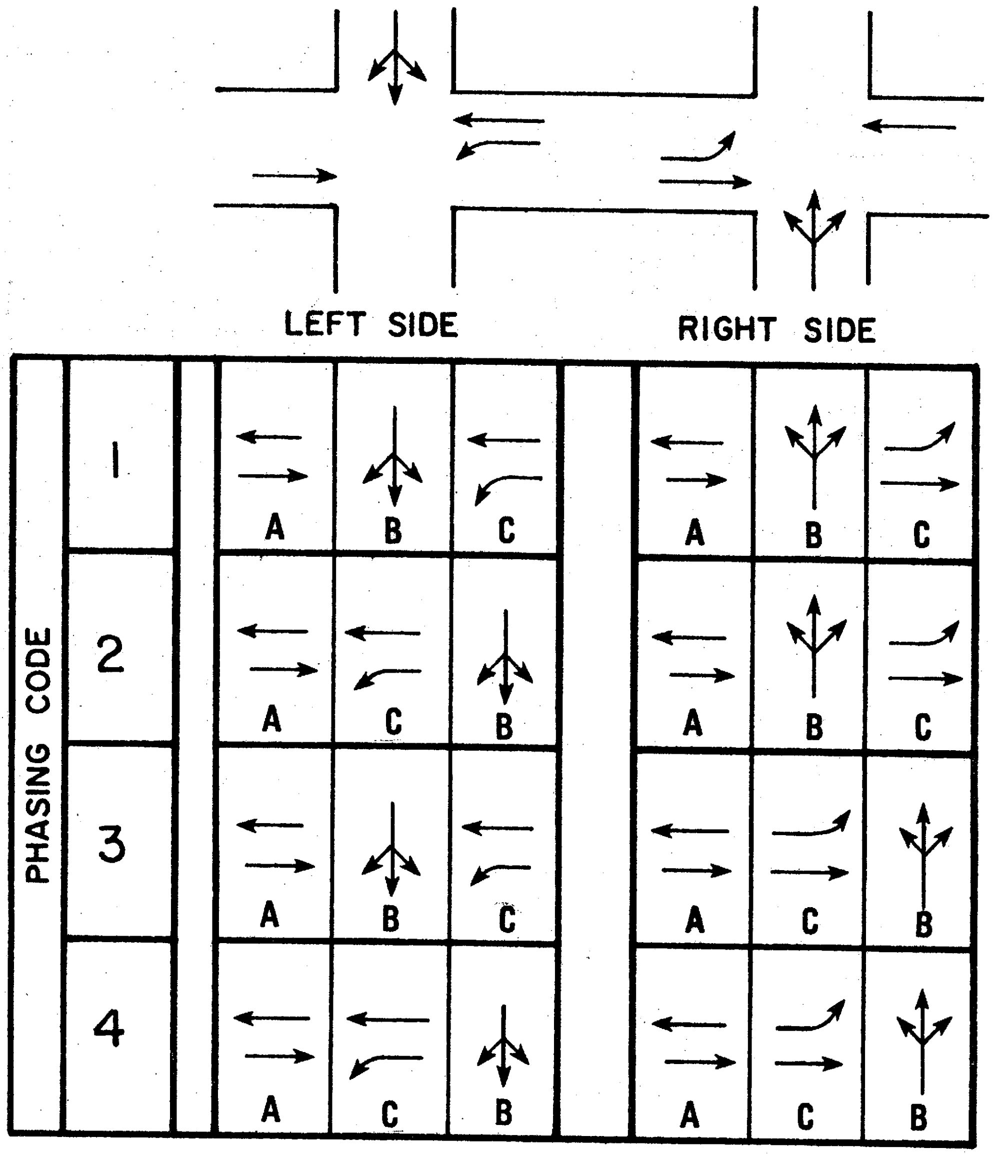


Figure 2-3. Interchange Phases for Phase Codes

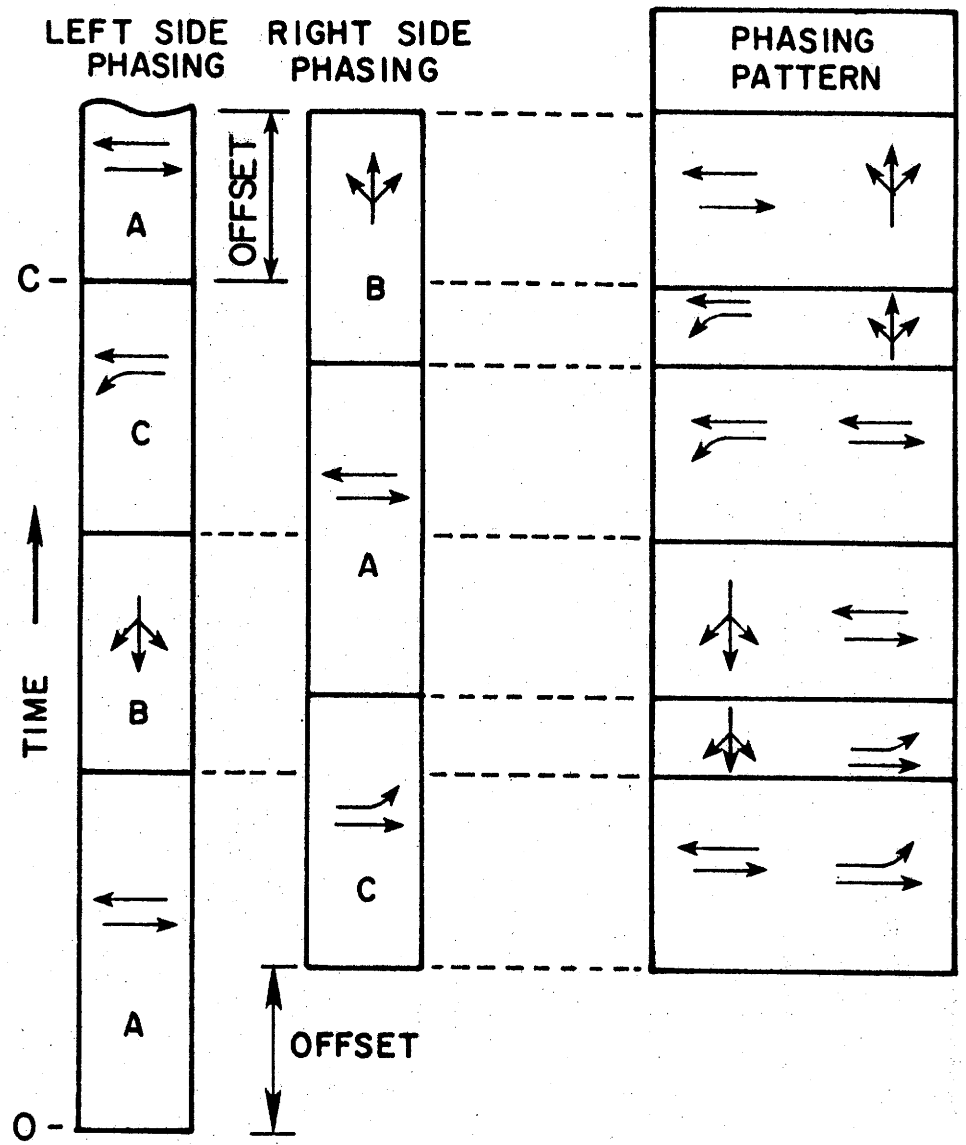


Figure 2-4. Development of Diamond Interchange Phasing Pattern From Phasing ABC:ABC and Offset

TABLE 2-1. BASIC INTERCHANGE PHASE CODE DESCRIPTIONS

Phase Code	Left-side Phase Order	Right-side Phase Order	Munjal Descriptions
1 (& 1A)*	ABC	ABC	lead-lead
2	ACB	ABC	lag-lead
3	ABC	ACB	lead-lag
4	ACB	ACB	lag-lag

* Phase Code 1A is denoted as Phase Code 5 in PASSER III input data.

The green times of phases A, B and C of Figure 2-2 are calculated, in the independent mode of operation, using Webster's formula (6):

$$G = \frac{y}{Y} \cdot (C-L) + \ell \qquad (2\text{-}1)$$

where

G = phase green on approach, sec.

$y = q/s$

q = approach volume, veh./sec.

s = approach saturation flow, veh./sec. green

Y = sum of all y at intersection

C = cycle length, sec.

ℓ = individual phase lost time, sec.

L = sum of intersection phase lost times, ℓ, sec.

Messer and Berry (5) have shown that a formula similar to Equation 2-1 should be used to calculate green times for four-phase overlap signalization (a special case of interchange phase code No. 1). In this case, green times on the four external approaches to the interchange are calculated from

$$G = \frac{y}{Y} \cdot (C + \emptyset - L) + \ell \qquad (2\text{-}2)$$

where

G = phase green on exterior approaches, sec.

$y = q/s$

q = approach volume, veh./sec.

s = approach saturation flow, veh./sec. green

Y = sum of all external y at interchange

C = cycle length, sec.

L = sum of four exterior phase lost times, ℓ, sec.

Ø = sum of interchange overlap (offset times), sec.

Exterior Delay

The operational performance of the diamond interchange is evaluated primarily on the basis of average vehicle delay experienced by all vehicles using the interchange. To begin the analysis procedure, delays are first calculated on the four exterior approaches to the interchange using Webster's delay equation (6)

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

where

d = average vehicle delay for exterior approach movement, sec./veh.

C = cycle length, sec.

q = approach movement flow rate, veh./sec.

λ = proportion of cycle green for approach movement

x = signal saturation ratio, qC/gs

A total of 14 separate exterior movements are analyzed for delay, one for each identifiable turning movement from the exterior approaches. The two arterial approaches have three movements: right turn, thru on arterial, and thru then left turn within the interchange. The two ramps (frontage roads) have four: right turn, thru, left turn then thru on the arterial, and left turn then left turn within the interchange (a ramp U-turn).

Interior Delay

Vehicle delays that occur within the interchange are calculated by a version of the deterministic delay-offset technique (7). Applications of the delay-offset technique have been described in several excellent papers as applied to signalized intersections (8, 9, 10). Validation studies of the delay-offset model for PASSER III are presented in a subsequent section.

As shown in Figure 2-5, a traffic link is defined as a section of street carrying a traffic flow movement in one direction between two signalized intersections. Delay is incurred at the downstream signal of the link, i.e., where

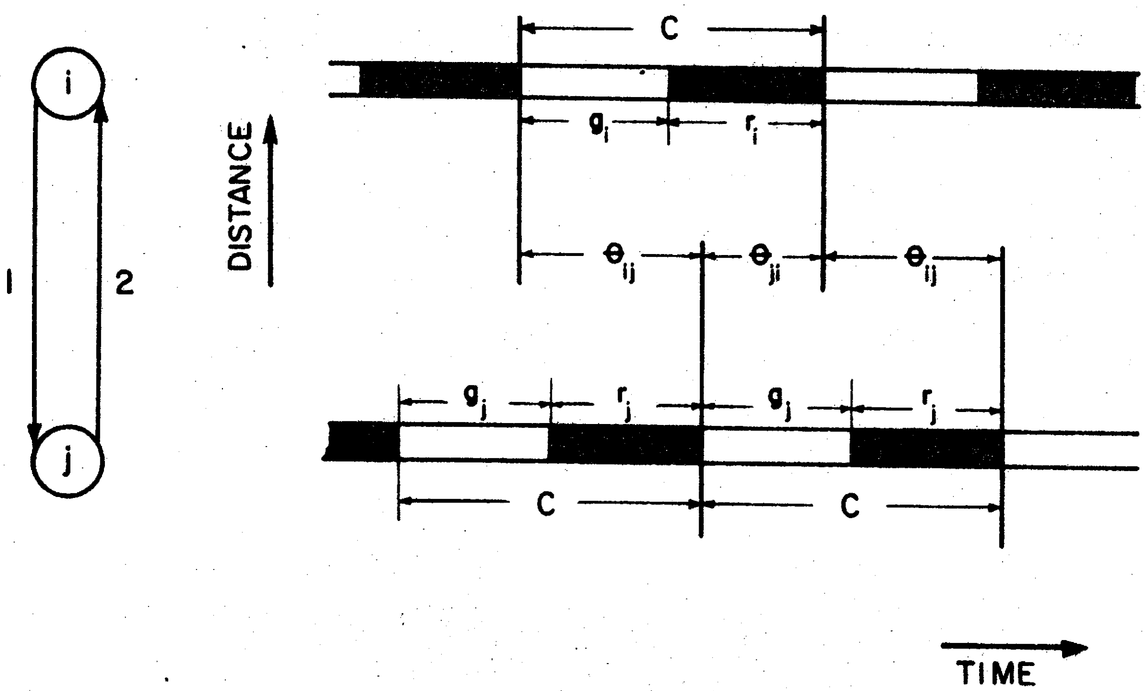


Figure 2-5. Traffic Links Connecting Pair of Adjacent Signalized Intersections

traffic exits the link. The offset across any link may be defined as the time difference between the starting point of green phase A at the upstream signal of the link and the starting point of the next green phase at the downstream signal. It is a directional quantity, assuming the direction of traffic flow along that link. This section describes the flow of traffic through the link's exit signal and the computational procedure for obtaining a delay-offset relationship, given the cyclic flow pattern on the link.

For the purpose of the present discussion, a zero value is assigned to the beginning of the red time at the exit signal of the link in order to establish it as a reference point. Thus, the time interval (0, C) consists of an effective red period (0, r) and an effective green period (r, r + g) so that

$$r + g = C,$$

where C denotes the cycle length. The following notations are also used:

$q_a(t)$ = arrival rate from upstream inputs (vehicles/second),
$q_d(t)$ = departure rate (vehicles/second),
$A(t)$ = cumulative number of arrivals,
$D(t)$ = cumulative number of departures,
s = saturation flow rate during green period.

The upstream arrival rate $q_a(t)$ can be any one of five values in PASSER III, depending on the resulting link time-space diagram. These include the following: q_{Ap}, q_{An}, q_{Bp}, q_{Bn}, 0; where q_{Ap} is the platoon saturation flow during phase A, q_{An} is the normal flow during phase A green when the platoon has cleared, corresponding flows for phase B, and no flow during phase C. Starting with the beginning of any red period at the exit signal, we have the following basic relationships:

$$A(t) = \int_0^t q_a(\pi)d\pi \qquad (2\text{-}4)$$

$$D(t) = \int_0^t q_d(\pi)d\pi \qquad (2\text{-}5)$$

The following assumptions are made:

1. Arrivals are periodic, i.e., for any integer number n,

$$q_a(t) = q_a(t- nC) \tag{2-6}$$

2. The signal is undersaturated, i.e.,

$$A_p < g \cdot s \tag{2-7}$$

where the total number of cars arriving during one cycle (the platoon size) is

$$A_p = \int_0^C q_a(t)dt \tag{2-8}$$

3. The arrival rate during the green time of the signal does not exceed the saturation flow rate,

$$q_a(t) < s \qquad \text{if} \quad r < t \leq r + g \tag{2-9}$$

This implies that, once a queue has vanished during the green period, it cannot rebuild before the next red period begins.

According to these assumptions, all vehicles arriving during a cycle in which the red period precedes the green can be accommodated in that cycle. It follows that the queue is always empty at the end of the green period, and delay time calculations can be confined to a single interval (0, C).

The queue length Q(t) at any time $0 < t \leq C$ is given by the difference between the cumulative number of arrivals and the cumulative number of departures.

$$Q(t) = A(t) - D(t) = \begin{cases} A(t) & \text{if } 0 < t \leq r \\ A(t) - (t-r)s & \text{if } r < t \leq t_0 \\ 0 & \text{if } t_0 < t \leq C \end{cases} \tag{2-10}$$

t_0 denotes the time when the queue disappears ($r < t_0 < C$). By definition, $t = t_0$ when

$$Q(t_0) = A(t_0) - \{t_0 - r\}s = 0 \tag{2-11}$$

If we follow this analysis, the departure rate is described by

$$q_d(t) = \begin{cases} 0 & \text{if } 0 < t \leq r \\ s & \text{if } r < t \leq t_o \\ q_a(t) & \text{if } t_o < t \leq C \end{cases} \qquad (2\text{-}12)$$

The delay incurred by Q(t) queueing vehicles during an interval dt is Q(t)dt. Therefore, the total delay time, $d(\theta)$, incurred by traffic during one cycle (0, C) is represented by the area under the queue-length curve.

$$d(\theta) = \int_0^C Q(t)dt = \int_0^{t_o} Q(t)dt \qquad (2\text{-}13)$$

The delay depends on the exit signal offset, θ. The average delay per car (per cycle), $\delta(\theta)$, is obtained by dividing by the total number of arrivals during one cycle.

$$\delta(\theta) = \frac{1}{A_p}\ d(\theta) \qquad (2\text{-}14)$$

The procedure described yields only one point on the delay-offset curve. To obtain the complete relationship requires that this procedure be repeated while the relative phasing between the exit signal settings and the arrivals is altered so that all possible offsets across the link under consideration are examined.

According to the principles of the combination method, where two or more links occur in parallel, joining two nodes, the delay function of the individual links can be combined with reference to the same offset, to yield a total delay function. Referring to Figure 2-5, $d_1\ (\theta_{ij})$ and $d_2\ (\theta_{ji})$ are calculated for $0 \leq \theta_{ij} \leq C$ and $0 \leq \theta_{ji} \leq C$ respectively. The two offset variables in this case are constrained by the following relationships:

$$\theta_{ij} + \theta_{ji} = C \qquad (2\text{-}15)$$

Consequently, only one of the two offsets can be determined independently. Relating the total delay D to offset θ_{ij}, it follows that:

$$D(\theta_{ij}) = d_1\ (\theta_{ij}) + d_2\ (C - \theta_{ij}) \qquad (2\text{-}16)$$

To obtain the average combined delay function $CD(\theta_{ij})$ from the individual average delay functions, the following formula is used:

$$CD(\theta_{ij}) = \frac{1}{A_{p1} + A_{p2}} \{A_{p1}\ \delta_1(\theta_{ij}) + A_{p2}\ \delta_2(C - \theta_{ij})\} \qquad (2\text{-}18)$$

An optimal offset θ_{ij}^*, between the adjacent pair of signals, is readily obtainable by searching for the minimal value of the combined function. Four links must be evaluated in PASSER III, thru and left turn in each direction.

Interchange Delay

Average delay per vehicle at a diamond interchange is calculated by combining the effects of exterior and interior interchange delays. For an otherwise given set of geometric, volume and signalization inputs, interchange delay changes only as the offset between the two intersections is varied, as illustrated in Figure 2-6.

Figure 2-6 was developed using the volume data in Figure 2-1 with an assumed U-turn volume of 150 vehicles per hour on both ramps, a 70-second cycle length and a 14-second travel time between the two intersections. Interchange delays were calculated using interchange phase code No. 1 (ABC:ABC). Delay is observed to drop to a minimum delay value at a 14-second offset, then begins to rise beyond this minimum delay offset. Also shown in Figure 2-6 is the component of interchange delay occuring within the interchange. External delay remained constant.

Figure 2-7 shows the variation in maximum queue lengths that would occur on the interior left turn and thru lanes for the left-to-right (east bound) arterial as a function of offset. Queue storage capacities, while unlimited in all analyses in this study, are important input constraints to the PASSER III program.

Interchange Phasing Analyses

In addition to the four basic interchange phasing codes (Table 2-1), a fifth interchange phasing code was studied. This code, denoted as No. 1A, represents a special case of the normal No. 1 (lead-lead) interchange phase code. While the phase sequences are the same as No. 1 (ABC:ABC), the four external green times are calculated such that they add to C + 2·(overlap time) (see Equation 2-2). The popular "four-phase with overlaps" signal phasing results if the offset between the two intersection signals is selected to be the same as the overlap time (3).

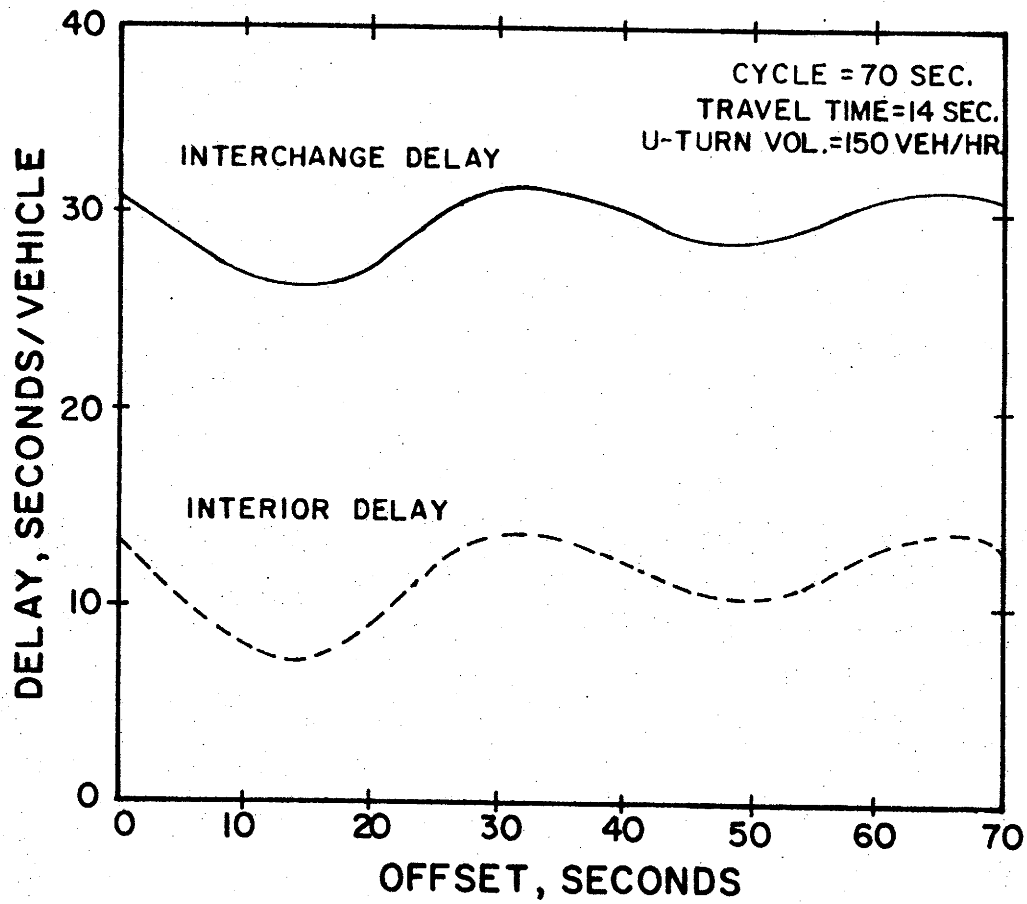


Figure 2-6. Variation in Interchange Delay with Offset For Phase Code No. 1

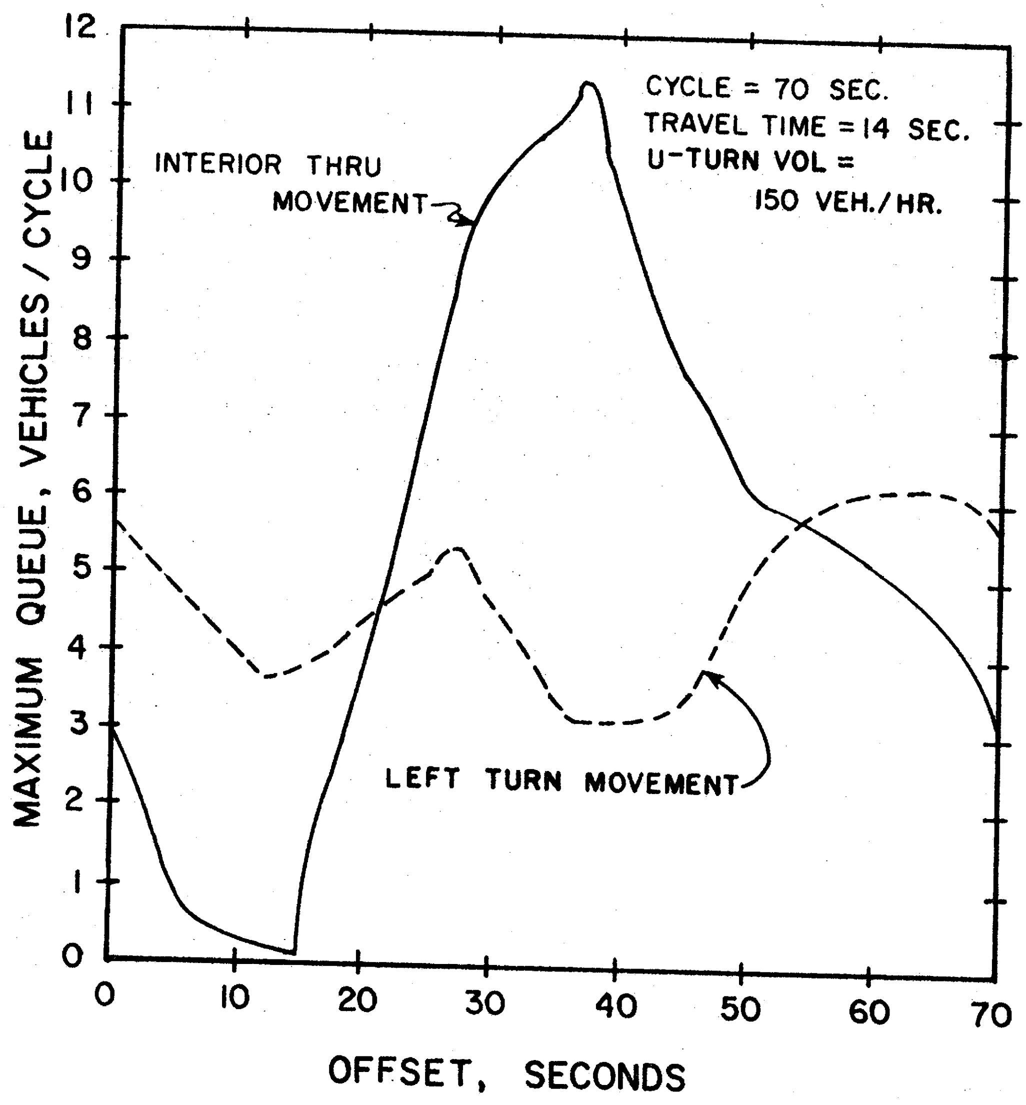


Figure 2-7. Variation in Maximum Queue Length With Offset For Phase Code No. 1

The performance of "four-phase with overlaps" can be determined in Figure 2-8 from the delay curve of interchange phase code No. 1A at an offset of 14 seconds. As might be expected, this offset results in the minimum delay for this set of conditions. Other offsets (overlaps) increase the average interchange delay. It should be noted that the normal unimpeded travel time between the two intersections is assumed to remain constant at 14 seconds regardless of the offset selected. In the real world, motorists may adjust their travel time slightly depending on the offset. If a queue forms on a movement in the interior of the interchange, a queue start up delay or signal lost time is also assumed to occur (11).

A comparison of the performance of two different types of interchange phasing arrangements, No. 1A (ABC:ABC) and No. 4 (ACB:ACB) can be made from Figure 2-8. Minimum delay for code No. 4 occurs at 0 and 70 second offsets, which are the same since the cycle length is also 70 seconds. A zero second offset for No. 4 results in a three-phase, lag-lag interchange signal phasing pattern. Munjal (3) has concluded in his subjective review of the signalization patterns traffic engineers typically use that the four-phase with overlaps and the three-phase, lag-lag patterns were generally preferred patterns. For this example problem, the PASSER III program outputs indicate that these two patterns would operate well in this case. More importantly, however, the phasing pattern which provides the minimum delay can be determined.

While interchange phase codes No. 1, or 1A, and No. 4 may be able to generate good operating conditions if the proper offset for each is selected, there are other possible basic interchange phasing arrangements (Table 2-1) which may provide even superior results. Until all of these phase codes are considered (2 and 3 in Table 2-1), an optimal interchange phasing pattern cannot be determined with certainty.

An example of the performance of all five interchange phasing codes is presented in Figure 2-9. For this problem, three phasing codes (1, 1A and 4) will provide relatively good operations at their respective minimum delay offsets. Interchange phase codes Nos. 2 and 3 do not perform as well as the others. Their performance curves lie in the middle range of delay values and are not as responsive to differences in offset. A total of 350 different interchange timing plans were analyzed to generate the results presented in Figure 2-9. A manual analysis would not be practical and a detailed microscopic simulation may not be economical.

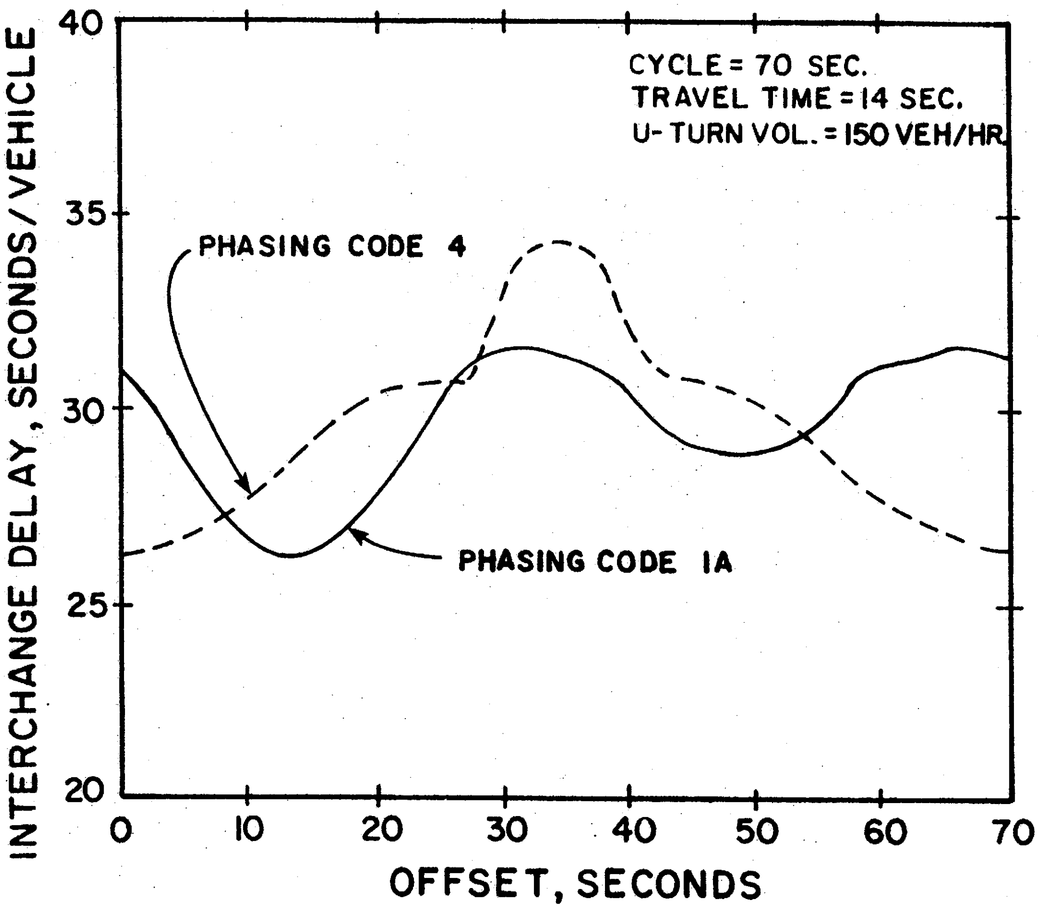


Figure 2-8. Performance of Phase Codes No. 1A (four-phase overlap at 14-second offset) and No. 4

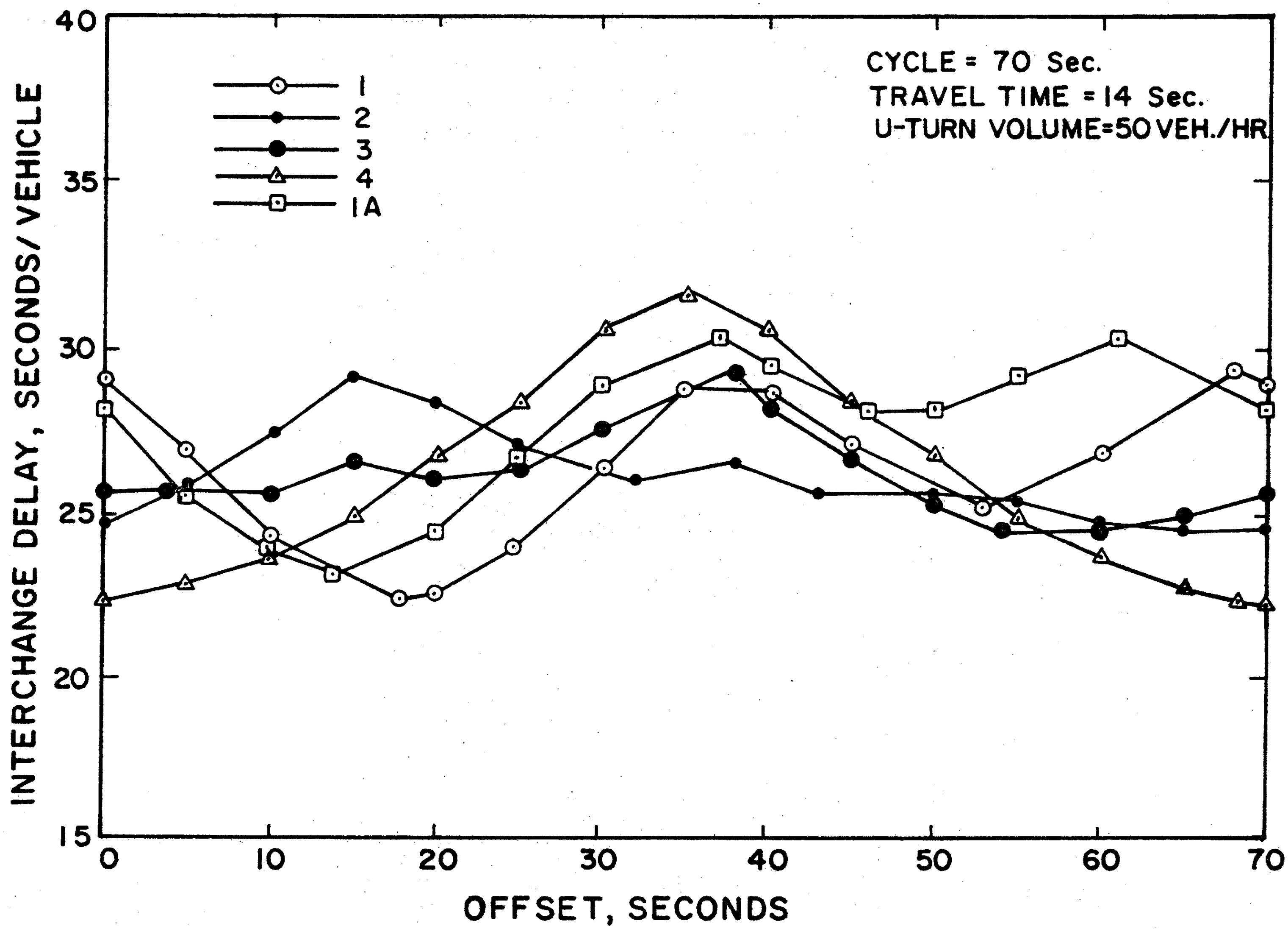


Figure 2-9. Variation in Interchange Delay with Offset for Five Phase Codes

These delay results tend to support the previously discussed general guidelines that the four-phase with overlaps (lead-lead) and the lag-lag signalization strategies are generally preferred signalization strategies. While this general guideline may be useful, it does not indicate which is better. Other phasing codes may operate better under a different set of conditions.

Field Validation Studies

Several methods were used to validate the delay-offset analysis technique. Example problems were mathematically analyzed manually and the results were compared to computer program results. Trend analyses of computer runs were made for consistency and scale. All of these studies resulted in favorable comparisons between calculated and expected results.

Field studies were conducted at signalized diamond interchanges located in Austin and Dallas to further verify PASSER III outputs. The U.S. 290 interchange of I-35 in Austin was studied for approximately two hours one afternoon before, during and after the afternoon peak hour. This interchange is under computer control by the City of Austin. Signal timing data were obtained by the research staff and manually checked in the field. Two computer controlled interchanges on U.S. 75 in Dallas were also studied in detail using portable video recording methods depicted in Figure 2-10. These two interchanges offered special advantages because their signal phasing arrangements (phase codes) could be changed on request by radio and automatically logged by the central computer. Both peak and off-peak studies were conducted.

Field data collected for comparative purposes in all cases consisted of the maximum number of vehicles stored in queue per cycle on the two interior left turn and thru movement approaches of the interchanges. These maximum queue counts per cycle were then used to compute an average maximum queue count for each movement. Volume and signal timing data were used to calculate theoretical average maximum queue counts using PASSER III.

The results of this study are presented in Table 2-2 for the Austin interchange and Tables 2-3 and 2-4 for the two Dallas interchanges. In general, comparisons between calculated and observed results are excellent. Since maximum queue counts are very sensitive measures of effectiveness, differences of 2.0 vehicles between calculated and observed values would have been considered acceptable. This acceptance criterion was not met in only 2 out of 50 cases. In one problem case, (Study #5 in Table 2-2) heavy left turn volumes were observed to block thru traffic and vice versa. The storage capacity of the left turn

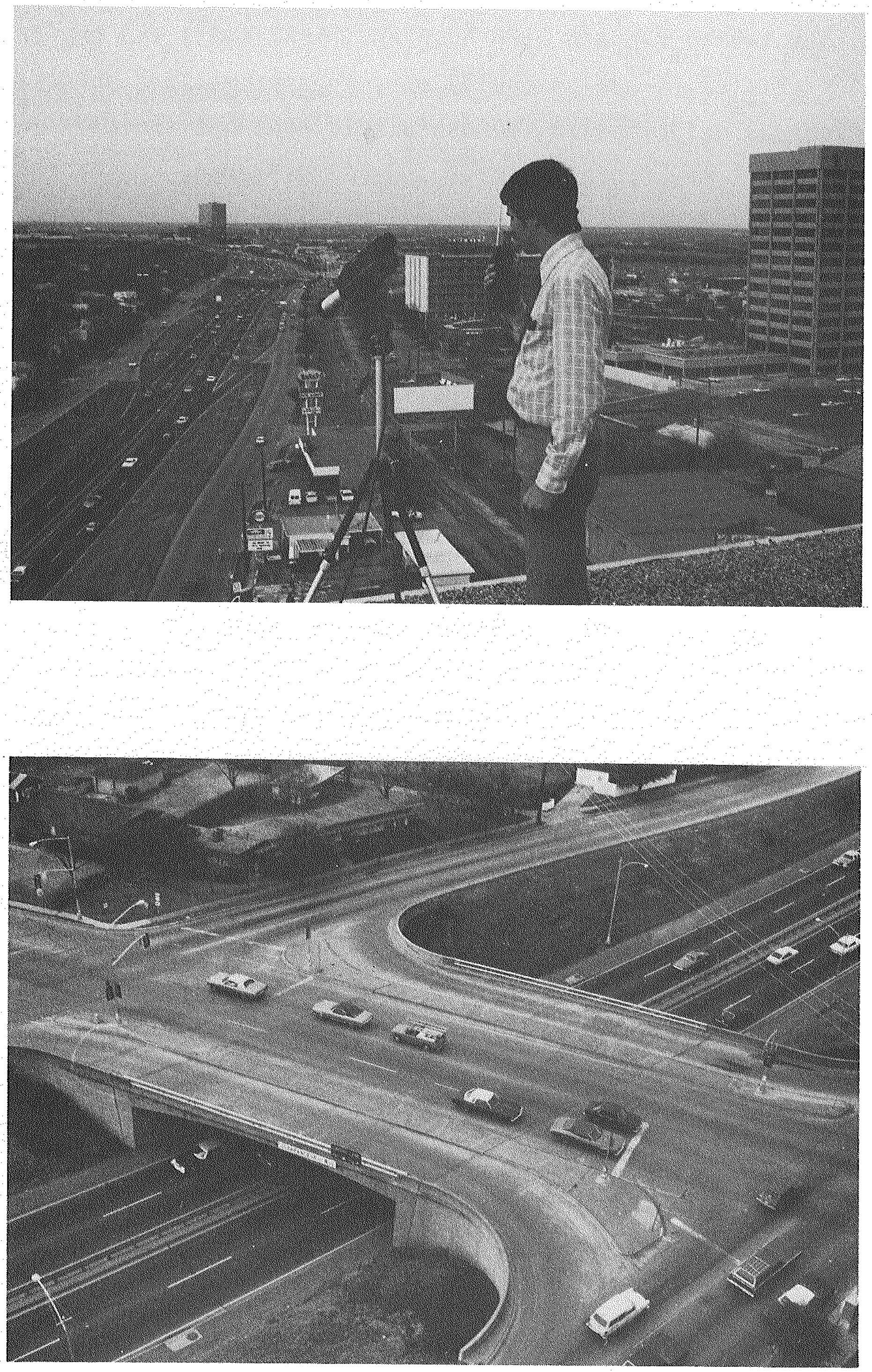


Figure 2-10. Portable Video Recording Methods Used for Diamond Interchange Studies in Dallas

TABLE 2-2. MAXIMUM QUEUE COMPARISONS ON FOUR INTERIOR MOVEMENTS BETWEEN PASSER III AND FIELD OBSERVATIONS AT U.S. 290 IN AUSTIN

Study No.	Signal Phasing	Study Method	Interior Traffic Movements Thru	Left	Left	Thru
1	ACB:ACB	Calculated	0.5	2.5	2.4	3.4
		Observed	0.4	3.5	-	4.3
2	ACB:ACB	Calculated	2.2	3.4	3.3	5.1
		Observed	1.7	3.6	3.2	5.4
3	ACB:ACB	Calculated	2.2	6.0	8.4	7.7
		Observed	2.0	5.9	8.4	7.6
4	ACB:ACB	Calculated	2.2	8.1	1.8	2.3
		Observed	1.2	8.8	1.8	1.1
5	ACB:ACB	Calculated	2.2	8.5	2.9	3.9
		Observed	1.0	12.3	3.0	3.1

TABLE 2-3. MAXIMUM QUEUE COMPARISONS ON FOUR INTERIOR MOVEMENTS BETWEEN PASSER III AND FIELD OBSERVATIONS AT UNIVERSITY DRIVE IN DALLAS.

Study No.	Signal Phasing	Study Method	Interior Traffic Movements Thru	Left	Left	Thru
1	ACB:ABC	Calculated	1.0	-	-	4.6
		Observed	0.6	-	-	4.9
2	ABC:ABC	Calculated	0.0	-	-	0.0
		Observed	0.0	-	-	0.0
3	ACB:ABC	Calculated	4.7	-	-	1.7
		Observed	4.2	-	-	1.2
4	ABC:ABC	Calculated	5.7	-	-	6.5
		Observed	3.2	-	-	6.4
5	ABC:ABC	Calculated	0.0	-	-	0.0
		Observed	0.0	-	-	0.0

TABLE 2-4. MAXIMUM QUEUE COMPARISIONS ON FOUR INTERIOR MOVEMENTS BETWEEN PASSER III AND FIELD OBSERVATIONS AT WALNUT HILL LANE IN DALLAS.

Study No.	Signal Phasing	Study Method	Interior Traffic Movements Thru	Left	Left	Thru
1	ABC:ACB	Calculated	4.2	0.6	0.4	4.3
		Observed	3.2	1.3	1.0	3.2
2	ABC:ABC	Calculated	0.0	0.0	0.0	0.0
		Observed	0.0	0.0	0.0	0.0
3	ABC:ACB	Calculated	14.9	1.7	0.2	4.3
		Observed	8.8	3.5	2.0	2.3
4	ABC:ACB	Calculated	5.2	0.2	0.1	2.3
		Observed	5.0	0.0	0.8	1.3
5	ABC:ACB	Calculated	5.3	0.4	0.3	4.1
		Observed	4.6	0.4	0.7	3.8

lane is only six vehicles. The reason for the large difference in Study #3 in Table 2-4 is unknown, although speculation centers around the computer implementing an incorrect offset which was not recorded in the field. Overall, however, PASSER III generated consistent and reliable outputs.

Minimum Delay Studies

A number of geometric, signalization and traffic flow studies will be presented to demonstrate PASSER III program features and to illustrate the need for a thorough investigation of the performance of design and signalization options that are available. Delay performance curves were developed, similar to Figure 2-9, for 18 basic signalization problems.

Throughout the studies, the interchange external volumes shown in Figure 2-1 were used and held constant. These volumes result in exterior volume-to-capacity ratios of about 0.8. Turning movement variations were allowed within the interior of the interchange such that frontage road U-turn volumes of 50 and 150 vehicles per hour occurred. A U-turn volume in excess of 100 may be considered large (12). Three interchange spacings were selected for study such that running travel times between the two intersections would be 6, 10 and 14 seconds. This range of travel times includes most signalized diamond interchanges found in the United States (3). Relationships between spacing and travel time have previously been published (5, 12). Lastly, cycle lengths of 60, 70 and 80 seconds were analyzed. Five interchange phasing codes (1, 1A, 2, 3 and 4) were analyzed for all possible offsets in one second increments. A minimum delay was then selected for each of the 18 problem sets.

Minimum Delay Results

Minimum delay results for the 50 vehicles per hour U-turn volume problems are presented in Table 2-5. Table 2-6 contains the minimum delay results for U-turn volumes of 150 vehicles per hour. Minimum interchange delays are shown together with minimum delays for phase codes 1 and 1A. The minimum delay offsets (not shown) for phase codes 1 and 1A in all cases would provide signal phasings in the family of "four-phase with overlaps".

The minimum delay interchange phasing codes for all 18 signal problems studied are given in Table 2-7. The most important finding of this study is that every one of the possible interchange phasing codes produced a minimum delay solution in at least one of the 18 problems studied, as can be determined from Table 2-7. As the travel time increases (the distance between the intersections increases) from 6 to 14 seconds, the interchange phase code which

TABLE 2-5. MINIMUM DELAY FOR INTERCHANGE AND PHASE CODES 1 and 1A FOR 50 V.P.H. U-TURN VOLUME

Travel Time, Seconds	Cycle Length, Seconds	Minimum Interchange Delay, Sec./Veh.		
		Optimal* Phasing	Phasing Code 1	Phasing Code 1A
6	60	20.85	21.89	23.95
6	70	23.17	25.33	27.52
6	80	25.68	27.85	31.03
10	60	19.92	20.30	21.28
10	70	22.92	23.61	24.81
10	80	25.95	26.93	28.31
14	60	19.35	19.35	20.00
14	70	22.05	22.22	23.07
14	80	24.80	25.42	26.31

* See Table 2-7.

TABLE 2-6. MINIMUM DELAY FOR INTERCHANGE AND PHASE CODES 1 and 1A FOR 150 V.P.H. U-TURN VOLUME.

Travel Time, Seconds	Cycle Length, Seconds	Minimum Interchange Delay, Sec./Veh.		
		Optimal* Phasing	Phasing Code 1	Phasing Code 1A
6	60	22.75	25.32	26.65
6	70	25.02	28.87	29.97
6	80	27.71	31.81	33.75
10	60	23.93	23.54	24.10
10	70	26.99	27.24	28.04
10	80	28.72	30.85	31.94
14	60	22.57	23.40	22.82
14	70	26.28	26.28	26.28
14	80	29.35	29.35	29.83

* See Table 2-7.

TABLE 2-7. MINIMUM DELAY PHASE CODES FOR 18 INTERCHANGE SIGNALIZATION PROBLEMS

Travel Time, Seconds	Cycle Length, Seconds	Optimum Phase Codes: U-Turn Volume, 50 V.P.H.	Optimum Phase Codes: U-Turn Volume 150 V.P.H.
6	60	2,3	2,3
6	70	2,3	2,3
6	80	2,3	2,3
10	60	4	1
10	70	4	2,3
10	80	4	2,3
14	60	1	4
14	70	4	1,1A
14	80	4	1

provides the minimum delay changes.

Discussion of Results

The varying opinions expressed by traffic engineers concerning the relative merits of different diamond interchange phasing schemes seems to have been justified, if the results of this study are as descriptive of the real world as we believe them to be. For example, four-phase can be better than three-phase in some cases; whereas, in other cases three-phase is better than four-phase. However, another phasing pattern may be better than either three-phase or four-phase.

PASSER III removes the guesswork out of selecting the optimal minimum delay signal phasing pattern at a pretimed diamond interchange. A total of 6300 interchange phasing options were analyzed to find the minimum delay phasing codes shown in Table 2-7 and their respective interchange phasing patterns. This analysis was done at a total computer cost of $25 on the local university computer system running on the lowest computer job priority level. A higher priority run (8 a.m. - 5 p.m.) would have cost only $100. The PASSER III program will automatically select the optimal solution if requested and output the resulting interchange phasing pattern.

Some of the literature might be interpreted to suggest that four-phase with overlaps signalization has unusual advantages that other types do not have. It is true that four-phase with overlaps does have some good features, e.g., arterial progression, but no diamond interchange signal phasing pattern has mystical powers, not even four-phase with overlaps. Four-phase with overlaps is simply a lead-lead phasing arrangement that is timed such that perfect progression results for the front of the two arterial thru movement platoons. As the intersection spacing and travel time increases, the green times on the external movements at both intersections must be increased to maintain the perfect progression of the arterial thru movements. This increase will result in an obvious increase in external signal "capacity". Increasing the external green times is done at the expense (loss) of the green on the interior left turn phases. In the standard lead-lead phasing arrangement (code No. 1), greens are split at the two intersections in proportion to the volumes at each intersection. Increasing the spacing does not change the green split. Progression may not be as good, however. As the previous results show, it is difficult to estimate what the net effects of these features will have on total average interchange delay.

Permissive Left Turns

A number of states are using the protected left turn phase at signalized intersections (left turn arrow) followed (or led) by a permissive left turn phase (left turn legal on circular green if clear) in order to increase the capacity of high volume intersections. Texas is using the protected plus permissive left turn phasing at many high volume diamond interchanges. This type of control effectively provides some left turn capacity on the arterial thru phase, phase A in Figure 2-2. This type of signalization may completely change the preferred signal phasing patterns engineers are accustomed to and may also change the minimum delay interchange phasing patterns for a given signalization problem (Table 2-7).

The PASSER III computer program has the capability of analyzing the protected plus permissive phasing concept in either the leading or lagging phase sequence. The effects of opposing queues and traffic flow are considered. A mathematical model of this process has been developed and validated by TTI from data collected in several Texas cities during 1974-75 (13).

An example of the reduction in overall interchange delay that would occur if a permissive left turn phase is added to phase A on both ramp (frontage road) intersections can be determined from Figure 2-11. In this case, an overall reduction in delay of approximately 2 seconds per vehicle (8% reduction) would result. A much higher reduction in delay to the interior left turn vehicles, where the capacity is increased, occurs together with a reduction in maximum queue lengths.

Summary of Delay Optimization

The results of this study show that the optimal minimum delay, pretimed diamond interchange signal phasing pattern can be determined using PASSER III. While signalization guidelines and preferred signal phasing patterns are helpful, their utility is limited and performance uncertain. A detailed analysis of all pretimed signalization options can now be performed efficiently.

Interchange Design Applications

PASSER III can be used in several ways to assist in the evaluation of various signalized diamond interchange design alternatives. Basic design options such as: (1) number of exterior and interior approach lanes, (2)

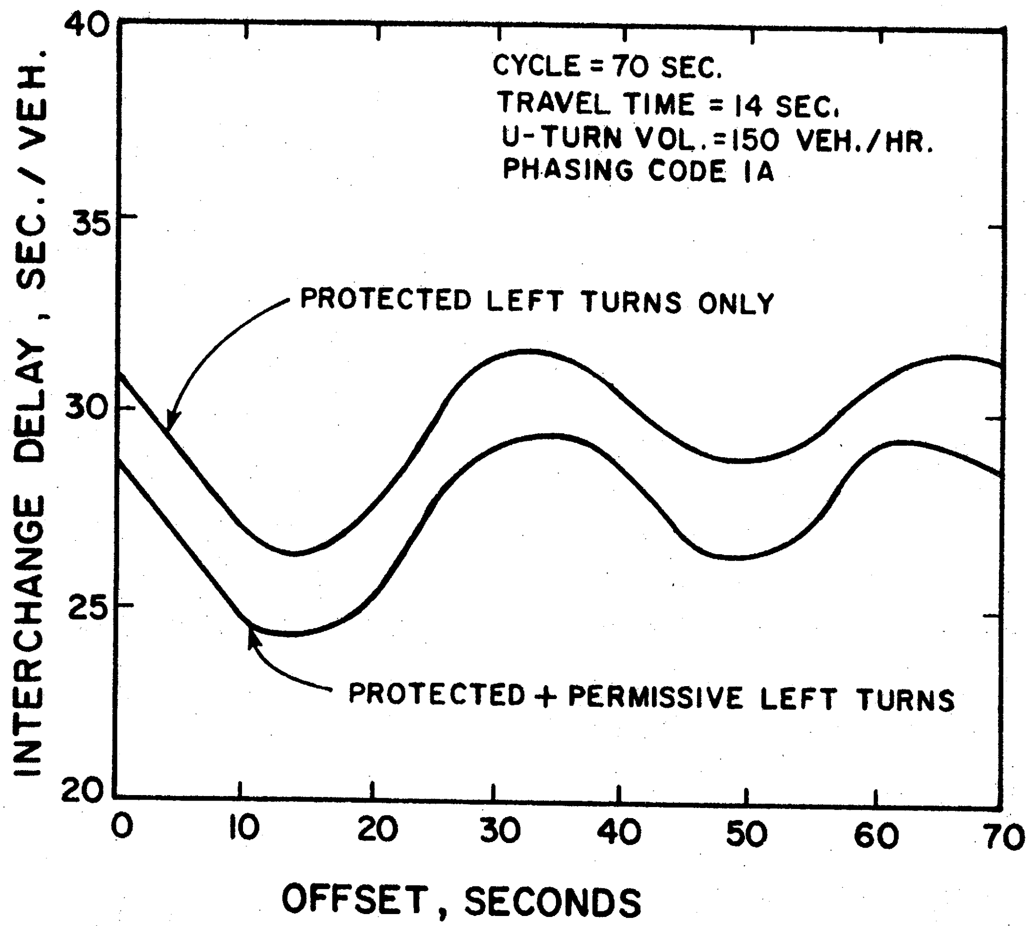


Figure 2-11. Reduction in Interchange Delay Due to Addition of Permissive Left Turns

separation distance between the intersections, and (3) storage length of the interior left turn bays can be evaluated under selected or optimal signalization patterns. PASSER III calculates several operational performance measures including: (1) a measure of overall interchange performance in terms of average delay per vehicle, (2) three individual movement measures of effectiveness (volume-to-signal capacity ratio, delay, and probability of clearing queues), and (3) ratios of maximum queue observed per cycle for each of the four interior movements (left turns and thrus) divided by their respective vehicular storage capacities as determined from the length of storage provided by the design.

Number of Lanes

Adding additional basic through lanes to the interchange might be considered if the operating conditions provided by the initial design were unsatisfactory. Similar considerations might be given to frontage road (ramp) approaches where basic frontage road capacity is deficient. Less expensive options might be considered for selected high-volume turning movements such as the provision of exclusive left turn, right turn, or U-turn lanes. The PASSER III program (2) defines a capacity for each of 18 traffic movements at the interchange in terms of the effective number of lanes of capacity provided each movement. A value of 1.0 means that 1800 vehicles per hour of green time could use the signal serving the movement under consideration.

Separation Distance

The separation distance is the center-to-center distance between the two frontage road (ramp) intersections of the interchange. In general, this distance is primarily determined by the freeway design, the type of surface-street crossing selected (over crossing or under crossing), and right-of-way considerations. However, the separation distance also affects operational performance at the interchange and some variation in separation distance may be desirable and cost-effective.

There are several situations where increasing the separation distance may be desirable. It is apparent that increasing the ramp-to-ramp separation distance would increase the storage space provided the four interior movements. This may be required or desirable, as will be discussed later. Increasing the separation distance may improve the quality of signal progression and/or capacity. A modest increase in spacing may result in a considerable improvement in the level of service at the interchange. However, it is possible that reducing the separation distance slightly from some given spacing may improve performance.

The separation distance affects the performance of signalization in part through the travel time required to start from one intersection after green is displayed and travel to the downstream intersection. Other factors affecting the required travel time include vehicle acceleration rate and roadway grade. Travel time data may be collected at local interchanges to determine these relationships. Data collected several years ago in Houston (12) are presented in Table 2-8 for illustration and reference. Using these data in Table 2-8 for reference, it can be seen that Table 2-5 was essentially an evaluation of three interchange design alternatives having intersection separation distances of 67, 200 and 376 feet which provided travel times of 6, 10 and 14 seconds, respectively.

Interior Storage

The amount of storage required for the two interior left turn and through movements depends on the movement volume, the traffic pattern, the green time available per cycle, the cycle length, the type of signalization used (See Table 2-1.), and on the signal offset between the two intersections. An illustration of the maximum queue occurring per cycle for two interior traffic movements as a function of offset was presented in Figure 2-7. PASSER III can provide printer plots similar to Figure 2-7 upon request (at the district terminals). However, ratios of maximum queue per cycle for the minimum interchange delay (or maximum progression) signal phasing pattern are routinely provided in the output data if the delay-offset routine is used.

For example, assume that the minimum delay phasing pattern was Code No. 1 with an offset of 10 seconds in Figure 2-7. The maximum left turn queue per cycle for the movement shown would be 4.0 vehicles. Assume the design provides storage capacity for 8 vehicles. The calculated storage ratio for this case would be 0.5 (4 in queue/8 spaces). Storage ratios should not exceed 0.8 with 0.6 being a more desirable maximum. Storage capacity, S, is calculated from

$$S = \frac{L \cdot P}{25\ (1 + T)} \qquad (2\text{-}19)$$

where

S = storage capacity of lane, vehicles

L = length of available storage, feet

TABLE 2-8. INTERCHANGE INTERIOR TRAVEL TIME AND OVERLAP AS A FUNCTION OF SEPARATION DISTANCE

Distance (feet)	Travel Time[a] (Seconds)	Overlap[b] (Seconds)
67	6	4
94	7	5
125	8	6
160	9	7
200	10	8
244	11	9
288	12	10
322	13	11
376	14	12
420	15	13

[a] Travel time of through vehicle from stop line on exterior approach to interior stop line at downstream intersection. Calculated from $T = 0.5 + (0.45 \cdot d)^{0.5}$; 30 mph maximum speed.

[b] Used primarily with ICODE = 5; four-phase overlap signalization. Provides a 2.0 second advance green.

P = decimal fraction of traffic in lane making maneuver.

T = decimal fraction of trucks and buses in flow.

If more than one lane is available for the movement (usually for the through movement), the storage provided by each lane must be added together to determine the total storage capacity. It is possible for P (in Equation 2-19) to be 1.00 for one lane and some other value (e.g., 0.56) in an adjacent lane, particularly for through storage capacity when no left turn bay is provided.

Various alternatives of interior geometrics of a diamond interchange are presented schematically in Figure 2-12 which provide generally increasing storage capacities from the upper to lower figures. No attempt has been made to illustrate optional right turn or U-turn lanes. The upper figure illustrates a basic design with no left turn bays. Storage capacity for the interior left turn lane depends on the volume distribution on the inside lane. No protected storage is provided; potential for rear-end and side-swipe accidents is high, and signal capacity is reduced. Some improvement can be obtained if additional through lanes are provided between the two intersections.

The second figure illustrates the provision of left turn bays by using some form of the S-type median design. Signal capacity and safety are increased considerably if the storage length is such that acceptable storage ratios result.

A note of caution is offered with regard to either of the upper two design alternatives. Care should be exercised to insure that adequate turning paths are provided for frontage road (ramp) traffic wishing to make a left turn into the interchange. The geometry of the center island channelization (and stop line) together with interior through lane widths should be such that Single-Unit (SU) vehicles turning two abreast can safely make the left turn maneuver. Unless this turn can be made easily, a high variation in lane volumes will occur on frontage road approaches resulting in a loss in capacity and an increase in interchange delay. For example, the interchange of I-35 with U.S. 290 in Austin does not provide sufficient turning paths for frontage road left turns. Left turns are made from two lanes. The second lane provides both left turns and frontage road through vehicles. Yet during five studies made on this approach, more vehicles turned left from the second frontage road approach lane than from the exclusive inside lane even though a large number of through frontage road vehicles were also using the second lane.

Returning to Figure 2-12, the lower two designs provide increased storage for interior left turns. The lower figure also provides additional exterior

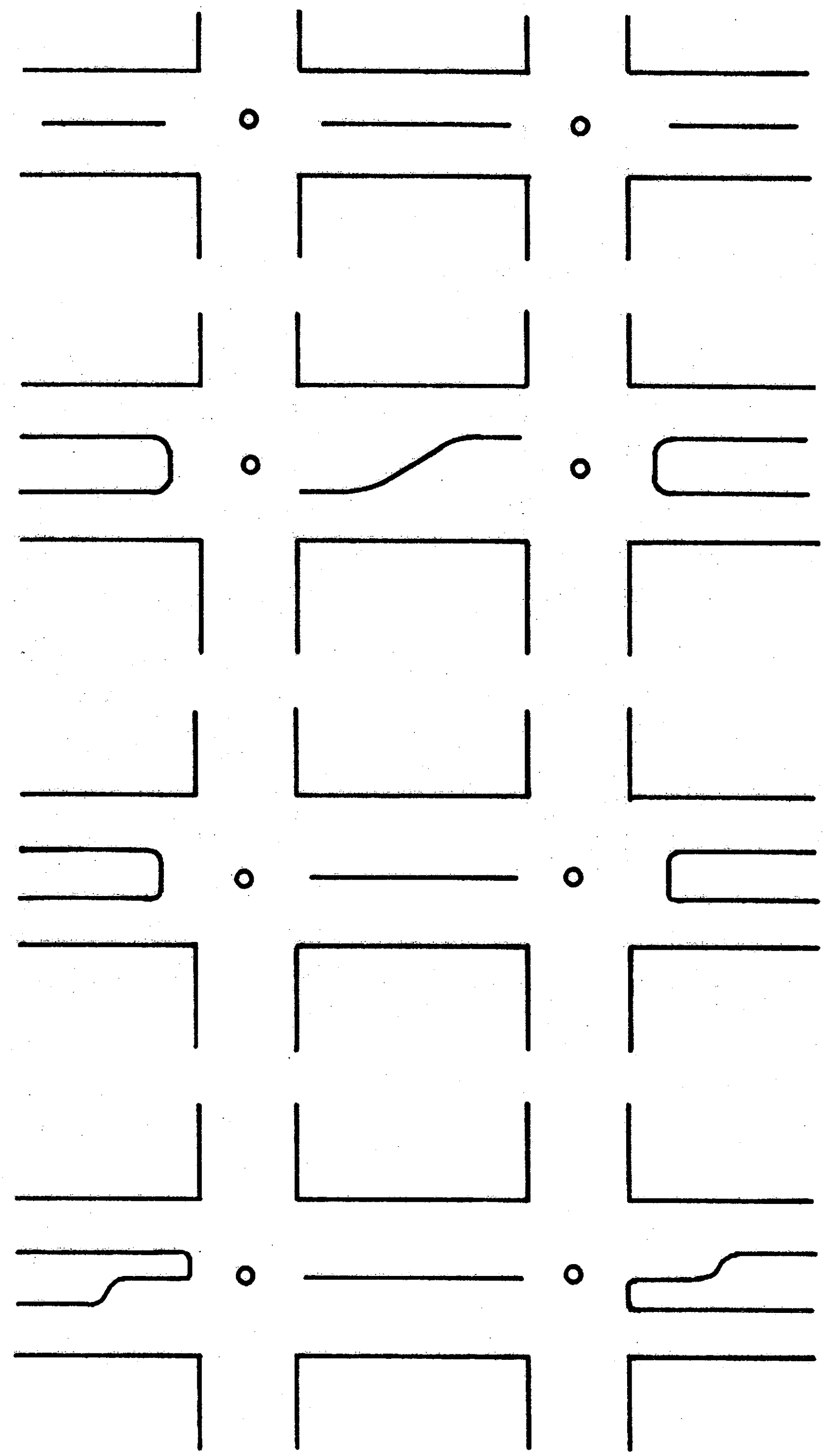


Figure 2-12. Design Alternatives For Left Turn Storage In Interior of Diamond Interchange.

left-turn storage capacity. However, some potential for wrong-way turning onto the frontage road may exist on the exterior arterial approaches.

Frontage Road Progression

In order to improve the quality of traffic flow along continuous one-way frontage roads, the frontage road signal phases at the diamond interchanges may be coordinated to achieve progressive traffic flow. Progression can be readily obtained in one direction by manual analysis methods.

Messer et al. (14) have previously developed the theory for optimizing progression simultaneously on both frontage roads. This optimization theory has been incorporated into PASSER III and a user's manual has been written to assist the traffic engineer in using the program effectively (2).

Assume four signalized diamond interchanges are connected by continuous, one-way frontage roads as illustrated in Figure 2-13. Also assume optimal progression is desired simultaneously on both frontage roads at a 60-second cycle length. Assume the distance between interchanges 1 through 4 (Figure 2-13) are 1,200, 1,800, and 600 feet, respectively. All speeds on the frontage roads arbitrarily are assumed to be 40 feet per second (27 mph).

The quality of frontage road progression depends on several variables. One of the more important variables is the type of diamond interchange signal pattern operating at each interchange. One pattern might result in good progression; whereas, another pattern might provide no progression at all. It is apparent that the more patterns which PASSER III can consider, the better the progression would likely be. PASSER III, like PASSER II, can analyze up to four signal patterns at each interchange.

Two methods are available in PASSER III to input the allowable signal patterns (2). One method is direct data entry. This method inputs the desired interchange phase code and offset (See Figure 2-4) to define each pattern permitted to be analyzed for progression. PASSER III will select a pattern. This pattern will be the one that maximizes the progression. When using this method, thoughtful consideration is required to define signal patterns that will: (1) promote good progression possibilities and (2) provide minimal interchange delay at the same time, as described earlier in this chapter.

The second method available in PASSER III for inputing allowable interchange phasing patterns is to request PASSER III to determine the four best patterns based on total interchange delay using the delay-offset technique. This analysis and selection process is done automatically by the program and

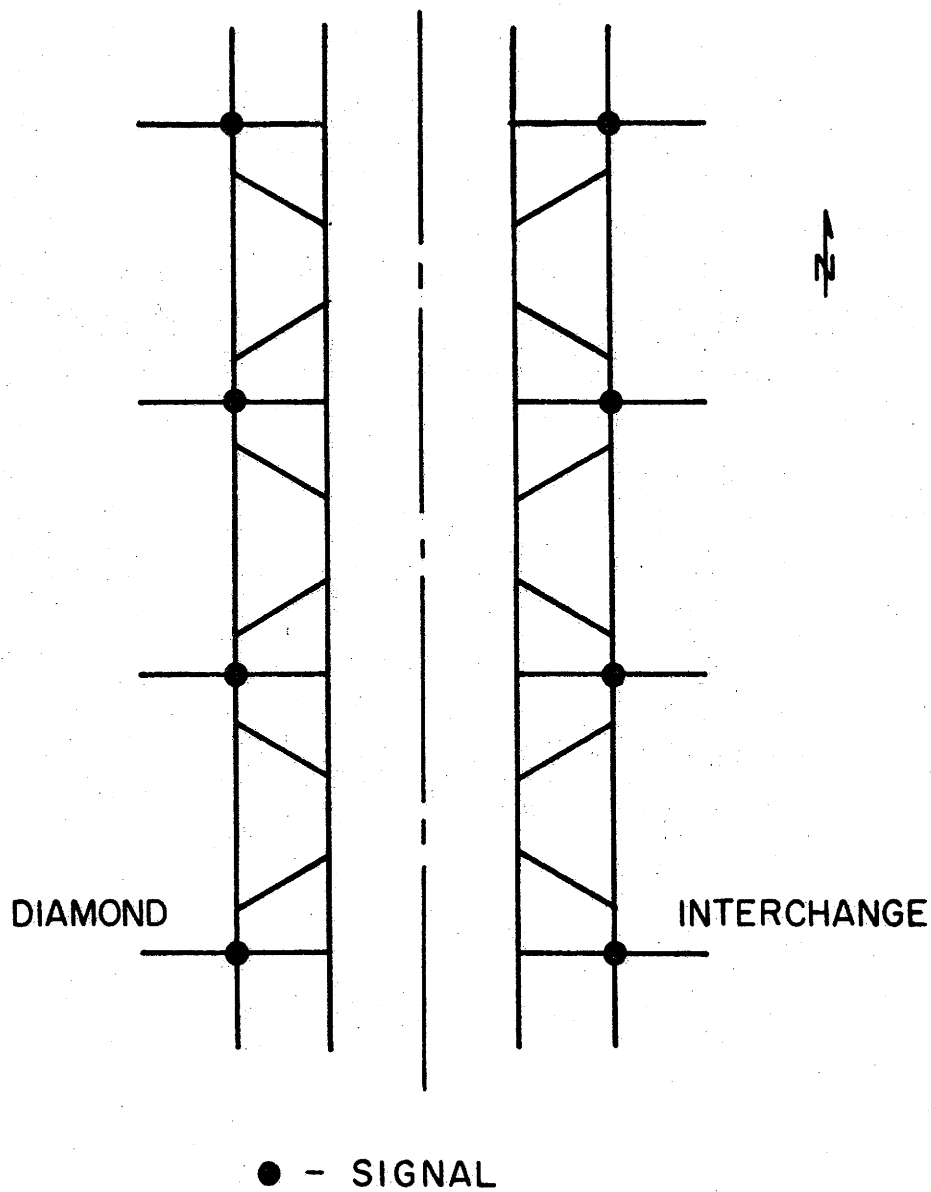


Figure 2-13. A Frontage Road Progression Analysis and Control Area

requires a minimum of data input coding. Basically, it is only necessary to define which of the phase codes (Table 2-1) are to be evaluated by the delay-offset technique. Computer costs will increase as the number of options considered increases.

Continuing with the example problem, assume that the first method described was used to input the allowable signal patterns to PASSER III. Only two phase patterns were permitted at each of the four interchanges: (1) ABC:ABC with a 7-second offset and (2) ACB:ABC with 0 second offset. These are shown in Figure 2-14. The frontage road progression green phases (phase B) are also emphasized.

The optimum frontage road progression time-space diagram is presented in Figure 2-15. The blocks represent the location of the two progressive green phases (phase B) in time for each interchange. For this example problem, the ABC:ABC phase sequence with a 7 second offset was the optimal progression signal pattern only at interchange No. 3. The ACB:ABC sequence with a 0 second offset was selected by PASSER III at the other three interchanges.

PASSER III outputs all timing information necessary for evaluation or implementation (2), including green times, offsets, level of service analysis, time-space diagrams, and phase interval timings. It should be noted that the delay-offset analysis of one or more interchanges can be analyzed without conducting a progression analysis; the converse is also true.

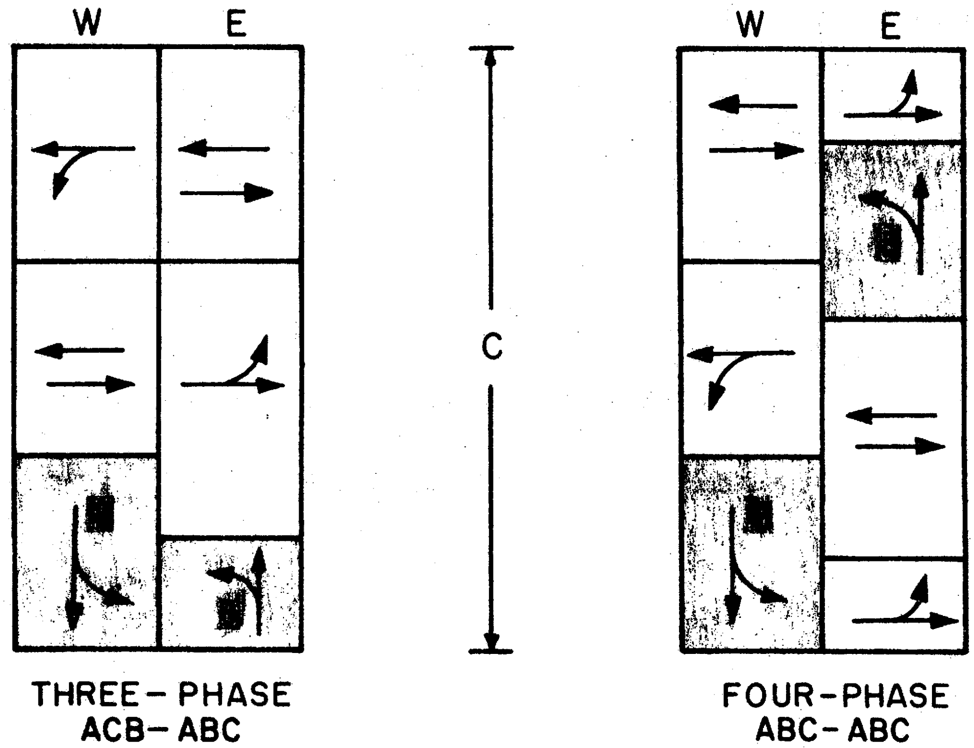


Figure 2-14. Locations of Frontage Road Progressive Movements in 3- and 4-Phase Sequences

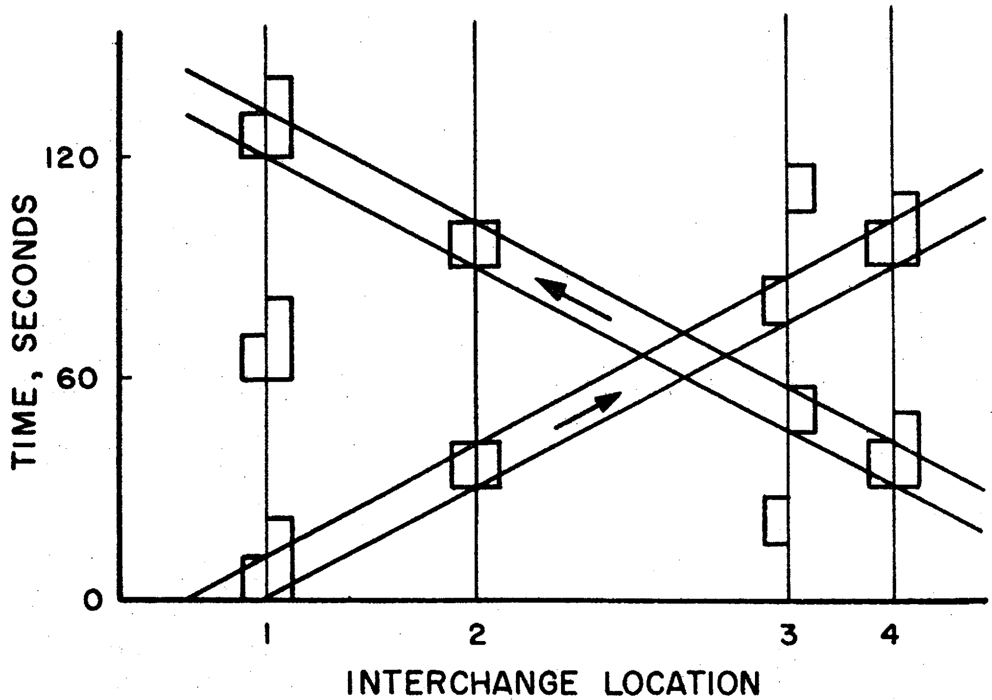


Figure 2-15. Optimal Frontage Roads Progression Solution

REFERENCES

1. Messer, C.J., Haenel, H.E. and Koeppe, E.A. A Report on the User's Manual for Progression Analysis and Signal System Evaluation Routine--PASSER II. Texas Transportation Institute Research Report 165-14, August, 1974.
2. Fambro, D.B., Messer, C.J. and Koeppe, E.A. A Description of the User's Manual for Diamond Interchange Signalization--PASSER III. Texas Transportation Institute Research Report 178-1, August, 1976.
3. Munjal, P.K. An Analysis of Diamond Interchange Signalization. Highway Research Record 349, 1971, pp. 47-64.
4. Munjal, P.K. and Fitzgerald, J.W. A Simulation Model for the Evaluation of Real-Time Computer Control of Diamond Interchanges. Systems Development Corporation Document TM - 4601/004/00, 1971.
5. Messer, C.J. and Berry, D.J. Effects of Design Alternatives on Quality of Service at Signalized Diamond Interchanges. Transportation Research Record 538, 1975, pp. 20-31.
6. Webster, F.V. Traffic Signal Settings. Her Majesty's Stationery Office, London, Road Research Technical Paper 39, 1958.
7. Wagner, F.A., Gerlough, D.L. and Barnes, F.C. Improved Criteria for Traffic Signal Systems on Urban Arterials. NCHRP 73, 1969.
8. Wagner, F.A., Barnes, F.C. and Gerlough, D.L. Improved Criteria for Traffic Signal Systems in Urban Networks. NCHRP 124, 1971.
9. Gartner, N. Microscopic Analysis of Traffic Flow Patterns for Minimizing Delay on Signal-Controlled Links. Highway Research Record 445, 1973, pp. 12-23.
10. Rach, L., Lam, J.K., Kaufman, D.C. and Richardson, D.B. Evaluation of Off-Line Traffic-Signal Optimization Techniques. Transportation Research Record 538, 1975, pp. 48-58.
11. Spitz, S. Signalization of Diamond Interchanges. Proceedings of Western Section Institute of Traffic Engineers, Anaheim, 1963.
12. Woods, D.L. Limitations of Phase Overlap Signalization for Two-Level Diamond Interchanges, Traffic Engineering, September, 1969.
13. Messer, C.J., Fambro, D.B. and Andersen, D.A. A Study of the Effects of Design and Operational Performance of Signal Systems. Texas Transportation Institute Research Report 203-2F, August, 1975.

14. Messer, C.J., Whitson, R.H. and Carvell, J.D. A Real-Time Frontage Road Progression Analysis and Control Strategy. Transportation Research Record 503, 1973, pp. 1-12.

CHAPTER 3

SIGNAL TRANSITION ANALYSIS

Introduction

An increasing number of traffic signal systems are being brought under computer control. Although the designs of these control systems vary in detail, nearly all can be described as a sequence of selected signal patterns. One of the primary advantages of computer control is the ability to change signal patterns in response to measured variations in traffic conditions. Associated with each pattern change is a phase transition period wherein the signal settings transform from one pretimed pattern to the next. Operational experience has shown that these transition periods can have disruptive effects on traffic flow.

Types of Transitions

In general, transition policies may be classified as (a) a smooth, staged transition that restricts the change in offset per cycle or (b) a procedure designed to minimize delay (1). The first is smooth from the viewpoint of the control system; such a policy is frequently used but few useful guidelines are available to aid in implementation which would avoid large delays. The second transition procedure is intuitively appealing and could well produce good results but the high level of technical complexity reduces its utility. The UTCS-2-GT and RAST algorithms are illustrative of these complex methods (1). The following transition models are presented to illustrate the two types of transition procedures.

Toronto Model (2)

In the signal transition procedure in use in the Toronto system, the old timing pattern is transformed to the new pattern immediately. The procedure involves the adjustment of both the major street and minor street green aspects, subject to a maximum allowable change, until the new offset pattern is achieved. The transition could take as long as three signal cycles, and the disruption to the traffic during the transition period may have a lasting effect on the signal system. This is, in fact, the major problem with the Toronto offset transition model, especially in cases where frequent control plan changes are made.

For a given signal, the amount of offset adjustment necessary in the transition process is determined in the following manner:

1. The difference (D) between the existing and new offset numbers for

intersection i is calculated by the equation:

$$D = a_i - b_i + k_i C_n$$

subject to the following constraint: $-\frac{C_n}{2} < D < \frac{C_n}{2}$

where a_i = old offset for intersection i with respect to the new cycle

b_i = new offset

C_n = new cycle length

k_i = an integer chosen to maintain the periodicity condition

2. The clock difference (P) between the existing and new cycle lengths expressed as

$$P = L - M$$

subject to the following constraint: $-C_n < P < C_n$

where L = existing cycle length clock time

M = new cycle length clock time

C_n = new cycle length

3. The offset adjustment (E) for intersection i is then given by:

$$E_i = D - P$$

subject to the following constraint: $-\frac{C_n}{2} < E_i < \frac{C_n}{2}$

P.M.M. Model (3)

Another offset transition method, developed for computerized traffic control systems by Peat, Marwick and Mitchell, is based on the fact that offsets are always maintained with respect to a master reference point, and a constant can always be added to a set of offsets without changing the offset relationship between signals. In changing from one set of offsets to another, there is an optimum value of this constant which when added to each of the new offsets will minimize the average length of time for signal transition. An optimum value of the offset transition constant can be found by minimizing the sum of the squares of the individual transition times. This places an extra penalty on the longer transition times. The function to be minimized is given by:

$$Y = \sum_{i=i}^{N} (a_i - b_i - X + k_i C)^2$$

where Y = sum of the squares of the individual offset adjustments

a_i = old offset for intersection i with respect to the new cycle length

b_i = new offset for intersection i

C = new cycle length

k_i = integer chosen to maintain the periodicity condition

X = the offset transition parameter which is to be found subject to the following constraint:

$$-\frac{C}{2} < a_i - b_i - X + k_i C < \frac{C}{2}$$

N = number of signals in the network

<u>Transition Simulation Analysis</u>

The scope of this research effort was directed toward evaluating the effects on delay caused by two basic types of "smoothed" transition procedures: (1) red time added to the affected phase (usually the cross street has the red time added to it while the main street has the same amount of green time added) while the new progression offset is being established and (2) green time subtracted from the affected phase (short-way offset implementation; could be on main or cross street). In either case, vehicular delay is increased on the affected phase. The Toronto model assumes this offset adjustment (E_i) can range from $-\frac{C}{2}$ to $+\frac{C}{2}$. The following study results will show that significant differences in delay do occur and that the above allowable range could be improved. A previously developed signalized intersection simulation program (4) was used to study the effects of various transitions on delay. The following study variables were considered:

Type of Transition	Number of Transition Cycles	Signal Saturation Ratios, qC/gs
Add	1	0.4
Subtract	2	0.6
	4	0.8

The results of these simulation studies of delay experienced with varying percentages of the cycle used to achieve the offset shift are presented in Figures 3-1, 2 and 3. An add offset adjustment (E_a) is equal to the percent offset shift multiplied by the cycle length; $0 \leq E_a < C$. A subtract offset adjustment (E_s) is equal to $C - E_a$.

The results of the simulation study and previous research were studied and evaluated. From these sources, the following operational guidelines for implementing offset transitions were developed.

Offset Transition Guidelines

1. The range of add offset adjustments, E_a, probably should not exceed 70% of the cycle length, if possible. Subtract offset adjustments, E_s, probably should not exceed 30% of the cycle length. It is usually better to lengthen the cycle length to achieve the desired new offset rather than shorten it for both vehicular and pedestrian reasons. Short phase green times may result if the subtraction method (shorter cycle) is used. Minimum phase green times should not be violated due to the transition process.
2. Using more than one cycle of transition time to produce the new offset should be considered in the following cases: (a) anytime the subtraction method is used, particularly if the minimum green time would otherwise be violated; (b) where the addition method might result in an excessively long queue forming on an approach and overflow into an upstream signalized intersection if only one cycle of transition were used; (c) where experience indicates that more than one transition cycle is beneficial.

   The simulation results show that some reduction in delay occurs at an (isolated) intersection if green time and/or cycle length adjustments are smoothed out. It is rational to propose that the add offset adjustment should not exceed 25% C per cycle, and that the subtract offset adjustment should not exceed 10% C per cycle. Thus, the maximum transition period would not exceed three cycles if the "short-way" guideline, guideline No. 1, were followed. If the all-add method were used, the transition period might last four cycles.

   However, other factors should also be considered. The longer the transition period, the longer operational conditions along the arterial are changing and disturbed. More cycles of control are floating and unstable with the quality of progressive flow provided being unknown.

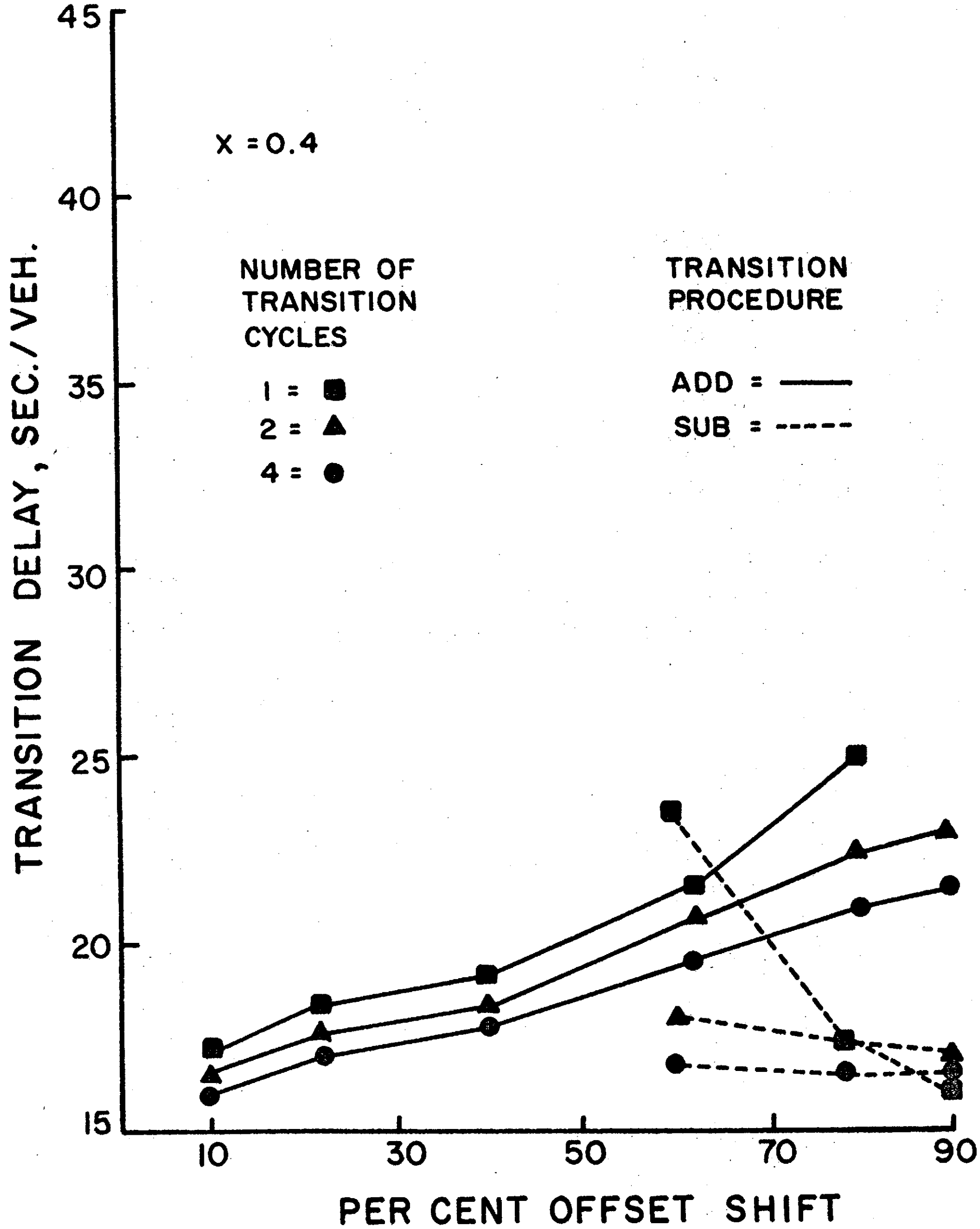


Figure 3-1. Delay for Various Per Cent Offset Shifts and Number of Cycles for Saturation Ratio of 0.4

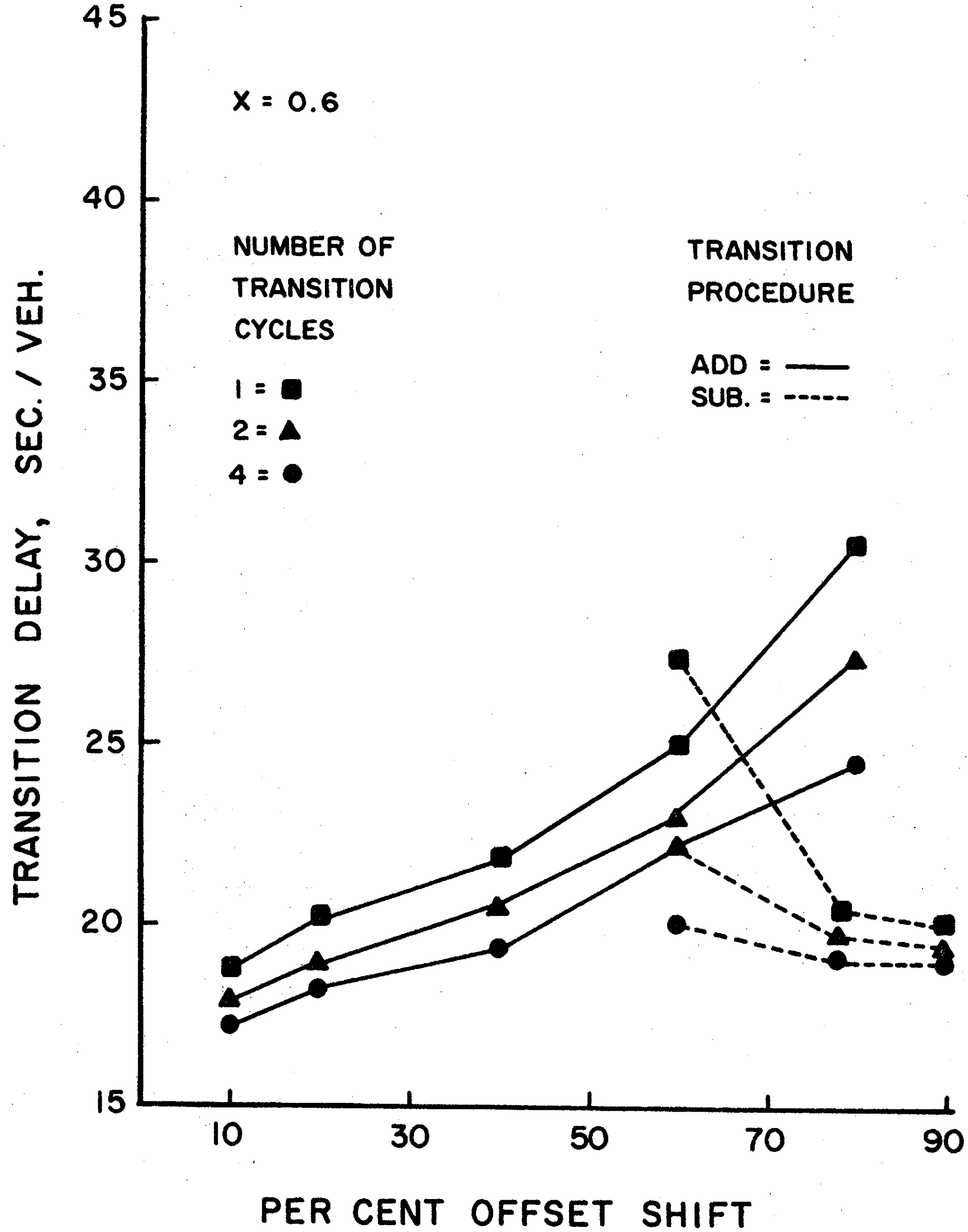


Figure 3-2. Delay for Various Per Cent Offset Shifts and Number of Cycles for Saturation Ratio of 0.6

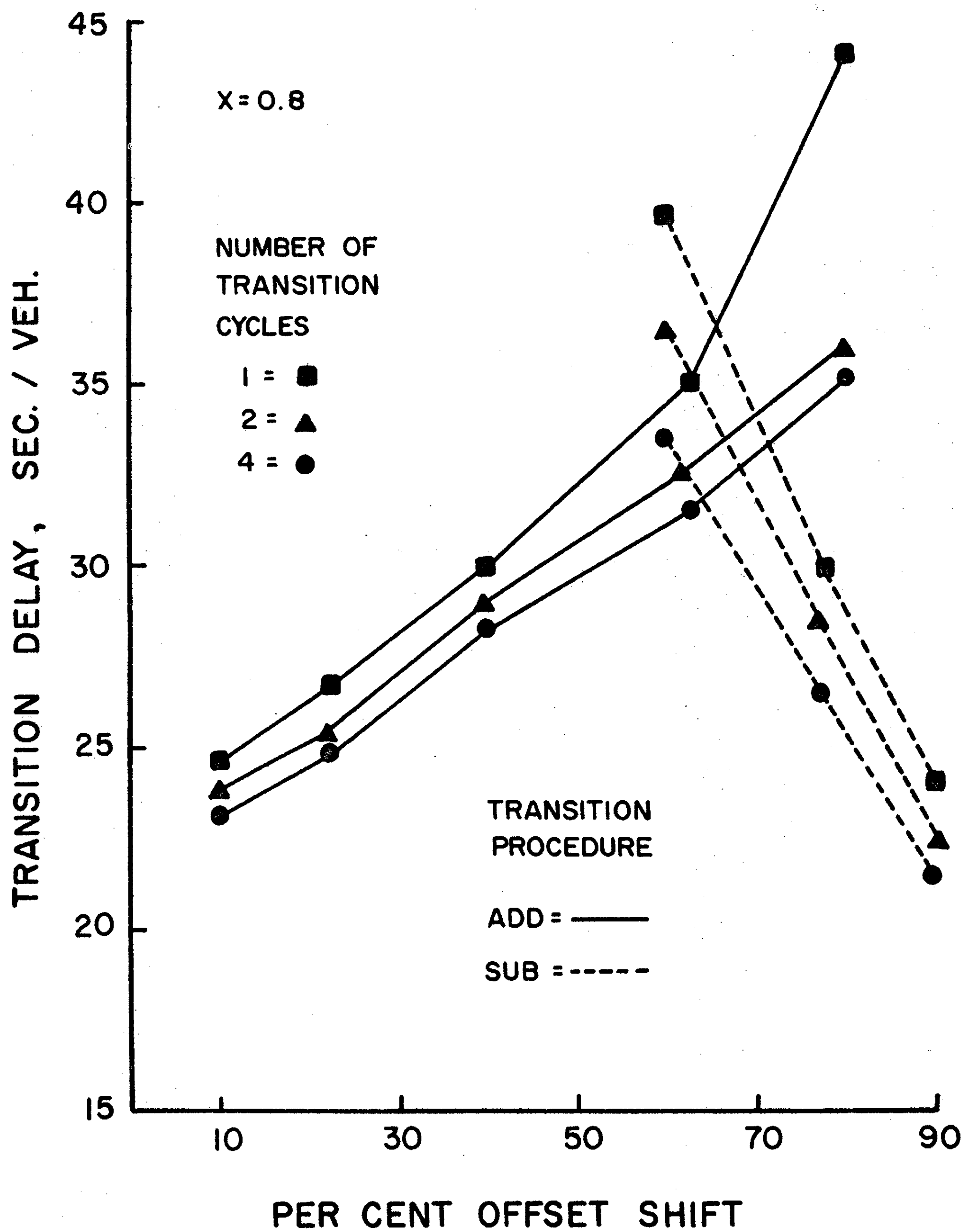


Figure 3-3. Delay for Various Per Cent Offset Shifts and Number of Cycles for Saturation Ratio of 0.8

It does not necessarily follow that more transition cycles mean less total disturbance to traffic using the arterial.

3. When possible, offset adjustments (either add or subtract) should be implemented on the street which has the critical (maximum) volume-to-signal capacity ratio (qC/gs). In addition, add offset adjustments (E_a) should be added to the through movement green phase, whereas subtract offset adjustments (E_s) should be subtracted from the red time of the signal. The critical street may be selected, to a high degree of accuracy, as being the street with the highest volume per lane. The main street usually would be the street selected. This guideline requires that volume data be available regarding current traffic flow conditions at the intersection.

These same basic guidelines may be considered to apply at signalized diamond interchanges. Several years of computer control of diamond interchanges in Dallas by TTI personnel indicate that motorists are much more adaptable and responsive to changes in signal patterns than they were intially thought to be. Project personnel are not aware of any traffic accidents that might have been related to signal pattern transitions. Care should be taken, however, to avoid skipping a green phase during high volume conditions as motorists may think the controller has failed and be tempted perhaps to run a red traffic signal.

REFERENCES

1. Lieberman, E.B. and Wicks, D. A Rapid Signal Transition Algorithm. Transportation Research Record 509, 1974, pp. 1-15.
2. McColm, T.H. Evaluation of Offset Transition Procedures. Toronto, Canada, Traffic Control Centre Internal Memorandum, February, 1973.
3. Kates, A.J. Offset Transition Procedure. Peat, Marwick, Mitchell and Co.
4. Messer, C.J., Fambro, D.B. and Andersen, D.A. A Study of the Effects of Design and Operational Performance of Signal Systems. Texas Transportation Institute Research Report 203-2F, August, 1975.

CHAPTER 4

TECHNIQUES FOR PREDICTING TRAVEL TIMES IN THE URBAN FREEWAY CORRIDOR

Introduction

Freeway ramp control systems have proved their effectiveness in relieving freeway congestion when operations are free of incidents. Incident conditions, however, are a frequently occurring phenomenon on urban freeways. Goolsby found that, within a 6-mile section on the Gulf Freeway in Houston (1), more than 13 lane-blocking incidents occur on the average during the time period of 6 a.m. to 7 p.m. from Monday through Friday. Stalled vehicles and accidents were the contributing causes of 97 percent of the incidents observed. Approximately 80 percent of the incidents reduced the capacity of the freeway in one direction by about one-half or more. Table 4-1 presents the incident characteristics observed on the Gulf Freeway and Table 4-2 provides observed incident capacities.

Freeway operational improvements have been implemented or proposed to improve the level of service during incidents. Several of these systems have consisted of some form of variable-message signs (2-6). One of the chief operational objectives of these signs is to increase the effective capacity of the freeway corridor during incidents on the freeway by achieving a higher utilization of the adjacent frontage road and surface street system. Driver preference questionnaire studies indicate that drivers will divert around congestion if accurate, reliable, and timely traffic information is provided to them. This diversion could occur from the freeway, at the frontage roads, or at major intersections located within the freeway corridor (7). One measure of the likelihood and desirability of diversion is the travel time saving that may occur to motorists if they are diverted (7, 8). This evaluation requires an estimate of the travel times along the freeway and along the alternate route during the incident conditions.

Variable Input-Output Freeway Travel Time Model

This section presents a method for predicting a motorist's travel time from selected freeway locations to the end of the freeway system during incident conditions. It is predictive in that it computes an estimate of what a motorist's travel time would be at some future time after the incident has occurred.

TABLE 4-1. CHARACTERISTICS OF LANE BLOCKAGE BY STALLED VEHICLES AND ACCIDENTS ON GULF FREEWAY (1)

Lane Blocked	Stalls Number	Stalls Percent	Accidents Number	Accidents Percent
One Lane		86.2		63.5
Outside	432		244	
Center	231		204	
Median	299		284	
Two Lanes	8	0.7	111	9.6
Three Lanes	0	0.0	22	1.9
Ramps	134	12.0	238	20.6
Other	13	1.1	51	4.4
Total	1117	100.0	1154	100.0

TABLE 4-2. CAPACITY OF INBOUND GULF FREEWAY DURING DIFFERENT INCIDENT CONDITIONS (1)

Condition	Number of Incidents	Sample Size (No. Min.)	Average Flow Rate (Veh/Hr)
Normal Flow		312	5560
Stall (one lane blocked)	4	43	2880
Non-Injury Accident (one lane blocked)	17	167	2750
Accident (two lanes blocked)	6	53	1150
Accident on Shoulder	23	254	4030

Members of the research team have previously developed a freeway travel time prediction model during freeway incident conditions (9). This model and related operational computer program (10) are rather complicated yet solve only a limited case solution: that of a freeway section having uniform input flow and only one reduced capacity incident output flow rate. A more practical solution approach was needed in addition to one that would analyze variable input and output flow rates.

Example Problem

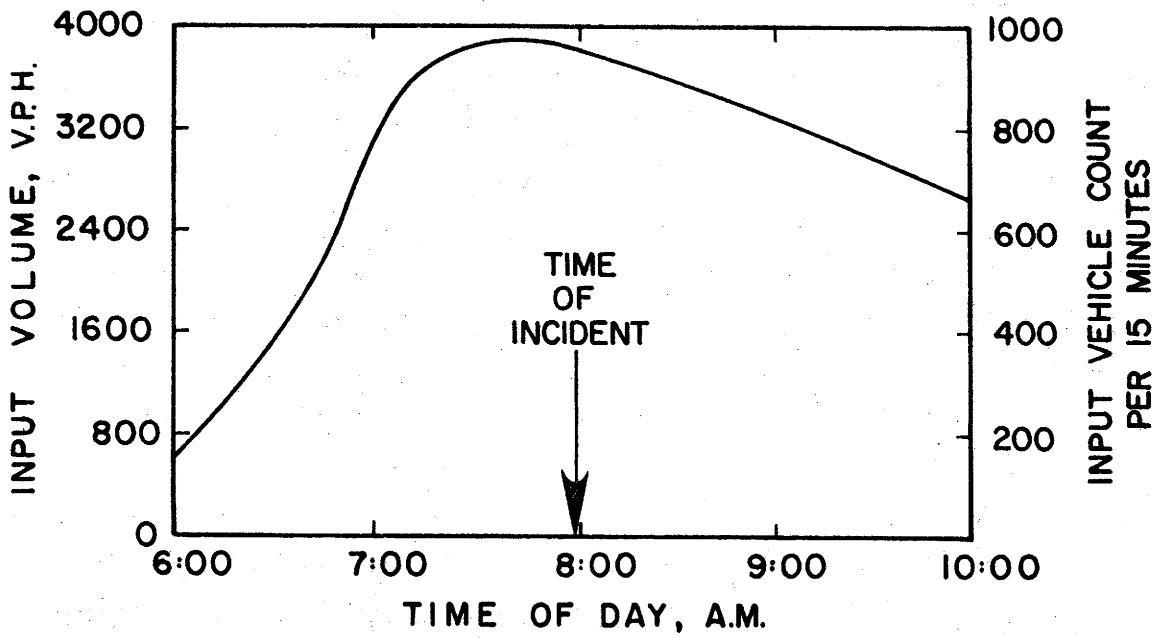
The approach is best illustrated using a typical example problem. Suppose travel time and delay estimates are desired along a 4.4-mile inbound section of a 4-lane urban freeway in Texas. A "typical" incident is assumed to have occurred 3.0 miles downstream at 8 a.m. Normal operating speeds along the section are assumed to average 30 miles per hour at 8 a.m. The incident is assumed to block one of the two inbound lanes; the result is a reduction in maximum possible (capacity) freeway flow at the scene of the incident from a normal capacity flow of 4000 vehicles per hour (VPH) to 1500 VPH during the duration of the incident. The normal 15-minute flow rates (expressed in equivalent hourly volumes) on the freeway at (near) the scene of the incident from 6:00 a.m. to 9:00 a.m. are depicted at the top of Figure 4-1. These "variable input" demand volume counts starting at 8 a.m. will serve as a data source to the variable input-output travel time model.

Approach

The basic approach selected was to mathematically model estimated demand-capacity (input-output) queueing functions as suggested by Wattleworth et al. (11). The delay incurred by a motorist $\{D(t)\}$ is the difference between the cumulative input $\{\Sigma I(t)\}$ and cumulative output $\{\Sigma O(t^*)\}$ functions, as illustrated in the lower drawing of Figure 4-1. Mathematical programming techniques were used to develop polynomial equations for the variable input and output functions based on estimates of input volume demand, as determined from traffic counts, and incident blockage characteristics.

Cumulative Input

Using the input demand volumes presented at the top of Figure 4-1, cumulative data points are calculated starting at 8:00 a.m. for subsequent 15-minute periods until 9:00 a.m. as presented in Table 4-3.



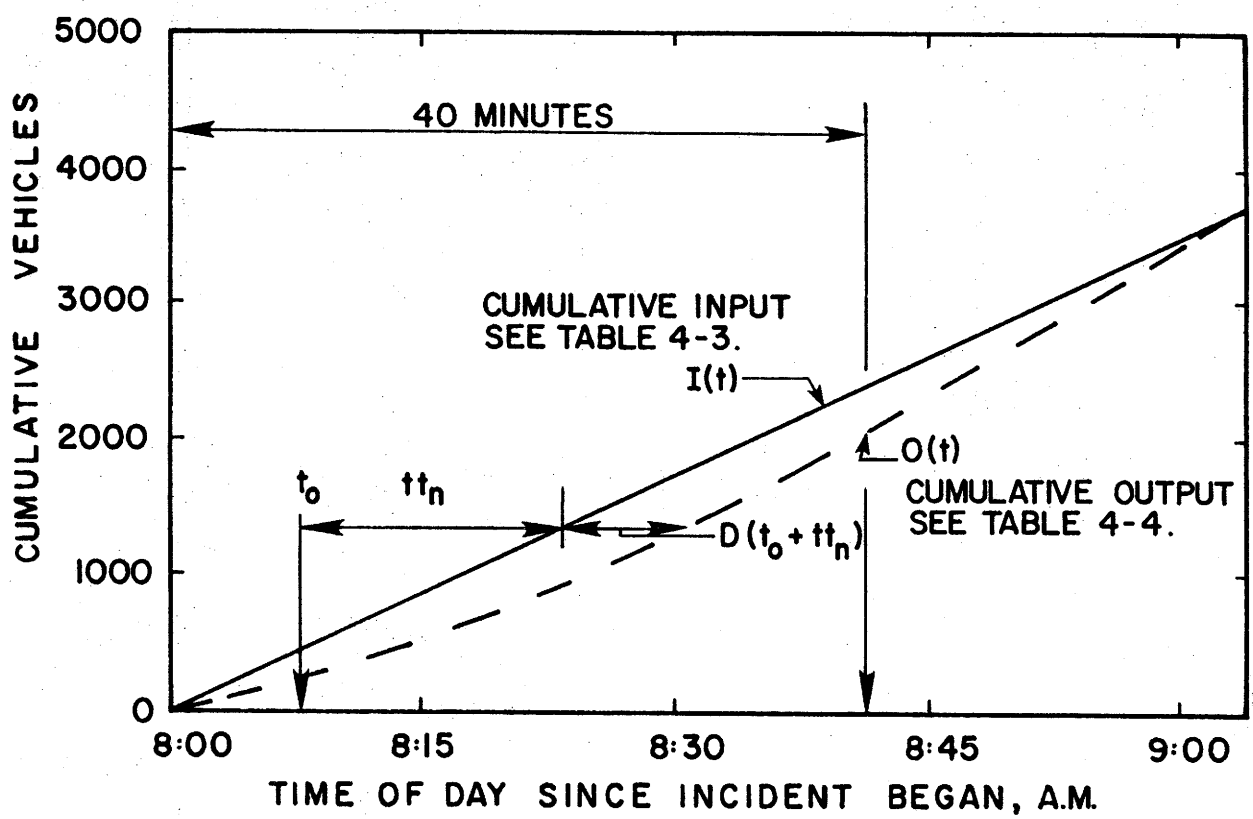


Figure 4-1. Method For Calculating Delay For Incident on Freeway With Given Input Volume

TABLE 4-3. CUMULATIVE INPUT DATA FOR EXAMPLE PROBLEM

Ending Time, t A.M.	Time Since Incident Occurred Min., (hrs.)	15 Minute Normal Vehicle Count	I(t), Cumulative Number of Vehicles
8:00	0 (0.0000)	955	0
8:15	15 (0.2500)	935	935
8:30	30 (0.5000)	905	1840
8:45	45 (0.7500)	870	2710
9:00	60 (1.0000)	820	3530

Five data points of cumulative input as a function of time in hours of the form $\{(t_1,0), (t_2, I(t_2)), (t_3, I(t_3)), (t_4, I(t_4)), (t_5, I(t_5))\}$ would be selected to calculate a polynomial equation for I(t) using Lagrange's interpolation formula (23). Time increments do not have to be uniform, although they may be. The resulting continuous function for I(t) would be:

$$I(t) = a_4 \cdot t^4 + a_3 \cdot t^3 + a_2 \cdot t^2 + a_1 t + a_0 \tag{4-1}$$

where:

$I(t)$ = cumulative input function, in vehicles, as a function of time, t, in hours.

a_i = calculated polynomial coefficients.

Cumulative Output

The same procedure is used to develop a cumulative output function for any continuous variable output function that may result due to the occurrence of an incident. Typical data are presented in Table 4-4. The variable cumulative output function would be of the form:

$$O(t) = c_4 \cdot t^4 + c_3 \cdot t^3 + c_2 \cdot t^2 + c_1 \cdot t + c_0 \tag{4-2}$$

where:

$O(t)$ = cumulative output function, in vehicles, as a function of time, t, in hours.

c_i = calculated polynomial coefficients.

TABLE 4-4. CUMULATIVE OUTPUT FOR 4-LANE AND 6-LANE FREEWAYS.

Time Since Incident Occurred, t min. (hrs.)	4-lane * Freeway O(t), vehicles	6-lane Freeway O(t), vehicles
0.0 (0.0000)	0	0
10.0 (0.1667)	318	580
20.0 (0.3333)	830	1367
30.0 (0.5000)	1435	2244
40.0 (0.6667)	2083	3186

* Used in example problem. See Figure 4-1.

The difficult problem is in accurately estimating the cumulative output function O(t), particularly for freeway incidents. Controlled incidents caused by lane blockages due to freeway maintenance operations (13) would be easier to estimate. The cumulative output function, O(t), is calculated from the expected output flow, E {o(t)}. Since the cumulative output function is the cumulative sum of vehicles flowing past the incident site as a function of time (t), then:

$$O(t) = \int_0^t E\{o(t)\}\ dt \qquad (4\text{-}3)$$

Since the duration of the usual freeway incident (accident, stall, etc.) is not known beforehand, estimates of its expected effect on output flow can be determined from expected values based on probability theory (14) and typical incident duration characteristics. The expected flow past the incident location as a function of time (t) after the incident begins may be calculated from:

$$E\{o(t)\} = P(d \le t) \cdot C_n + P(d > t) \cdot C_i \qquad (4\text{-}4)$$

$E\{o(t)\}$ = expected or average output flow past incident location at time t after incident begins, vehicles per hour.

$P(d \le t)$ = percent of incidents having duration less than time t, decimal.

$P(d > t)$ = percent of incidents having duration greater than time t, decimal.

C_n = normal, short-term, freeway capacity, vehicles per hour.

C_i = average capacity of freeway during incident, vehicles per hour.

Many factors affect the period of time an incident may block the freeway, including the type and location of incident. In Houston, approximately 45 percent of incidents were stalled vehicles and 45 percent were single-lane blockage accidents (15). Both of these types of incidents had similar incident capacities and durations of blockages. Durations of incidents were also studied in Dallas for 4-lane and 6-lane sections of the North Central Expressway (16). As shown in Figure 4-2, 40.5 percent of all incidents on the 4-lane section lasted 10 minutes or less while 43.3 percent of all incidents on the 6-lane section (Figure 4-3) had a duration of 10 minutes or less.

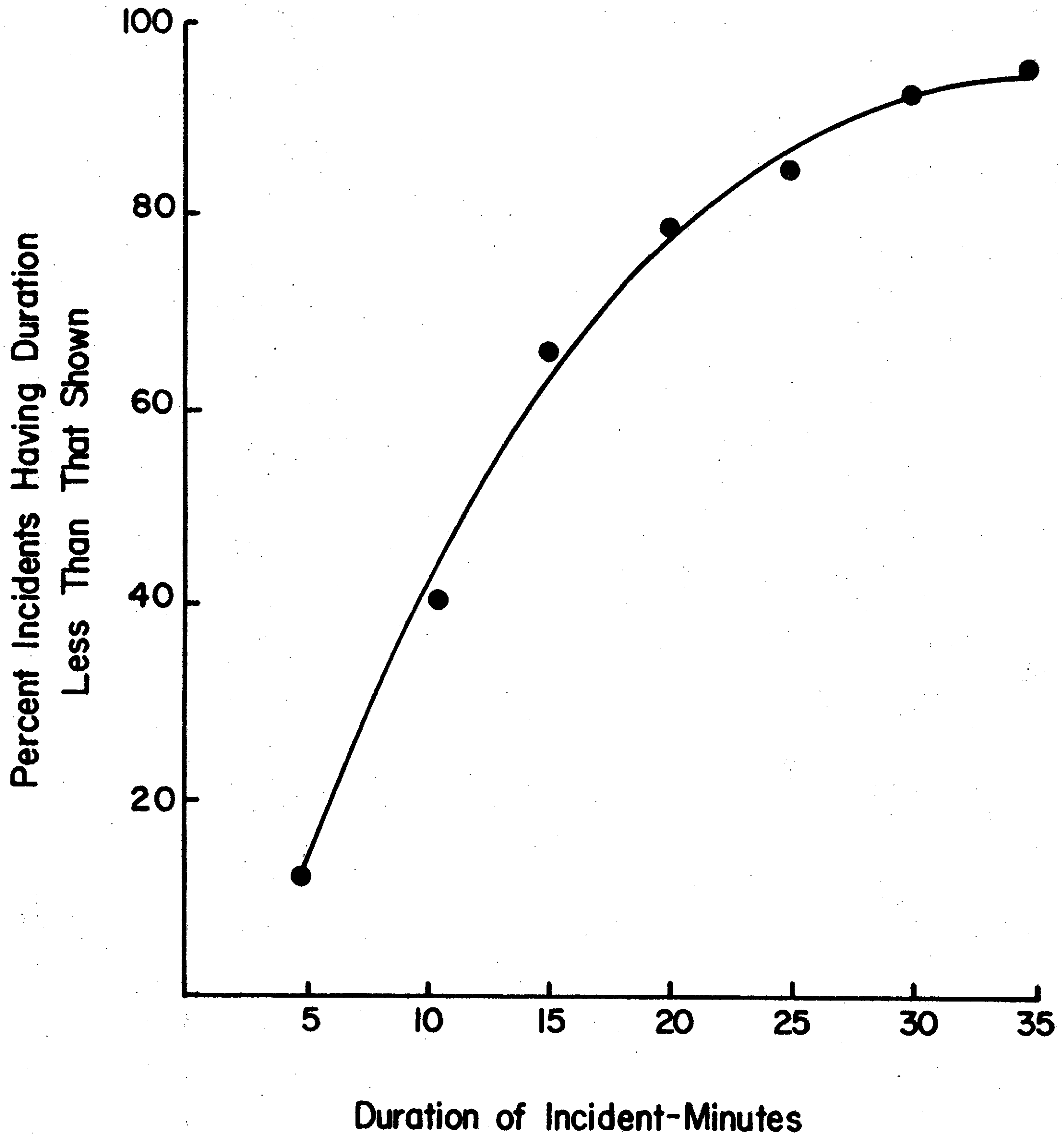


Figure 4-2. Cumulative Distribution of Incidents--4 Lane Section-NCE

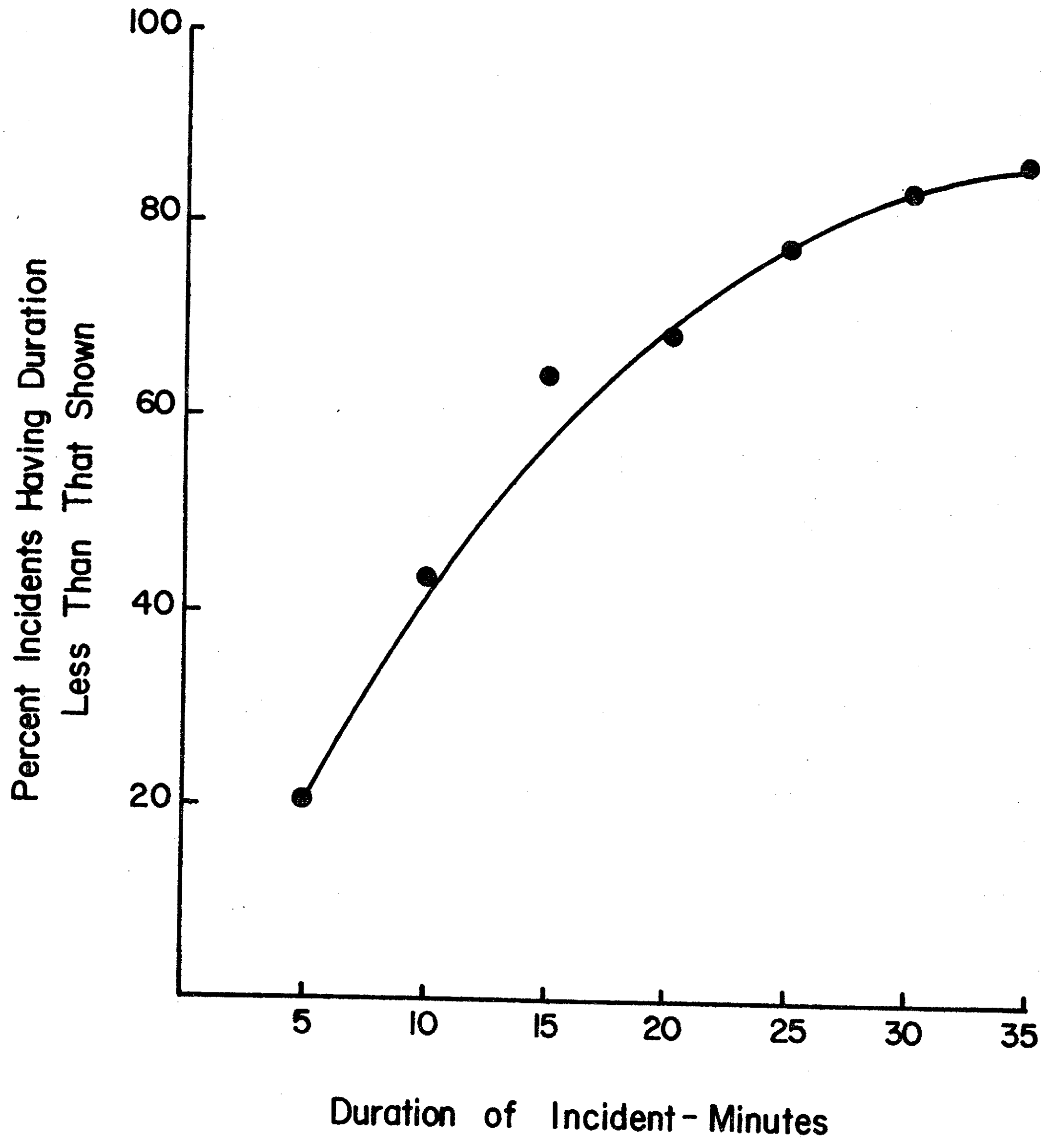


Figure 4-3. Cumulative Distribution of Duration of Incidents--6 Lane Section-NCE

The average capacity of a freeway incident (C_i) depends on the type of incident to some degree, but more on the number of lanes blocked. Approximately 90 percent of all incidents on the 6-lane Gulf Freeway in Houston (15) blocked one lane. The average flow through the incident section, C_i, was about 2800 vehicles per hour, which is the value assumed for 6-lane freeways. A one lane blockage on a 4-lane freeway reduces the incident capacity, C_i, to about 1500 vehicles per hour (16).

The normal capacity of a freeway (C_n) can be estimated using well known procedures (17) or it may be measured from surveillance techniques. However, the short-term (i.e., 5 to 15 minute) capacity flow rate should be used rather than an hourly flow rate. Normal, short-term capacity flow values for C_n of 4000 and 6000 vehicles per hour were assumed for 4-lane and 6-lane freeway sections.

Substituting the data of Figures 4-2 and 4-3, the capacity values previously described for E $\{o(t)\}$ in Equation 4-4, and solving Equation 4-4, yields the cumulative vehicle output results presented in Table 4-4. These results may be used as typical output data for the variable input-output travel time program if no other typical incident output flow data are available.

Travel Time

Assume the travel time of a vehicle is to be calculated for any time, t, after the start of the incident at 8:00 a.m. As illustrated in Figure 4-4, assume the vehicle enters from an entrance ramp 0.5 miles downstream, travels through the scene of the incident, and continues to the end of the freeway section. Input and output volumes are as previously described.

The normal freeway travel time to the location of the incident would be

$$tt_n = \frac{D}{u_n(t)} \tag{4-5}$$

where:

tt_n = normal freeway travel time to location of incident, in hours.

D = freeway travel distance, in miles.

$u_n(t)$ = normal average speed for freeway over D at time, $t = t_o$.

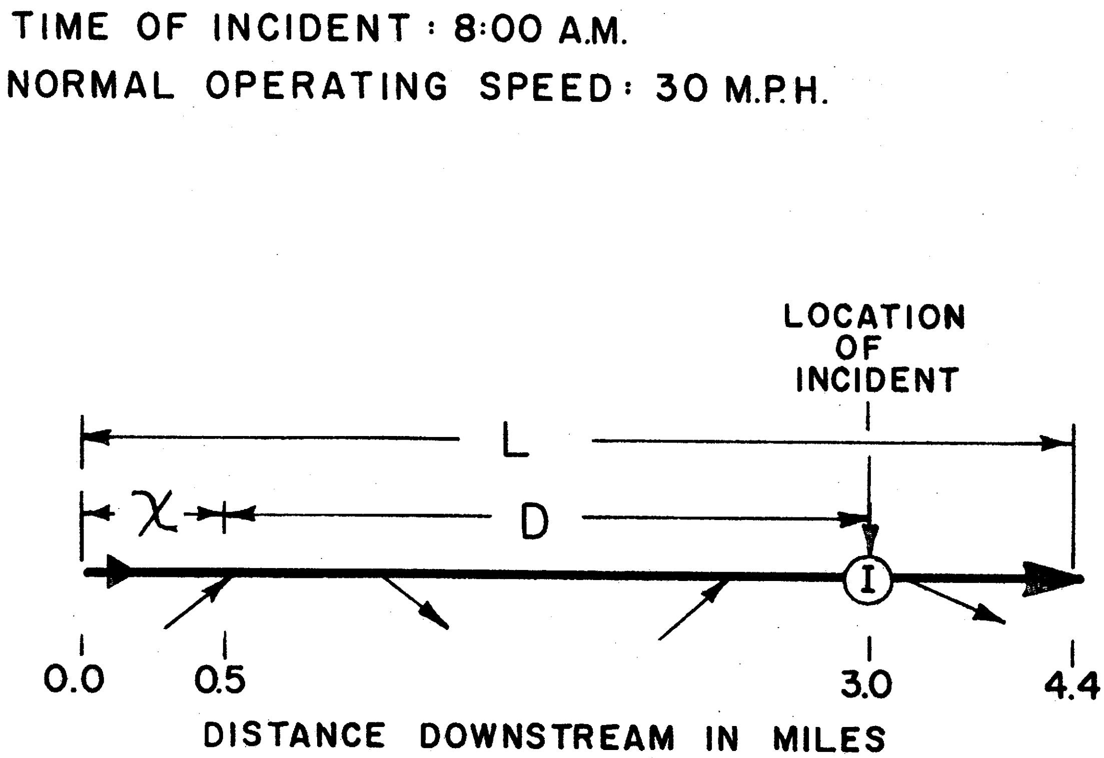


Figure 4-4. Freeway Section where Travel Times are to be Predicted.

Referring to the lower illustration in Figure 4-1, the cumulative input function must be evaluated at time $t = t_o + tt_n$, using Equation 4-1 as $I(t_o + tt_n)$. Since the delay this vehicle will experience is $D(t_o + tt_n)$ (See Figure 4-1.), it is necessary to determine the time, $t=y$, when the cumulative output function $O(t = y)$ will equal $I(t_o + tt_n)$. The solution to this problem (solving the 4th order polynomial for time $t = y$ given $O(t = y)$ is accomplished using the Newton-Raphson solution technique (11).

The predicted travel time through the freeway section is calculated from

$$TT(x,t) = \frac{L-X}{u_n(t)} + D\,(t_o + tt_n) \qquad (4\text{-}6)$$

where:

$TT(x,t)$ = predicted travel time in hours for vehicle at position x, in miles downstream, at time $t = t_o$, in hours after incident occurred.

L = length of freeway section, in miles.

X = milepost of vehicle entry, in miles.

$u_n(t)$ = normal average travel speed over section, L-X, at time t, in miles per hour. (Assumed to be 30 m.p.h.)

$D(t_o + tt_n)$ = predicted delay to vehicle, in hours, $= y-(t_o + tt_n)$

Travel Time Estimation

The results of the model are presented in Figure 4-5 for the example problem. The delay and travel times are those that a motorist entering the freeway system 0.5 miles downstream is predicted by the model to experience if he entered the freeway at the time shown. These results can be predicted as soon as the location and time of the incident are known. The model can predict up to seven entry locations at the same time.

Simulated Vehicle Travel Time Model

The theoretical variable input-output travel time model previously described can be used to predict future travel times along an urban freeway. For the operating freeway surveillance and control systems capable of measuring

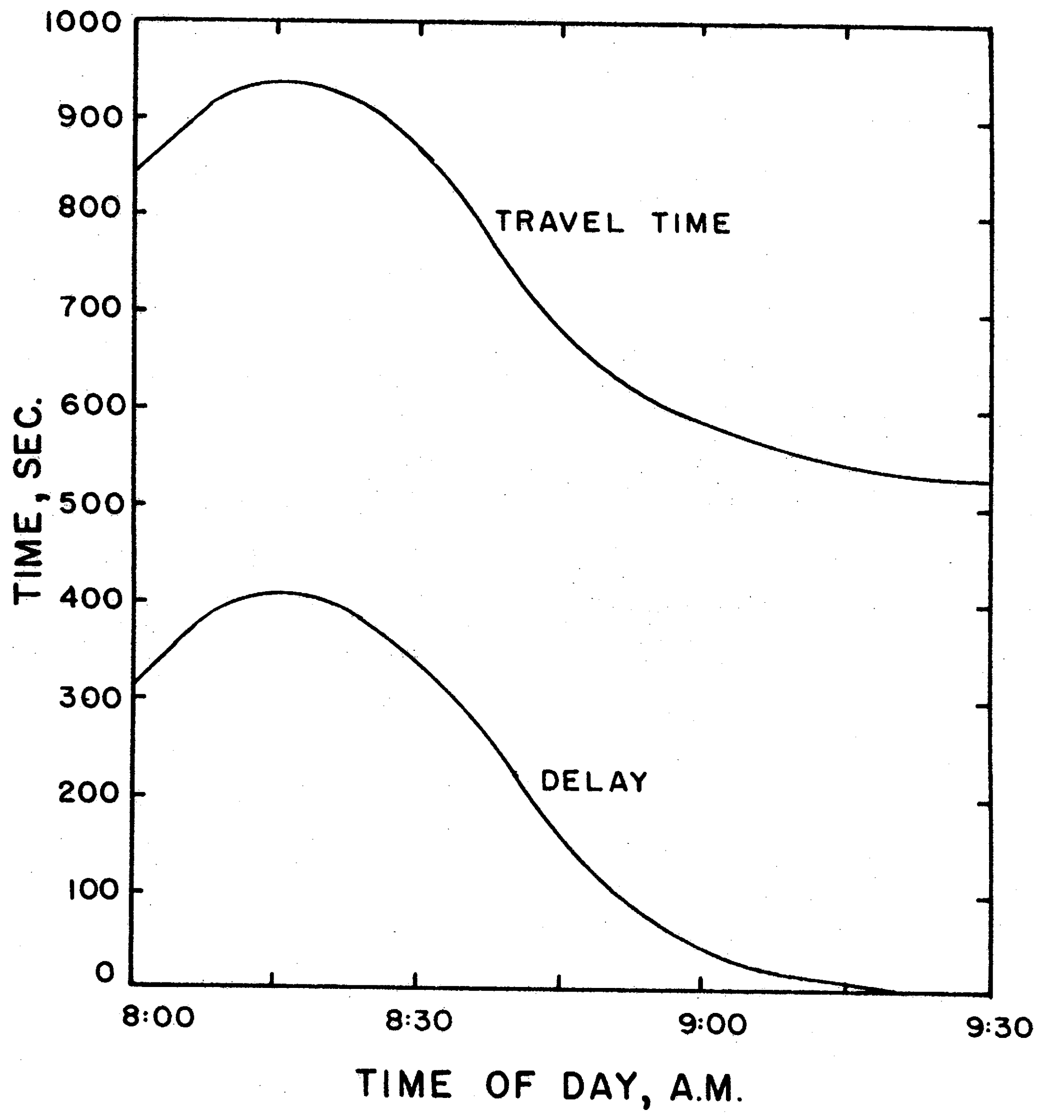


Figure 4-5. Delay and Travel Time Predicted by Variable Input-Output Travel Time Model for Example Problem

traffic speeds along the freeway, an efficient method is needed to automatically collect "real-time" freeway travel time data without having to maintain and conduct an elaborate systems input-output data collection program (18). In addition, it would be very useful to the freeway operating agencies to be able to test, evaluate and calibrate any freeway travel time prediction model (9, 10, 20) that the operating agency may elect to use, including the variable input-output model previously described.

An efficient freeway travel time model was developed, programmed and implemented on U.S. 75 (North Central Expressway) in Dallas as part of this study. The North Central Expressway computer surveillance and control system is capable of collecting traffic speed data from freeway detectors approximately every one-half mile along the freeway.

Imaginary Vehicle Model

Using the speed detectors located in the outside lane, and using the relationship for time given speed and distance, the model projects an "imaginary" vehicle down the freeway. This "vehicle" can be started down the freeway at any point in time. The available speed data are used in predicting a travel time from one station to the next. A step by step example of how this program works is perhaps the most effective explanation of it.

Suppose travel time data is desired for the freeway during incident conditions starting at 8:00 a.m. The program will use the speed (u) at speed station #1 at 8:00 a.m. and the distance (d) between station #1 and station #2, and using the equation

$$t = \frac{d}{u} \qquad (4\text{-}7)$$

will predict the time (t) it takes to get to speed station #2. The program will take this predicted arrival time and use the speed at speed station #2 at the predicted arrival time and then project an arrival time to speed station #3. This procedure is repeated to the end of the freeway section and the travel times between stations are summed. This sum is the time that it would take an average vehicle to travel the freeway section under consideration.

The speed values used should be the average speed of the traffic stream for the past minute or so and not the speed of an individual vehicle. In Dallas, exponential smoothing is used to obtain average speeds. It would be desirable if the average speed value at a speed station also represented the average speed across all freeway lanes. It may not be practical or necessary to have a speed detector in every lane. Study results in Dallas using only one lane speed detection in the outside lane (probably the least desired condition) provided good correlation between actual measured vehicle travel times and model results.

This travel time model has proven to be very representative of the travel times a motorist experiences on the freeway. Actual travel times were measured on the North Central Expressway to determine the accuracy of this program. The freeway was driven by project staff in a standard auto and travel time measured using a stop watch. Table 4-5 shows the measured and calculated travel times and the percent difference between the two. It is to be noted that the freeway detector system was calibrated prior to the travel time study. These travel times include travel made over four-lane and six-lane freeway sections. Most runs were made over the outside lane. Insufficient travel time data were collected to reliably measure travel time in the median or middle lanes. It is speculated that the middle freeway lane is more representative of average freeway conditions. Staff observations made during the Dallas travel time study suggests that travel time differences of no more than 1 minute occurred between the median and outside freeway lanes over the ten mile section studied during both peak and off-peak conditions.

Travel time along a freeway varies with volume and congestion conditions as shown for a 4.3 mile section of outbound North Central Expressway in Figure 4-6. As expected, travel time increased as volumes increased until demand exceeded capacity. At this point, volumes decreased and travel time continued to increase with the freeway breakdown. If a motorist were traveling 30 mph (capacity flow) on this section, the travel time would be 520 seconds. Travel times exceeding 520 sec. in Figure 4-6 are due to a freeway breakdown and are not caused by an incident. As the volumes decreased, the travel times decreased; but when the vehicles began to progress smoothly at the 30 mph point, the volumes began to increase again and approach the freeway capacity.

Effects of Rain on Freeway Travel Time

The effects of rain on freeway operations has typically been discussed in terms of headways, speeds, and volumes. Research conducted by the Texas Transportation Institute has found that as rain intensity increases the headways on the freeways increase (21). This research has also found that motorists tend to reduce their speeds as rain intensity increases because of reduced

TABLE 4-5. U.S. 75 TRAVEL TIME CALIBRATION STUDIES

Date	Study No.	Time	Direction	Measured Travel Time	Calculated Travel Time	% Difference
5/10	1	4:07pm	OB	14:25	13:59	-3.01%
5/10	2	4:25pm	IB	13:24	13:43	+2.36%
5/10	3	4:41pm	OB	24:00	22:33	-6.04%
	4*	5:18pm	IB			
5/10	5	5:30pm	OB	26:49	26:26	-1.43%
5/11	6	7:35am	IB	18:53	18:23	-2.65%
5/11	7	7:57am	OB	12:17	12:40	+1.81%
5/11	8	8:13am	IB	14:02	14:38	+4.28%

* Study discontinued because of stalled vehicle.

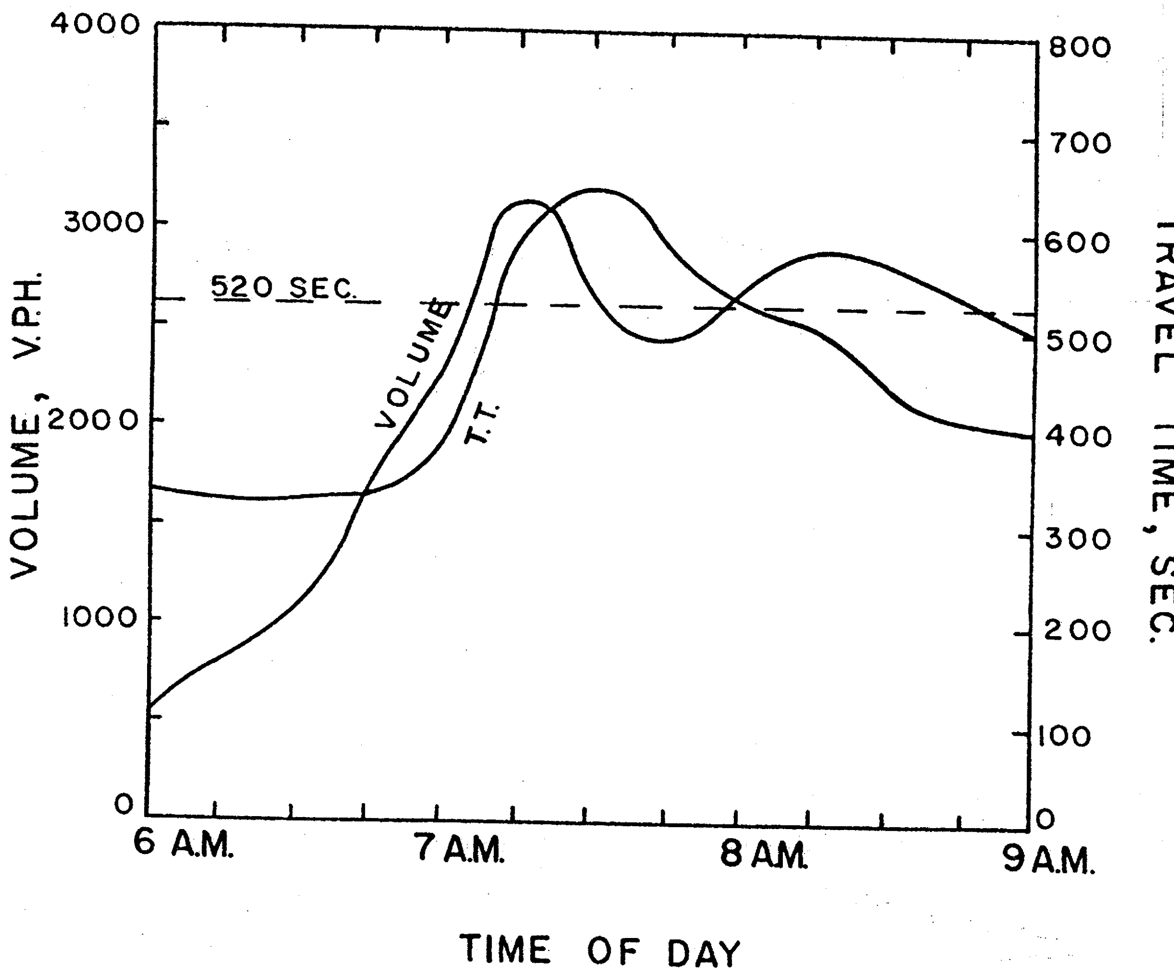


Figure 4-6. Volumes and Travel Times for the 4.3 Mile Section of North Central Expressway

visibility. This reduction in speed and increase in headways result in an increase in travel time along the freeway.

Travel time data were collected along North Central Expressway using the simulated vehicle program and are shown in comparison to normal condition travel times in Figure 4-7. The results show a dramatic increase in travel time along this section on the day of the rain. Note that the travel times decreased between 3:00 p.m. and 3:50 p.m., when the rain stopped, but again increased when the rain resumed. The usual peak period congestion was amplified by the rain on the freeway.

Summary

A simulated vehicle travel time model was developed in the research to provide a quick and accurate means of collecting travel time data without the need of actually driving on the freeway. This program was developed for use on the North Central Expressway, but could be applied to any freeway which has some type of computer control system. The program listing is presented in the Appendix.

A Study of Frontage Road Travel Times on North Central Expressway

Introduction

In order to calibrate and confirm results of the PASSER III - Frontage Road Level of Service Program studies were made along the frontage road of U.S. 75 (North Central Expressway) from Mockingbird Lane to Caruth. This section includes four signalized diamond interchanges with studies being made for both the inbound and the outbound direction. Studies did not include the Mockingbird interchange and the Caruth interchange, but included all the interchanges between those two.

With the results of these studies, conclusions were made as to a method of developing a means to predict travel times along an alternate route such as a frontage road or city street when an incident occurs on a freeway main lane. Insight into the factors which cause delay and increase travel time was obtained from the data and implemented into the PASSER III program.

Similar studies were conducted on the frontage roads of I-10 (Katy Freeway) in Houston and I-35E (Stemmons Freeway) in Dallas. These studies were not as intense as those made on North Central Expressway, but did provide insight into the characteristics of delay at the signalized intersection.

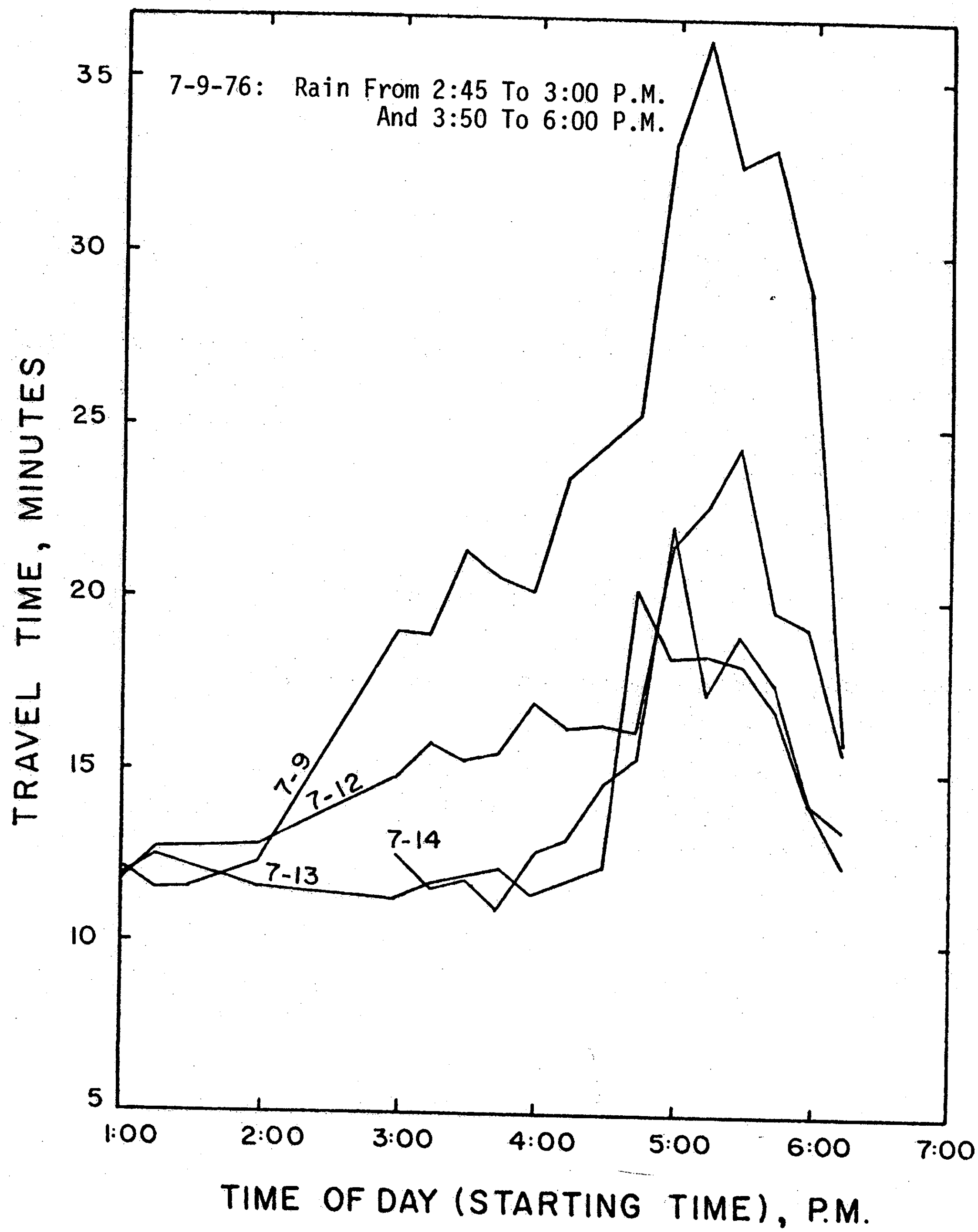


Figure 4-7. Effects of Rain on Travel Time on North Central Expressway

The results of these studies will be incorporated in this report, but, the procedure will not be discussed as it parallels closely the travel time studies conducted on North Central Expressway.

Methods of Research

The studies were conducted in a TTI vehicle, located at the Dallas office, from 7:30 a.m. to 10:15 a.m. Two researchers drove the frontage road section and recorded the delay and travel time data. Each study was started from a known point before arriving at the first signalized diamond interchange. Cumulative travel times were recorded with a stop watch between intersection centerlines to a known point beyond the last interchange.

The delay encountered at each signalized interchange was recorded by the driver. It can be shown that delay occurs whenever the speed drops below one half the running speed. Typically the running speed was 35 mph which is the speed limit. However, the driver tried to drive with the flow of traffic, and during light traffic conditions this speed was about 40 mph. As the driver approached the signalized interchanges he recorded the time the speed was below 17 mph. This time was then recorded on the data sheet next to the cumulative travel time for that particular intersection. Delay was recorded for each intersection and was not recorded as a cumulative time.

The final recorded value was the estimate of the traffic conditions. This was recorded at the end of each run, and was evaluated in terms of light, moderate, heavy, and incident traffic conditions.

A total of 60 runs were made on North Central Expressway with 20 runs made in a similar manner on Katy Freeway. These studies were started before the peak period and were continued until peak traffic cleared. Stemmons Freeway studies were done only during the peak hour.

The diamond interchange signals on North Central Expressway were all of the interconnected, traffic responsive fixed sequence type and progression was provided along the study section. The interchanges on West Katy and Stemmons were fully actuated with no progression provided.

Results

In order to effectively evaluate these data, several different factors were evaluated for each traffic condition (light, moderate, heavy, and incident) in both directions of travel. These factors were: (1) number of runs, (2) average travel time, (3) number of stops per run, (4) stops per mile, (5) stops per intersection, (6) average delay time (7) average delay per intersection,

(8) average delay per stop, (9) average delay per mile, (10) average travel time per mile, (11) delay per mile divided by travel time per mile, (12) running travel time per mile divided by travel time per mile, (13) delay per mile minus travel time per mile.

As expected each of these factors worsened as traffic conditions along the frontage road worsened, except for delay per mile minus travel time per mile. Figure 4-8 shows four of the different factors plotted with traffic conditions versus time and delay per mile minus travel time per mile plotted as a straight line. This difference yields the running speed between intersections, and it can be seen that it remains relatively constant for all traffic conditions.

The fact that the running speed between signals remains constant with varying traffic conditions would indicate that the travel time along the frontage road is a function of the delay at the signalized interchanges. The delay at the signal is a function of volumes and types of signalization with the latter being more important. With actuated equipment like that on Katy and Stemmons, large delays (i.e. greater than 2 minutes) were encountered; whereas, on North Central delay was about 50 seconds during similar volume conditions.

Progression along the frontage road reduced frontage road travel time considerably as can be determined from Table 4-6. During moderate traffic conditions, the average delay experienced per vehicle per intersection was 19.33 seconds along North Central Expressway and 43.87 along Katy Freeway. During heavy (peak hour) traffic, the average delays experienced were 31.03 (North Central), 52.24 (Katy) and 78.01 (Stemmons). The running speed travel time per mile divided by the travel time per mile is a type of efficiency ratio. This is the time it would take to travel a mile at 35 mph divided by the actual average time it took to travel one mile of the study section. As this ratio approaches 1.00 the frontage road comes closer to providing the motorist with a travel time with no delay incurred. During moderate traffic, this ratio is 0.86 for North Central and 0.57 for Katy. During heavy traffic, this ratio was 0.72 (North Central), 0.50 (Katy) and 0.39 (Stemmons). Note that, in reality the frontage road operating speed during heavy traffic along North Central is 25.2 mph (0.72 x 35), 17.5 mph (0.50 x 35) along Katy, and 13.7 mph (0.39 x 35) along Stemmons. North Central progressive control provided better frontage road operations in all cases.

Conclusions

Travel time along frontage roads can be expressed in terms of four variables:

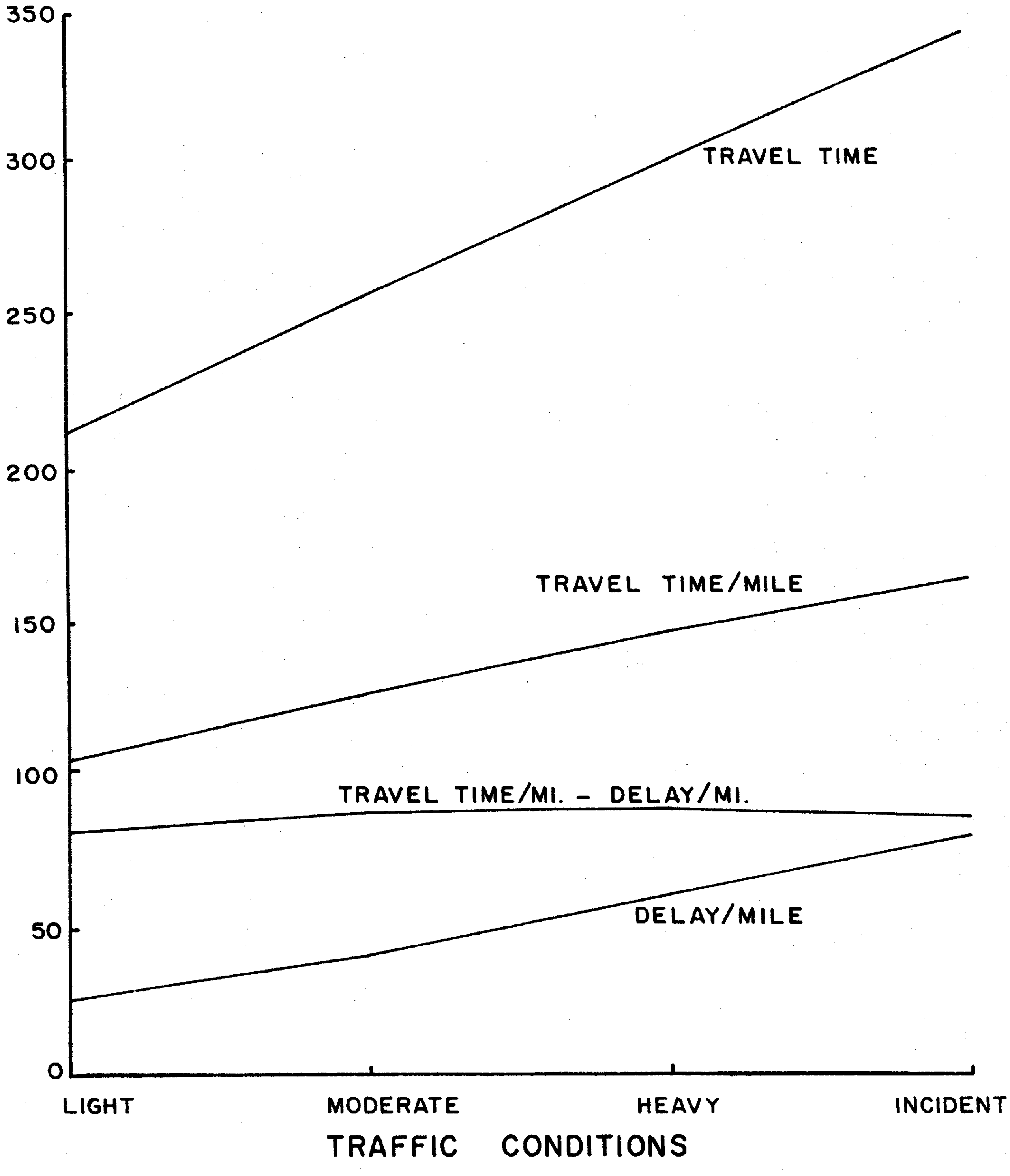


Figure 4-8. Frontage Road Travel Time Characteristics For Different Traffic Conditions

TABLE 4-6. COMPARISONS OF FRONTAGE ROAD OPERATIONAL MEASURES ALONG NORTH CENTRAL EXPRESSWAY, WEST KATY FREEWAY AND STEMMONS FREEWAY

Operational Measures	North Central Expressway[a]	Katy Freeway[b]	Stemmons Freeway[b]
	Moderate Traffic Conditions		
Stops/intersection	0.72	0.87	-
Delay/intersection	19.33	43.87	-
Delay/mile	37.98	80.27	-
Travel Time/mile	121.02	181.68	-
35 mph T.T./mile / Travel Time/mile	0.86	0.57	-
	Heavy Traffic Conditions		
Stops/intersection	0.87	0.93	0.94
Delay/intersection	31.03	52.24	78.01
Delay/mile	61.05	100.90	156.02
Travel Time/mile	146.32	211.16	261.22
35 mph T.T./mile / Travel Time/mile	0.72	0.50	0.39

[a] Interconnected fixed sequence

[b] Isolated full-actuated

time of day, type of signalization, freeway operating condition, and speed limit; with these variables known, traffic engineers could estimate travel time along a frontage road after some local data had been collected.

Research shows that frontage road travel time was lower where progression was provided through the signalized diamond interchanges. PASSER III estimates an average travel time along a frontage road where progression is provided. It does this by calculating the travel time between interchanges from a given running speed between each interchange and adding a calculated average delay for each intersection to get the travel time along the section. It should be noted that the running speed between interchanges could be affected by the number of lanes, parking, high volume exit ramps, and queues at entrance ramps where ramp metering is used.

Perhaps the most important aspect of this research was determining the effect that different types of signalization at interchanges have on travel time. It was found that some type of fixed-time or computer controlled configuration providing progression is more efficient than actuated control on a frontage road system. Excessive delays will be incurred at actuated signals where moderate or high cross street volumes exist.

REFERENCES

1. Goolsby, M.E. Influence of Incidents on Freeway Quality of Service. Highway Research Record 349, 1971, pp. 41-47.
2. Gervais, E.R. Project Report. National Proving Grounds for Freeway Surveillance, Control, and Electronic Aids, Aug. 1966.
3. Hoff, G.C. A Comparison of Selected Traffic Control Devices. Highway Research Record 366, 1971, pp. 116-129.
4. West, J. California Makes Its Move. Traffic Engineering, Jan. 1971, pp. 12-18.
5. Dudek, C.L., and Jones, H.B. Evaluation of Real-Time Visual Information Displays for Urban Freeways. Highway Research Record 366, 1971, pp. 64-77.
6. Wattleworth, J.A., Courage, K.G., and Carvell, J.D. An Evaluation of Two Types of Freeway Control Systems. Texas Transportation Institute, Res. Rept. 488-6, April 1968.
7. Dudek, C.L., Messer, C.J., and Jones, H.B. Study of Design Considerations

for Real-Time Freeway Information Systems. Highway Research Record 363, 1971, pp. 1-10.

8. Heathington, K. W., Worrall, R.D., and Hoff, G.C. Attitudes and Behavior of Drivers Regarding Route Diversion. Highway Research Record 363, 1971, pp. 18-26.

9. Messer, C.J., Dudek, C.L., and Friebele, J.D. Method for Predicting Travel Time and Other Operational Measures in Real-Time During Freeway Incident Conditions. Highway Research Record 461, 1973, pp. 1-16.

10. Messer, C.J. and Dudek, C.L. Development of a Model for Predicting Travel Time on an Urban Freeway. Texas Transportation Institute, Research Report 165-8, January 1974.

11. Wattleworth, J.A. and McCasland, W.R. Study Techniques for Planning Freeway Surveillance and Control. Highway Research Record 99, 1965, pp. 200-223.

12. Froberg, C.E. Introduction to Numerical Analysis. Addison-Wesley Publishing Company, Inc., 1965, pp. 142-146.

13. Kermode, R.H. and Myrra, W.D. "Freeway Lane Closures," Traffic Engineering, February 1970.

14. Morse, P.M. Queues, Inventories and Maintenance. Operations Research Society of America. Publications in Operations Research Number 1, 1967.

15. Messer, C.J., Dudek, C.L. and Loutzenheiser, R.C. A Systems Analysis for Real-Time Freeway Traffic Information System for the Inbound Gulf Freeway Corridor. Texas Transportation Institute, Research Report 139-5, April 1971.

16. Stockton, William R. A Motorist Information System for the North Central Expressway Corridor. An unpublished report. Texas A&M University, May 1973.

17. Drew, Donald R. Traffic Flow Theory and Control. McGraw-Hill Book Company, 1968.

18. Ritch, G.P. Documentation of Computer Control of the Ramp Metering System on the Gulf Freeway. Texas Transportation Institute Research Report 139-12, 1971.

19. Makigami, Y. and Woodie, W.L. Freeway Travel Time Evaluation Technique. Highway Research Record 321, 1970, pp. 33-45.

20. Chow, W.M. A Study of Traffic Performance Models Under an Incident Condition. Transportation Research Board 567, 1976, pp. 31-36.

21. Mounce, J.M., Stockton, W.R. and Walton, N.E. Freeway Operations Near

Entrance Ramps During Adverse Weather (Rain). Texas Transportation Institute, an unpublished report pending publication, 1976.

CHAPTER 5

FRONTAGE ROAD RAMP TO CROSS-STREET DISTANCE REQUIREMENTS IN URBAN FREEWAY DESIGN

Introduction

The modern urban freeway was conceived and constructed to move large numbers of persons and goods safely and efficiently over considerable distances. The basic design objective was to provide a high level of service in an economical manner. One of the consequences of this design objective was that relatively long spacings of freeway ramps and interchanges have been selected to the extent possible to minimize the effects of weaving on the freeway flow. Apparently, little attention has been given to the resulting negative effects on the connecting ramps and frontage roads (1, 2). Neither AASHTO (1) nor Leisch (2) discuss this separation requirement between the intersection and ramps in any great detail. The traffic engineer currently has a minimum of design criteria or procedures available to use in objectively selecting a desirable ramp to cross-street separation distance. Some designers have used rule-of-thumb procedures such as 500 feet for exit ramps and 750 feet for entrance ramps.

Significant operational problems have been observed on urban freeway ramps and frontage roads near diamond interchanges, especially in cases where ramp metering systems are in operation. In most cases, operational problems on connecting exit and entrance ramps are directly related to insufficient ramp-crossroad space. These problems are of three different types:

a. Interchange signal queues blocking merge areas of exit ramps and the frontage road (Fig. 5-1)
b. Interchange signal queues backing onto the freeway main lanes
c. Ramp metering queues backing into the cross-street intersection (Fig. 5-2)

Freeway exit ramp location and design should be capable of storing enough vehicles to prevent the "spillback" of the ramp vehicles onto the freeway. The dangerous condition of spillback should not be tolerated as a recurring event and may occur only as a result of unusual circumstances. Entrance ramp location and design should have sufficient length to minimize queue spillback into the adjacent cross-street intersection due to ramp metering. The installation of ramp metering is a strong possibility on many urban freeways, even on newly constructed facilities, and its requirements should be considered in the freeway

Figure 5-1. Exit Ramp Blockage Caused By Interchange Queues

Figure 5-2. Intersection Blockage Caused By Entrance Ramp Queues

design process.

Study Objective

The objective of this study was to investigate the location of entrance and exit ramps with regard to their effects on the operation along the frontage road approaches to an interchange and to develop criteria to determine the appropriate distance to space ramps from interchanges. This chapter will identify the different components which determine ramp distance requirements and incorporate them into analytic exit and entrance ramp models. The chapter will also provide information on various field studies conducted on freeways throughout Texas. Finally, design criteria and recommendations based on the data collected through this research will be presented.

Exit Ramp Spacing

Exit Ramp Model

The approach used for determining the storage length needed to prevent blockage of the ramp merge area considers three storage length components. The geometric configuration shown in Figure 5-3 gives the three traffic operational components used to compute the exit ramp-to-interchange spacing. These three components: weaving, stopping, and queueing distances, will be discussed in the paragraphs to follow, and perhaps it would be best to develop the length from the exit ramp to the signalized intersection of the interchange.

The weaving distance needed to perform the weaving maneuver is the first distance which is to be provided. The basic weaving model presented in the Highway Capacity Manual is used (3). Table 5-1 presents the required weaving distances for three levels of weaving "quality of flow" (3) based on urban and suburban arterial operational criteria. Total weaving volume must be estimated from exit ramp and frontage road volumes and their respective turning movements at the interchange. It was felt desirable that no braking should be required to occur during this weaving movement. The motorist should not be required to perform but one basic driving task at a time. Therefore, this movement should be completed before the motorist brakes to a stop.

The next component of the assumed driving maneuvers is the safe stopping distance. This length can be readily found using the equation:

$$SSD = 1.47\ V \cdot T + \frac{V^2}{30 \cdot f}$$

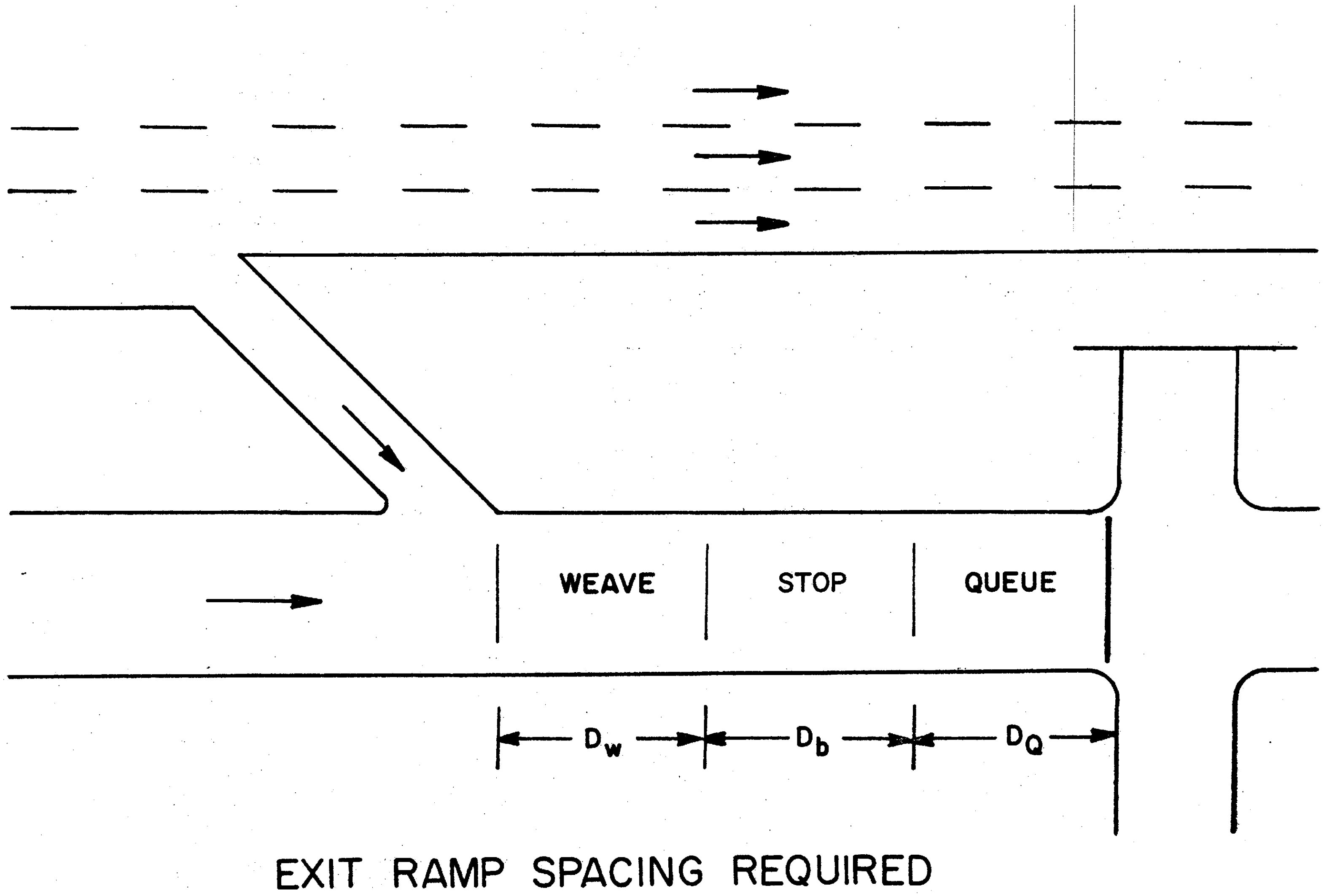


Figure 5-3. Exit Ramp Spacing Required

TABLE 5-1. WEAVING LENGTHS FOR VARIABLE WEAVING VOLUMES AND DESIGN LEVELS

Total Weaving Volume[a] (epcph)	Design Levels 1	2	3
	Quality of Flow III-IV (A-B)[b]	IV (C-D)[b]	V (E)[b]
100	50	50	50
200	50	50	50
300	50	50	50
400	100	50	50
500	100	50	50
600	100	50	50
700	200	100	50
800	250	100	50
900	300	150	50
1000	350	200	50
1100	400	200	50
1200	450	250	50
1300	500	300	50
1400	550	300	50

[a]Total Weaving volume is assumed to be 63 percent of total frontage road approach volume.

[b]Levels of service based on urban and suburban arterial criteria, p. 173-5, HCM. Assumes number of lanes is adequate for weaving.

where

SSD = safe stopping distance, feet.

V = frontage road speed, miles/hour.

T = perception-reaction time, sec.

f = coefficient of friction.

The driver must safely stop before he reaches the end of the queued vehicles stopped at the interchange traffic signal. Solutions to the safe stopping sight distance equation are shown in Figure 5-4 for perception-reaction times (T) of 1.0 and 2.5 seconds. Deceleration rates and resulting coefficients of friction vary with approach speed. Values used in this paper are those given by AASHTO (1). A 1.0 second perception-reaction time should be considered to provide only a minimum condition reaction time; whereas, a 2.5 second time is considered desirable.

The queue length of the interchange is the final component in the exit ramp model. The design queue length can be obtained from Figure 5-5, which previously has been developed by the Texas Transportation Institute for the Texas State Department of Highways and Public Transportation (4).

Exit Ramp Studies

In order to develop and test the model, several types of studies were conducted. It was noted that these studies should be conducted on several different freeway locations in order to account for the varying situations which might exist. U.S. 75 (North Central Expressway) in Dallas, U.S. 59 (Southwest Freeway) and I-10 (Katy Freeway) in Houston and some studies in Corpus Christi were chosen. These freeways are varied with respect to geometrics and volume experienced.

It was considered desirable to conduct three types of studies on exit ramps: (1) volume counts, (2) queue counts, and (3) spacing between the ramp and interchange. Exit ramp volumes were taken on I-10 in Houston and on U.S. 75 in Dallas. Cumulative exit ramp volumes used to classify volume levels are shown in Figure 5-6. Eighteen exit ramps are included in the A.M. and P.M. I-10 studies. Twenty-two exit ramps are included in the U.S. 75 counts. A particularly troublesome high volume exit ramp in Corpus Christi has a peak hour volume of 1025 vehicles per hour. These volumes provide a base for calculating weaving distances.

Figure 5-7 presents study data collected on measured exit ramp distances versus cumulative percent of ramps for twenty ramps on U.S. 75 in Dallas and

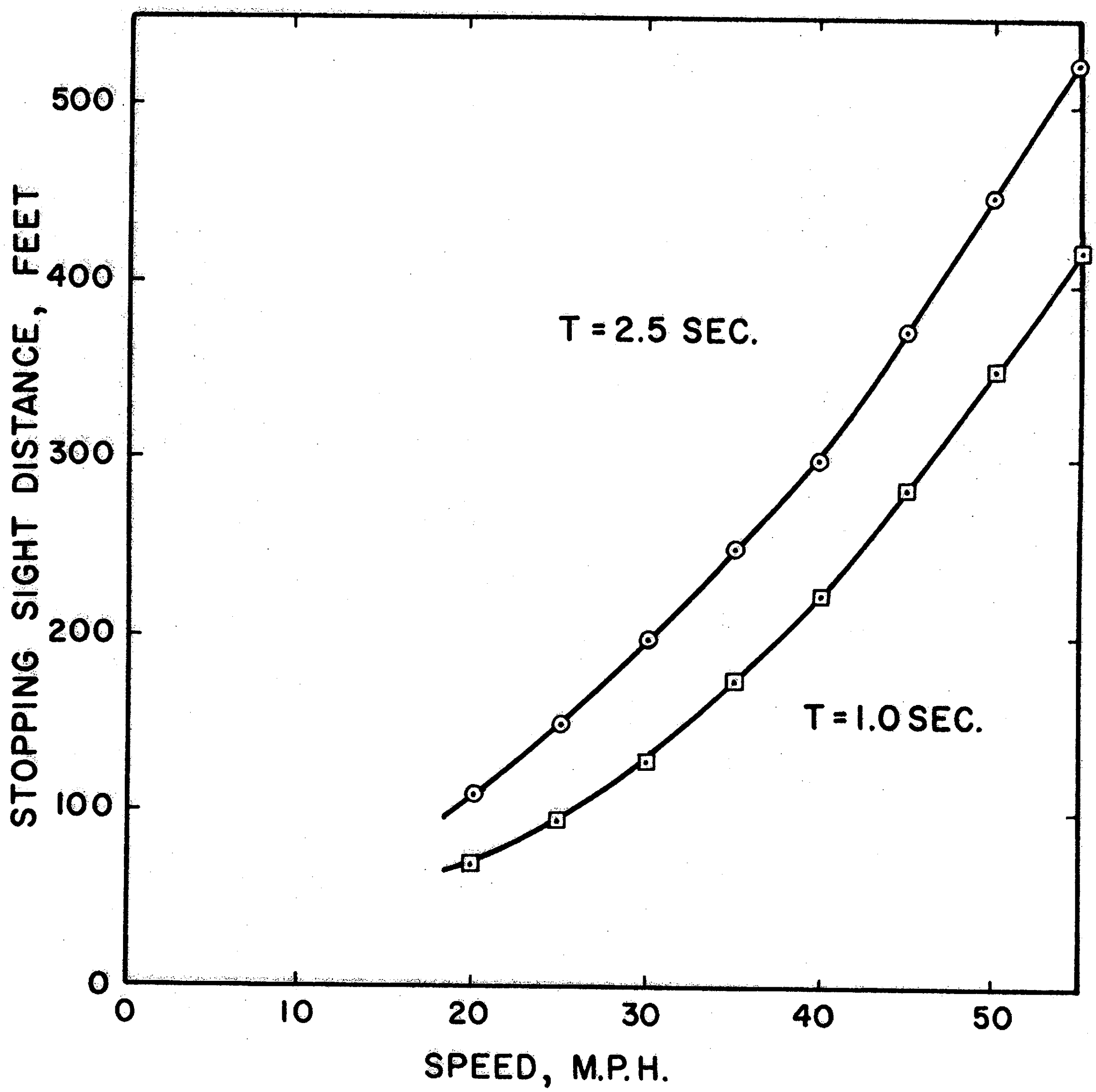


Figure 5-4. Stopping Sight Distance Versus Speed With Perception-Reaction Times of 2.5 Seconds and 1.0 Seconds.

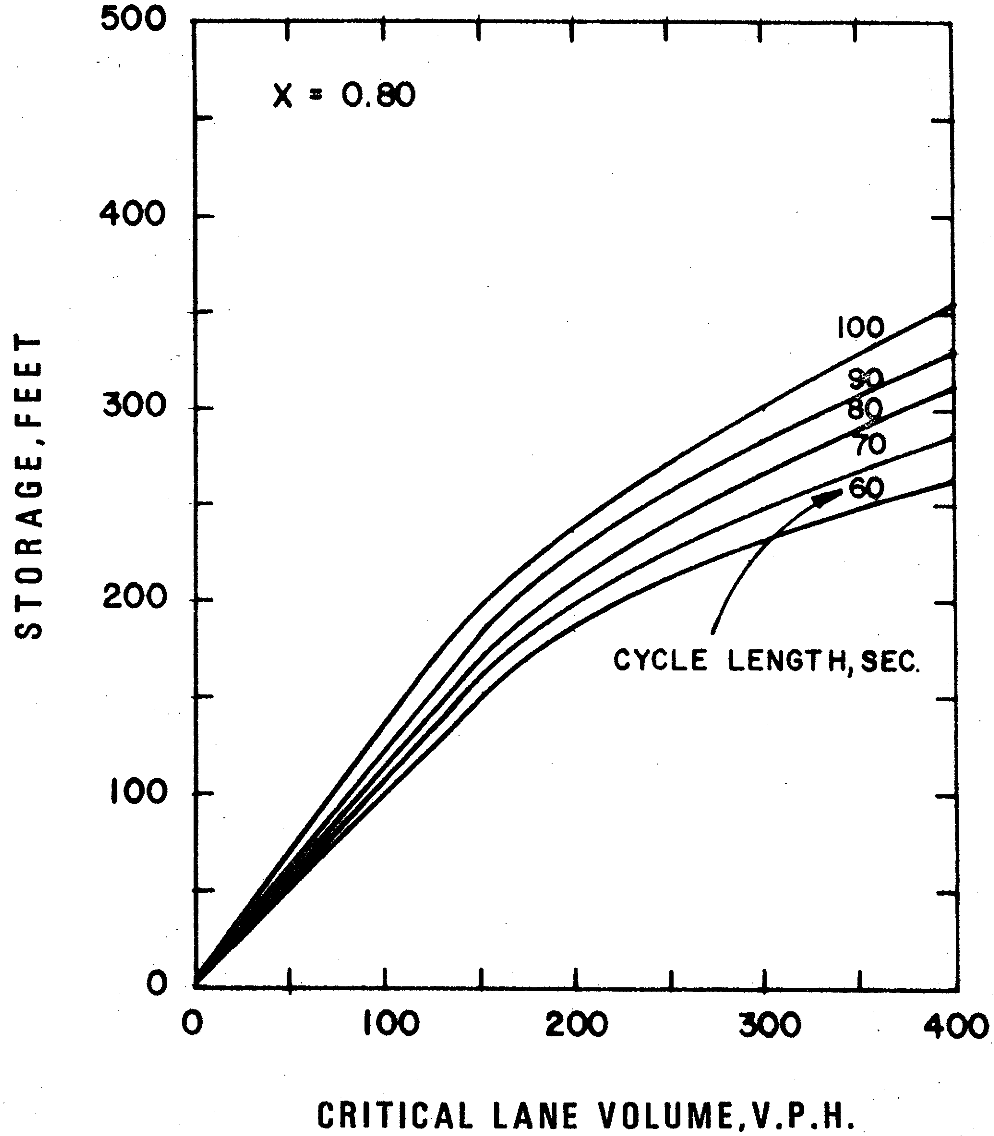


Figure 5-5. Storage Versus Lane Volume
(Modified Poisson)

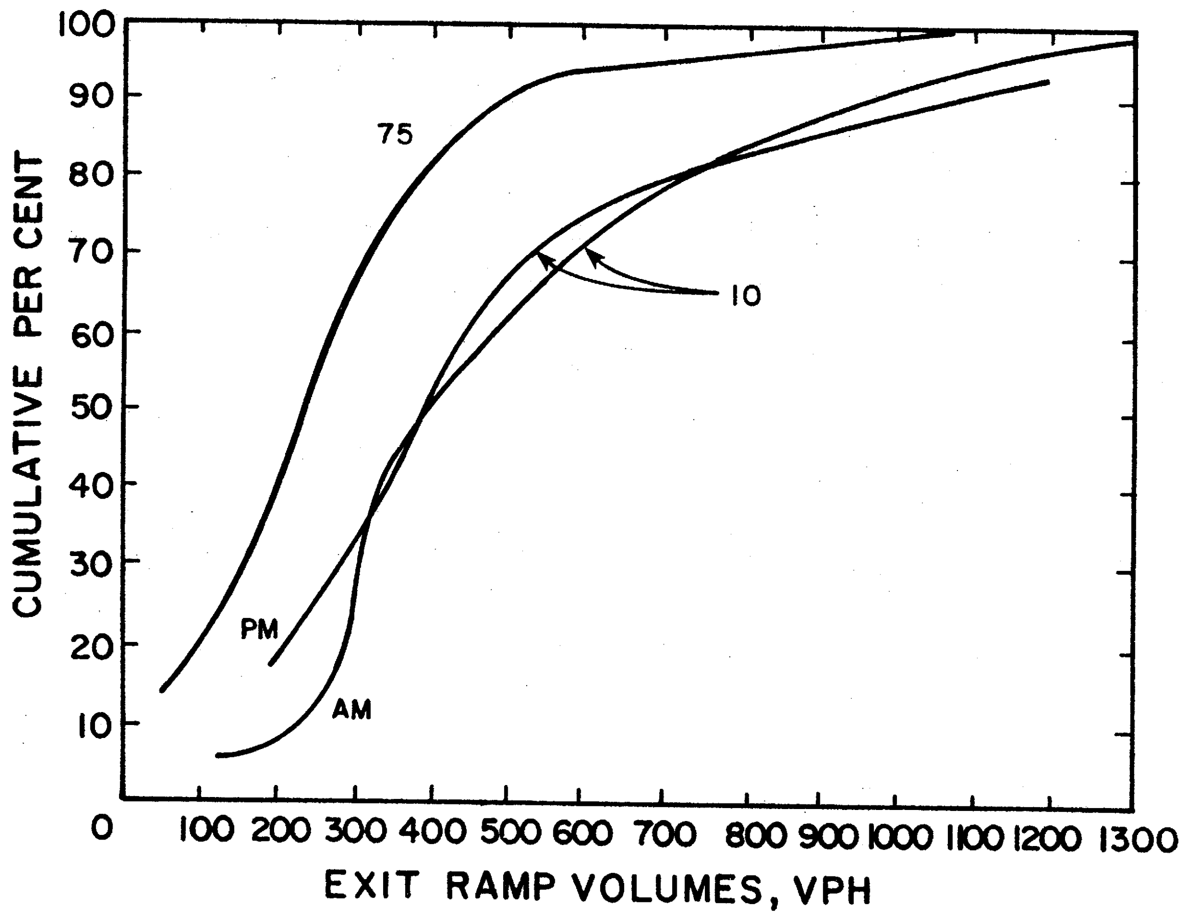


Figure 5-6. Exit Ramp Volumes in Cumulative Percent for U.S. 75 (Dallas) and I-10 (Houston)

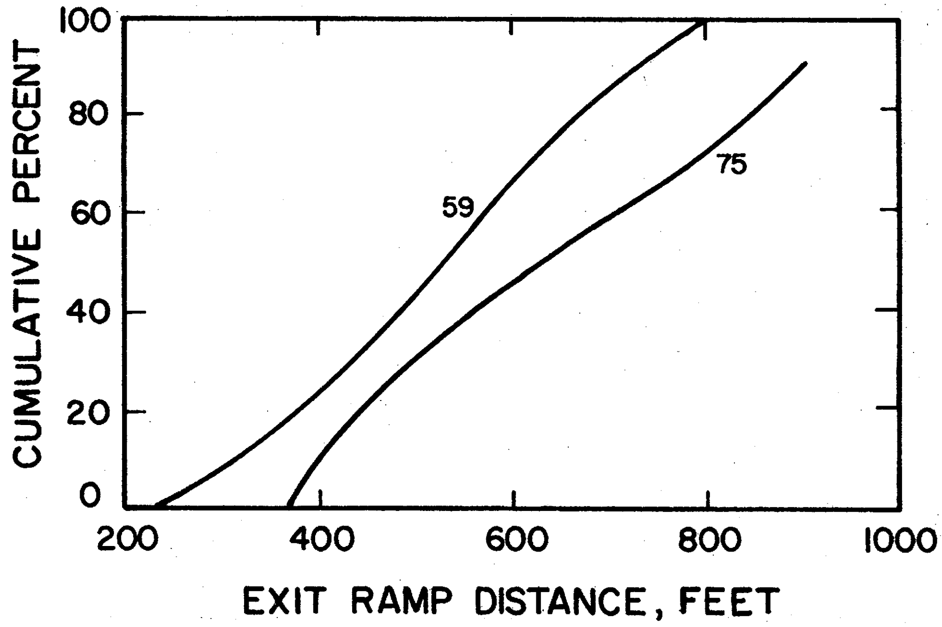


Figure 5-7. Exit Ramp Distances in Cumulative Percent for U.S. 75 (Dallas) and U.S. 59 (Houston)

ten ramps on U.S. 59 (Southwest Freeway) in Houston. The median (50-th percentile) distance is about 500 feet on U.S. 59 and 600 feet on U.S. 75. Distances were measured from the physical nose to the stop line of the intersection. These studies may be used to compare model results with existing ramp spacings.

Exit Ramp Design Criteria

The exit ramp spacing model has been formulated in terms of three component lengths: weaving distance, stopping distance and queue length. The distance required due to weaving is primarily related to the exit ramp volumes and the total weaving volume. The exit ramp volume data indicates that 95%-tile exit ramp volumes of the two study sites in Figure 5-6 are approximately 690 and 1100 vehicles per hour. These are termed "moderate" and "high volume" conditions, which are the two basic design volume conditions defined in this chapter. Assumed volume distributions for the 1100 vehicle per hour exit ramp flow are presented in Figure 5-8. Frontage road, U-turn and lane distribution were selected as being representative of high volume conditions. Any other exit ramp volume (including 690) is assumed to have the same percentage distribution of traffic movements as does the 1100 volume level. Volumes would be scaled to a lower or higher level than that shown in Figure 5-8 depending on how the exit ramp volume compared to 1100. These volumes would then be used to determine the total weaving volumes (Table 5-1) and resulting required weaving distances.

In order to define trade-off options between freeway level of service and frontage road operating conditions, exit ramp design levels of performance were defined as: 1) desirable, 2) usual minimum, and 3) absolute minimum. While these design levels are not defined specifically in terms of equivalent levels of service, they represent approximately Levels of Service C, D and E, respectively. Design criteria selected for the model variables are presented in Table 5-2. These variables include quality of weaving, safe approach speed for stopping, perception-reaction time and signalized intersection cycle length. The values selected define reasonable "desirable" conditions for operations but certainly not ideal conditions. Design Level No. 3 is an absolute minimum or "capacity" level. Design at absolute minimum conditions is not recommended.

Exit Ramp Spacings

Exit ramp spacings calculated by the model for total frontage road volumes ranging from 200 to 2000 vehicles per hour are presented in Table 5-3 for the three previously defined design levels. Values of the design criteria used in

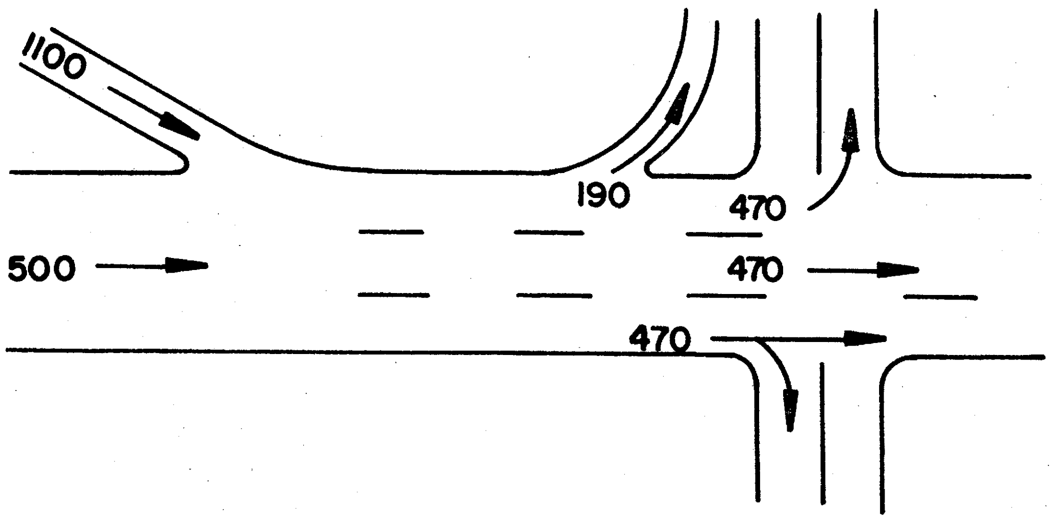


Figure 5-8. Volume Distribution For Exit Ramp Design Example Problem

TABLE 5-2. EXIT RAMP DESIGN CRITERIA FOR THREE DESIGN LEVELS

Design Criteria	Design Level #1 Desirable Design	Design Level #2 Usual Minimum	Design Level #3 Absolute Minimum
Operating Speed, mph	35-40	30-35	20
Weaving Quality	III-IV	IV	V
Weaving Volume, %	63	63	63
Perception-Reaction, sec.	2.50	1.75	1.00
Stopping Distance, feet	275	175	75
Cycle Length, sec.	90	80	70
Signal Saturation, X	0.80	0.80	0.80
Maximum Lane Volume[a]	$F \cdot V_T$	$F \cdot V_T$	$F \cdot V_T$

[a] $F = 0.4 + 0.6e^{-.13 \frac{V_T C}{3600}}$

V_T = total frontage road approach volume.

TABLE 5-3. EXIT RAMP TO CROSS-STREET SEPARATION DISTANCE REQUIREMENTS IN FEET FOR DIFFERENT DESIGN LEVELS

Total[a] Frontage Road Volume V.P.H.	Approximate[b] Exit Ramp Volume V.P.H.	Design Level[c]		
		#1	#2	#3
		Desirable	Usual Minimum	Absolute Minimum
200	140	500	380	260
400	275	560	460	360
600	410	630	500	400
800	550	690	540	430
1000	690	760	590	450
1200	830	870	640	480
1400	960	970	690	500
1600	1100	1070	770	530
1800	1240	1180	860	550
2000	1380	1300	970	580

[a] Exit ramp volume plus existing frontage road volume.

[b] Exit ramp volume assumed to be 69 percent of total volume.

[c] See Table 5-2 for assumed design criteria.

the model were given in Table 5-2. Distances are from the exit ramp centerline merge point with the frontage road to the stop line at the signalized intersection. This distance may be 100-200 feet less than the actual distance from the physical nose of the exit ramp to the intersection due to the exit ramp entry angle (4^{0}) and due to any pedestrian crosswalk requirements at the intersection.

Exit Ramp Summary

Experience has shown that exit ramps may experience operational blockages at their merge point with frontage roads due to queue spillback from the adjacent signalized intersection. It is highly probable that a considerable number of exit ramps experience this type of problem, and these ramps should be redesigned to provide more frontage road spacing. Table 5-3 can be used to guide the evaluation process. A "desirable" level of design should be provided where possible. Trade-off analyses could be made between the freeway and frontage road operations where providing a desirable exit ramp spacing would result in lowering the level of service on the freeway.

Careful consideration should be given before designing the spacing of exit ramps less than those required by the 600 vehicle per hour total volume level in Table 5-3. Planning data and projected volumes are based on numerous assumptions and estimations of future events, and consequently exit ramp volume projections may be in considerable error. Likewise, exit ramps which are expected to feed adjacent major arterials or major traffic generators probably should not be designed for a volume level less than 1600 vehicles per hour total frontage road volume design guidelines presented in Table 5-3.

Entrance Ramp Spacing

Entrance Ramp Model

Ramp metering systems are becoming an accepted practice on urban freeways today. This fact has caused concern about the spacing provided between diamond interchanges and the entrance ramp merge point to the freeway main lanes. Queues form at these metered ramps and sometimes back into the cross-street intersections, as shown in Figure 5-2. The number of vehicles stored behind the ramp signal over a period of time depends on the ramp demand volume and the operating capacity of the ramp metering signal.

With the assumption of Poisson arrivals to the ramp and Poisson departures from the ramp metering signal, Morse (5) shows that the probability of a ramp of a known queue storage, N, overflowing is given by

$$\text{Probability of Overflow} = \left(\frac{\text{volume}}{\text{capacity}}\right)^{N+1}$$

Results of this model are presented in Figure 5-9 for volume-to-capacity ratios (V/C) of the ramp metering signal of 0.80, 0.90 and 0.95. The higher the V/C ratio, the longer the ramp storage required for a given probability of queue overflow. For a given ramp demand volume level, the required ramp storage increases as the freeway level of service decreases.

From a theoretical viewpoint, the queue length distribution over time and space can be determined if the ramp demand volume and metering capacity are known. Few studies have been published relating ramp metering capacity to freeway lane number one (outside lane) volumes. Brewer et. al., however, developed a theoretical model of merging control operations which was later validated in Houston (6). An approximation of this model is

$$C_r = 1620 - 0.81 \cdot V_1$$

where

C_r = Capacity of metered entrance ramp, vehicles per hour

V_1 = Lane No. 1 (outside) freeway lane volume, vehicles per hour

The normal ramp capacity theoretically cannot be less than the minimum acceptable cycle length if one car per cycle is metered onto the freeway. Cycle lengths should not be less than 4.0 to 4.5 seconds (900 to 800 vehicles per hour). Ramp volumes in excess of 800 vehicles per hour usually experience high violation rates and multiple vehicle entries during the green.

The volume existing on the outside lane of the freeway (lane 1) varies with the total freeway flow. An estimate of the volume in lane 1 can be determined from Figure 5-10 from the SDHPT design manual (7). Using level of service criteria given in the design manual, a range of Level of Service "D" lane 1 volumes and resulting ramp metering capacities were developed. From these results, required ramp vehicle storages were calculated at 5% probability of overflow using Morse's equation previously described, as presented in Table 5-4. Level of Service "D" was selected since peak hour metering frequently

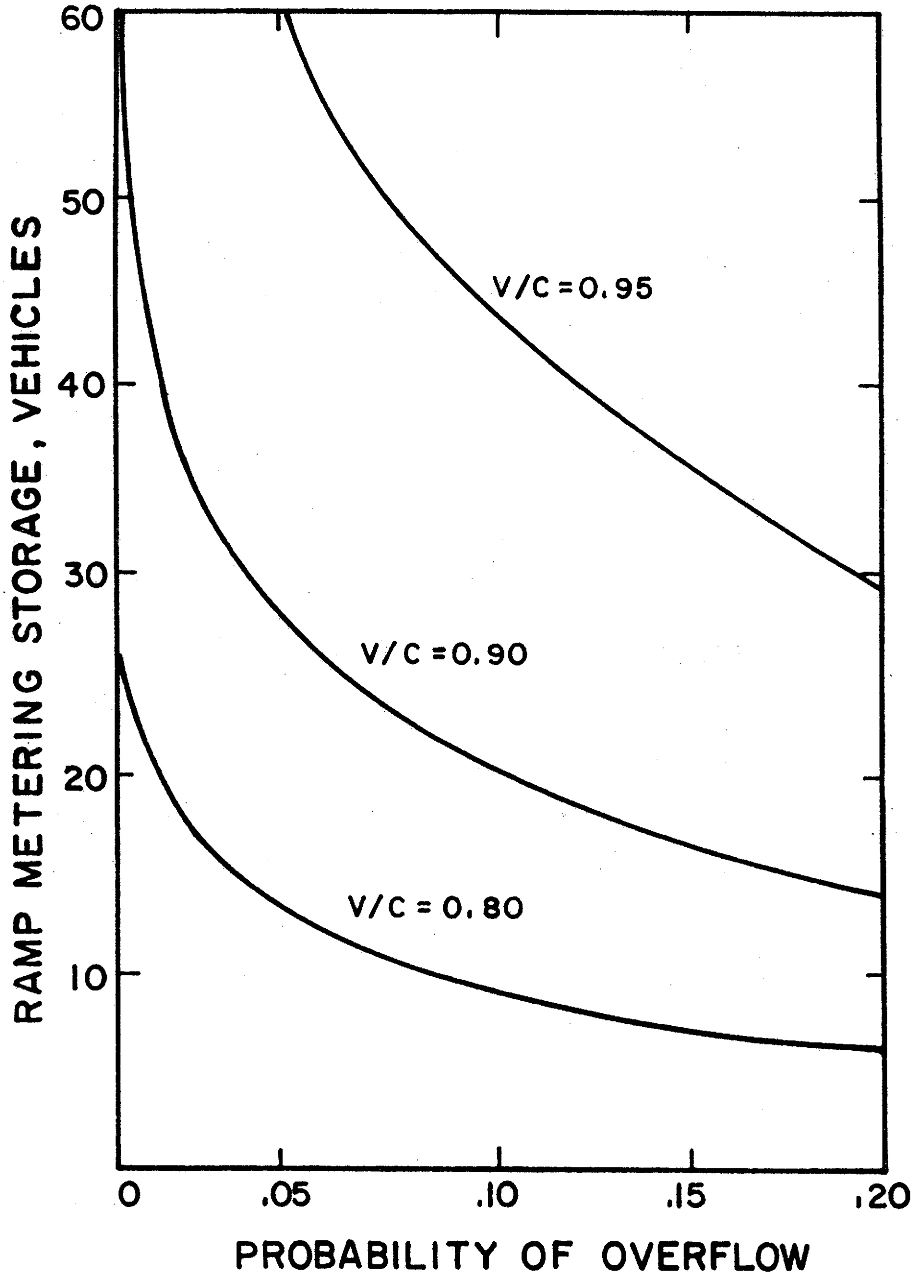


Figure 5-9. Probability of Ramp Storage Overflow as Related to Volume/Capacity Ratio of Ramp Metering

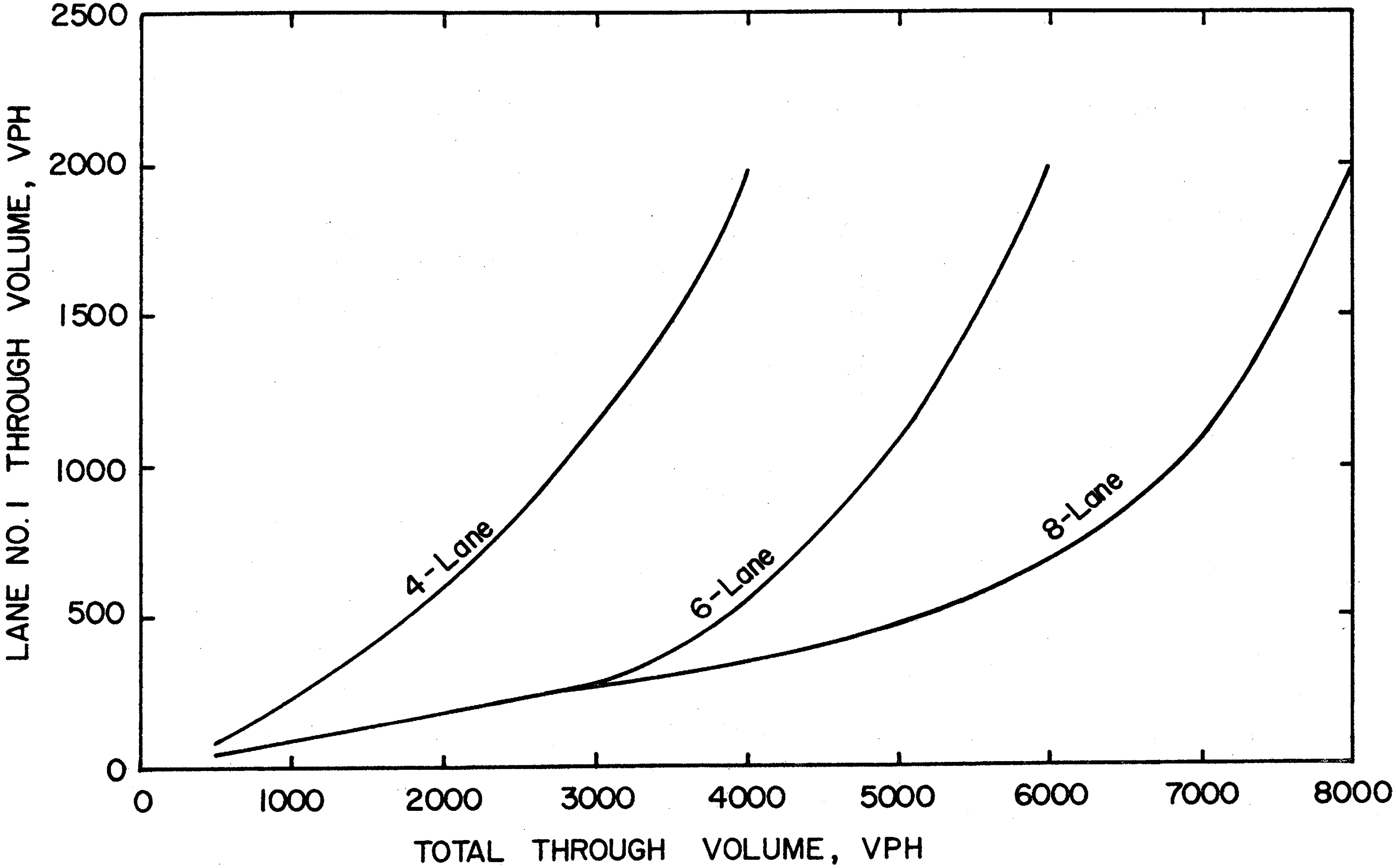


Figure 5-10. Approximate Volume of Through Traffic in Lane No. 1 in the Vicinity of Ramp Gores

TABLE 5-4. CALCULATED VEHICLE STORAGE NEEDED FOR GIVEN RAMP VOLUMES FOR 5% OVERFLOW FOR VARIOUS FREEWAY LEVELS OF SERVICE "D"

	Freeway Level of Service "D" Near "C"	Mid "D"	Near "E"
Lane No. 1 Volume =	1000	1200	1400
Ramp Metering Capacity =	810	648	486
Ramp Demand Volume	Number of Vehicles in Queue		
300	2	3	6
400	4	6	15
500	5	11	*
600	7	38	*
700	11	*	*
800	25	*	*

* Large Queue. Cannot be calculated by theory.

operates within this high volume stable flow region.

Entrance Ramp Studies

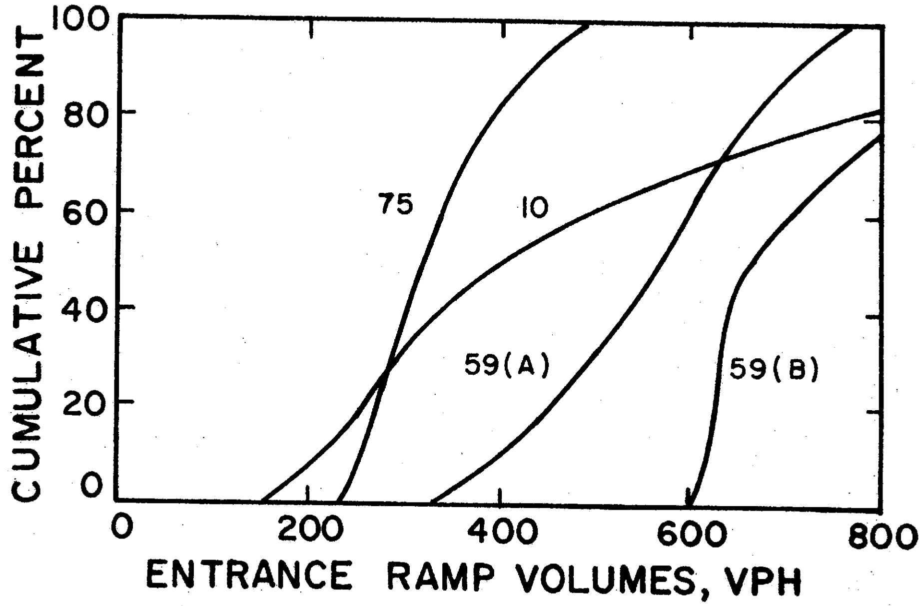
Several studies of geometric and operational characteristics were conducted to assist in the establishment of model parameters, to confirm the realism of the model results, and to provide field data to objectively evaluate existing freeway geometrics. Additional data were obtained from a concurrent study being conducted in Houston by SDHPT and TTI personnel.

Entrance ramp volume data were collected for 21 metered entrance ramps along U.S. 75 (North Central Expressway) in Dallas during the peak hour for two days during April, 1976. A cumulative frequency plot (converted to percent) is presented in the top illustration of Figure 5-11 as curve 75. Most metered ramp volumes ranged from 250 to 400 vehicles per hour. The maximum observed ramp volume was 510 vehicles per hour. No connecting roadways from interchanges to the freeway were included in this sample. All ramps were also on continuous frontage road sections.

Metered entrance ramp volume studies were also conducted along U.S. 59 (Southwest Freeway) in Houston during the Spring of 1976. Four high volume ramps, Bellaire, Westpark, Chimney Rock, and Enloe, were initially observed to study high volume and delay conditions. These ramps are in an area of southwest Houston beyond loop I-610 near a large shopping center complex and most do not have frontage roads or other convenient alternate freeway routes. Cumulative percent plots of ramp volume data taken during the peak hour are presented in the top illustration of Figure 5-11 as curves 59 (A). Most ramp volumes are between 450 and 650 vehicles per hour; two-thirds are above 500 vehicles per hour.

Another set of entrance ramps along U.S. 59 inside of loop I-610 were studied. Four ramps near the Summit arena, inbound and outbound Buffalo Speedway together with outbound Shepherd and Kirby, were counted. The ramps are in a continuous frontage road section of a eight-lane freeway section of U.S. 59. Curve 59 (B) in Figure 5-11 shows that high volumes occur on these ramps.

A fourth set of ramp volume data was obtained from a study being conducted by SDHPT along I-10 (Katy Freeway) in Houston between the outer loop (I-610) and West Belt. A total of 16 entrance ramps were counted during the peak hour in the inbound or outbound direction. These average volume results are also depicted in Figure 5-11 as curve 10. Notice that the curve is much flatter having a wider range of volumes with about 20 percent exceeding 800 vehicles per hour. These ramps were not metered. The capacity of a metered ramp will seldom exceed



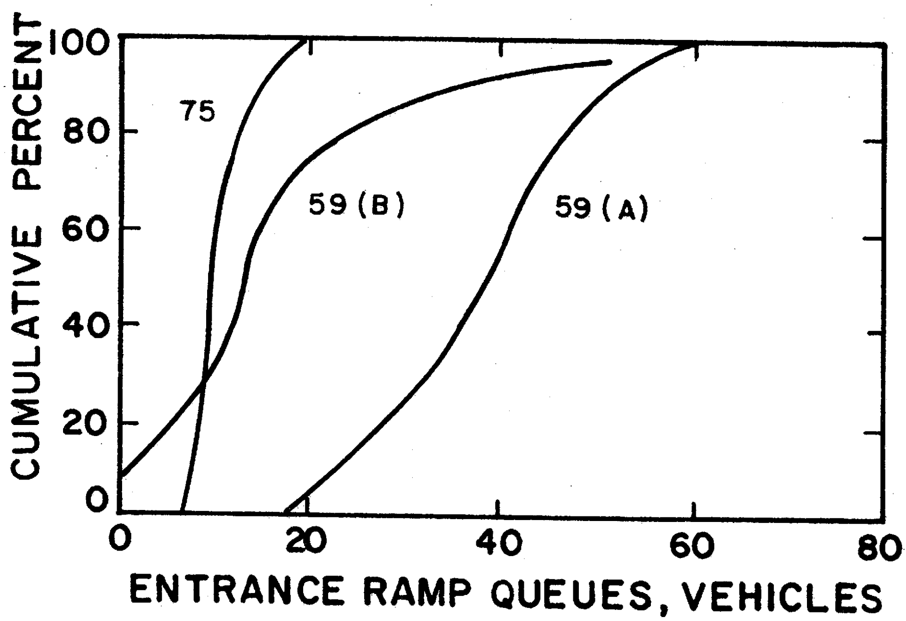


Figure 5-11. Entrance Ramp Volumes and Queues For Ramps in Dallas and Houston

800 vehicles per hour. Metered ramp volumes tend to balance the traffic demand over a section of freeway, thereby reducing "hot spots" or local pockets of high density flow.

Queue count studies at metered entrance ramps were also conducted along U.S. 75 (North Central Expressway) in Dallas and U.S. 59 (Southwest Freeway) in Houston. These peak hour results are presented in the lower illustration of Figure 5-11. Most queues observed along U.S. 75 in Dallas ranged from 5 to 15 vehicles although none of the 21 ramps in the continuous frontage road sections studied operate at what might be considered high volumes. Some of the lower volume ramp queue counts were not included in this data set. As mentioned previously, the ramp metering combined with continuous frontage roads and progressive operations tend to balance out the ramp loadings. Motorists were frequently observed to divert from joining a ramp queue when it exceeded 8-10 vehicles.

High-volume conditions along the metered section of U.S. 59 in southwest Houston, beyond loop I-610 near Sharpstown Shopping Center, produced some nearly unbelievable results. Queue counts between 40 and 60 were noted on most of the four ramps, often for most of the entire rush hour. Three days of counts were made which were considered typical days by the resident TTI staff in Houston. No good alternate freeway routes were available, however. The results of this queue count study are depicted as curve 59 (A) in the lower illustration of Figure 5-11.

Another controlled section of U.S. 59 inside loop I-610 near Greenway Plaza and the Summit which has frontage roads was studied. This "more typical" urban freeway section included the outbound ramps at Buffalo Speedway, Shepard, and Kirby, and the inbound ramp at Buffalo Speedway. These results are shown as curve 59 (B) in Figure 5-11. Most of the observed queues on these four metered ramps ranged between 10 and 15 vehicles.

Referring to Table 5-4 and Figure 5-11, ramp volumes in the 300-400 vehicle per hour range in Table 5-4 would be expected to experience queue lengths in the 5-15 vehicle range during the peak hour as did the U.S. 75 ramps in Figure 5-11. Ramp volumes in the 400-600 range normally would be expected to operate with moderately long queues from 6 to 38 vehicles at mid-Level of Service D (Table 5-4). This rather wide queueing range is illustrated by the higher volume results of curve 75 in Figure 5-11 and the lower volume queues of curve 59 (A). Ramp volumes above 500 vehicles per hour may experience large ramp queues, as did

U.S. 59 (A), with some queues frequently exceeding 50 vehicles when the freeway is operating near or at Level of Service E.

On the other hand, ramp volumes in the 600-800 range may have only moderate queues, as did U.S. 59 (B), if the freeway level of service is operating near "C" at the metered ramp.

Based on the results of this study, it appears that the number of vehicles in a ramp queue varies primarily with the operating level of service on the freeway, the ramp demand volume, and whether continuous frontage roads (with frontage road progression) are available.

Entrance Ramp Design Criteria

It is recommended that the design of entrance ramps in urban areas provide adequate spacing between the adjacent diamond interchange and the entrance ramp merge point on the freeway such that ramp metering can be installed and operated without queues being likely to overflow into the adjacent interchange.

There are basically only two parts required in determining spacing requirements - the metering section and the queue storage. The first part of the ramp design must provide adequate distance between the ramp signal and the merge point to permit adequate distance to accelerate to a reasonable merge speed and select a gap if available. Everall, in an FHWA report (8), indicates that 200 to 250 feet is required to provide adequate time to merge. Ramp metering design in Dallas and Houston supports these guidelines. However, 200 feet should be considered a minimum distance to the merge point and 250 desirable.

Recent research in Texas has shown that vehicles store at about 25-foot intervals behind traffic signals (4). Thus, the queue storage needs previously discussed multiplied by 25 feet/car and added to the 200-250 required from the ramp signal to the merge point determine required ramp spacings in feet.

Entrance Ramp Spacings

Recommended entrance ramp spacing design requirements are presented in Table 5-5. Desirable and minimum spacings were selected based on the previous study results and the considered judgment of the researchers.

A comparison can be made between the recommended design spacings of Table 5-5 and existing entrance ramp spacings determined for two of the freeways previously described. These spacings for U.S. 75 in Dallas and U.S. 59 near the Summit arena are shown in Figure 5-12. In general, minimum ramp spacings are being provided by the two current freeway designs although some ramps may be deficient. An individual ramp volume and spacing analysis of the U.S. 75 data

TABLE 5-5. RECOMMENDED ENTRANCE RAMP SPACING DESIGN REQUIREMENTS FOR URBAN FREEWAYS

Ramp Demand Volume, Veh./Hr.	Desirable Spacing, Feet	Minimum Spacing, Feet
300 or less	750'	450'
400	1000'	575'
500	1250'	700'
600	1500'	825'
700	1750'	950'
800 or more	2000'	1075'

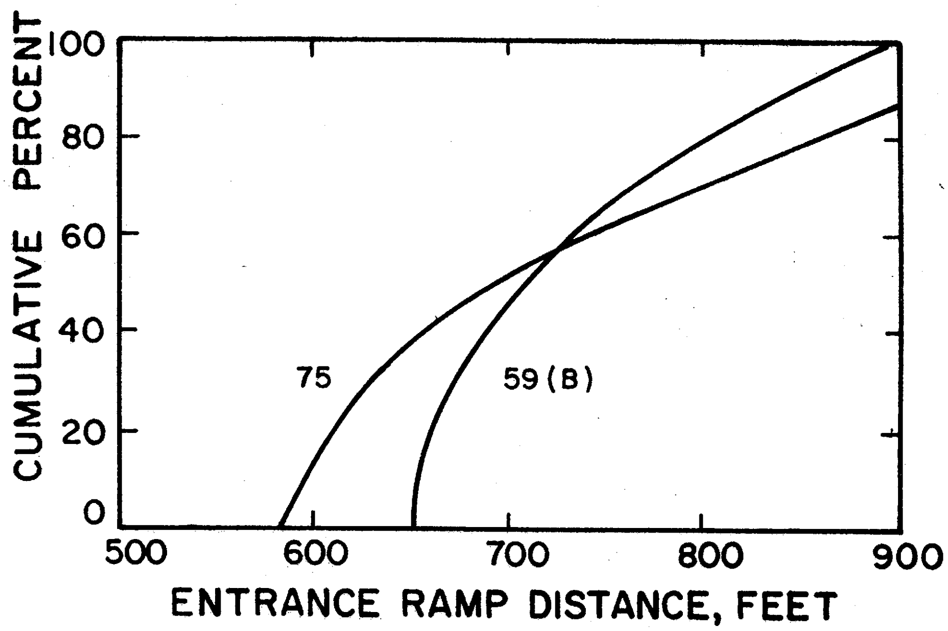


Figure 5-12. Entrance Ramp Distances in Cumulative Percent for U.S. 75 (Dallas) and U.S. 59 (Houston)

shows that 55% of the entrance ramps do not meet the desirable spacing criteria. All U.S. 59 (B) ramps studied fail to meet the desirable spacing criteria due to the high volume levels experienced. It was observed in the U.S. 75 data set of ramp volumes versus spacing (not shown) that no correlation existed between ramp volumes and ramp spacing provided. That is, a low volume ramp was just as likely to have a long spacing as a higher volume ramp.

Summary

The design of entrance ramps in urban areas should consider the possibility of entrance ramp metering being installed. Adequate ramp spacings are required between the adjacent diamond interchanges and the entrance ramp merge point to the freeway to insure smooth ramp metering operations and little queue overflow into the interchange. Minimum and desirable ramp separation distances have been presented which should be considered in future design work. An investigation should be conducted of the current adequacy of all entrance ramps in urban centers in Texas to evaluate the potential need to redesign those ramps with deficient spacings.

REFERENCES

1. A Policy on Geometric Design of Rural Highways. 1965. American Association of State Highway Officials, Washington, D.C., 1965.
2. Leisch, J.E. Spacings of Interchanges on Freeways in Urban Areas. American Society of Civil Engineers 1961 Transaction, Vol. 126, Part IV.
3. Highway Capacity Manual 1965. Highway Research Board Special Report 87, 1965.
4. Messer, C.J., Fambro, D.B. and Andersen, D.A. A Study of the Effects of Design and Operational Performance of Signal Systems Final Report. Texas Transportation Institute Research Report 203-2F, August 1975.
5. Morse, P.M. Queues, Inventories and Maintenance. Operations Research Society of America. Publications in Operations Research Number 1, 1967.
6. Brewer, K.A., Buhr, J.H., Drew, D.R. and Messer, C.J. Ramp Capacity and Service Volume as Related to Freeway Control. Highway Research Record 279, 1969.
7. Highway Design Division Operations and Procedures Manual, Part IV, Texas Highway Department, 1970.
8. Everall, P.F. Urban Freeway Surveillance and Control: The State of the

Art, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1972.

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```
      INTEGER AVSP,DIST(2,17)
      DIMENSION LABL(2,3),ISTAC(2),IFRST(2),ISTA(2,17)
      DIMENSION TTS(17),TTM(17)
      DIMENSION LABLN(3),LABLS(3),ISTAN(17),ISTAS(17),IDSTN(17),
     XIDSTS(17)
      DATA LABLN/' N','OR','TH'/
      DATA LABLS/' S','OU','TH'/
C DEFINE NO. SPEED STATIONS NORTH/SOUTHBOUND
      DATA ISTAC/16,17/
C DEFINE BEGINNING SPEED STATION NUMBER
      DATA IFRST/2,37/
C DEFINE SPEED STATIONS
      DATA ISTAN/4,6,8,10,41,12,14,16,18,22,24,26,28,30,32,34,0/
      DATA ISTAS/35,33,31,29,27,25,21,19,17,15,13,11,9,7,5,3,1/
C DEFINE SPEED STATION DISTANCES
      DATA IDSTN/1030,2880,2490,3470,2450,1060,3680,2050,2410,2600,1601,
     X2780,3510,3460,4000,6330,0/
      DATA IDSTS/9360,3940,4100,2750,2880,2122,1910,2455,1990,1180,2380,
     X1140,2870,1310,1680,2640,3840/
C TRANSFER ONE DIMENSIONAL TO TWO DIMENSIONAL ARRAYS
      DO 55 J=1,3
      LABL(1,J)=LABLN(J)
      LABL(2,J)=LABLS(J)
   55 CONTINUE
      DO 65 J=1,17
      ISTA(1,J)=ISTAN(J)
      ISTA(2,J)=ISTAS(J)
      DIST(1,J)=IDSTN(J)
      DIST(2,J)=IDSTS(J)
   65 CONTINUE
C GET CURRENT DATE
      CALL CNVDT(IM,ID,IY)
      IY=IY-1900
C DEFINE MPH FACTOR
      FMPH=3600./5280.
C REQUEST DECECTOR OUTTAGE SUBSTITUTIONS
    2 WRITE(9,110)
  110 FORMAT('ENTER SUBSTITUTION DETECTOR OONN')
      READ(8,120) NOLD,NNEW
  120 FORMAT(2I2)
C DISCONTINUE WHEN ZERO
      IF(NOLD+NNEW) 3,4,3
C UPDATE DETECTOR TABLE
    3 DO 45 I=1,2
      LIM=ISTAC(L)
      DO 45 J=1,LIM
      IF(ISTA(I,J)-NOLD) 45,1,45
    1 ISTA(I,J)=NNEW
      GO TO 2
   45 CONTINUE
      GO TO 2
    4 DO 35 L=1,2
C DEFINE SPEED STATION COUNT NORTHBOUND(L=1) OR SOUTHBOUND(L=2)
      LIM=ISTAC(L)
```

```
C INITIALIZE TRAVEL TIME SUMMING COUNTER
      DO 25 I=1,LIM
      TTS(I)=0.
   25 CONTINUE
C REQUEST KEYBOARD ENTRY OF STARTING TIME
      WRITE(9,10) (LABL(L,J),J=1,3)
   10 FORMAT('ENTER STARTING TIME HHMM'3A2'BOUND')
      READ(8,80) ITIME
   80 FORMAT(I4)
C CONVERT MILITARY TIME TO LINEAR MINUTE
      LINM=(ITIME/100)*60+ITIME-(ITIME/100)*100
C SAVE BEGINNING TIME FOR UPDATE
      TIMES=LINM
C TEMPORARILY REPORT ALL SPEED STATION DATA
      CALL RSPED(LINM)
C START NEW PAGE W/HEADER
      WRITE(3,20) (LABL(L,I),I=1,3),IM,ID,IY,ITIME,LINM
   20 FORMAT('1DALLAS NORTH CENTRAL EXPRESSWAY'3A2'BOUND TRAVEL TIMES'
     X10X,I2'/'I2'/'I2,10X'STARTING TIME ='I5,1X,'('I4')'/
     X14X,'FPS    TTM'/' ST DIST   MIN MPH    TTS')
C GET FIRST LINK'S SPEED IN FPS(K) AND MPH(KK)
      K=AVSP(IFRST(L),LINM)
      KK=FLOAT(K)*FMPH
C LIST FIRST SPEED STATION W/SPEED IN FPS
      WRITE(3,90) IFRST(L),LINM,K
   90 FORMAT('0'I2,I10,I4)
C REPORT MPH AT FIRST SPEED STATION
      WRITE(3,100) KK
  100 FORMAT(I17)
C BEGIN SUMMING LOOP FOR SPECIFIC DIRECTION
      DO 5 I=1,LIM
C DETERMINE TRAVEL TIME IN SECONDS TO THIS SPEED STATION, BASED ON SPEED
C AT PREVIOUS STATION
      TTSA=FLOAT(DIST(L,I))/FLOAT(K)
C SUM TOTAL TRAVEL TIME FROM ALL STATIONS UP TO THIS POINT
      DO 15 J=1,I
      TTS(J)=TTS(J)+TTSA
      TTM(J)=TTS(J)/60.
   15 CONTINUE
C UPDATE LINEAR MINUTE ACCORDING TO ARRIVAL TIME FROM BEGINNING SPEED STATION
      LINM=TIMES+TTM(I)+0.5
C FETCH THIS STATION'S 1 MINUTE AVERAGE SPEED IN FPS(K) AND MPH(KK)
      K=AVSP(ISTA(L,I),LINM)
      KK=FLOAT(K)*FMPH
C REPORT INTERMEDIATE TRAVEL TIMES IN MINUTES
      WRITE(3,30) ISTA(L,I),DIST(L,I),LINM,K,(TTM(J),J=1,I)
   30 FORMAT('0'I2,I5,I5,I4,17F6.2)
C REPORT INTERMEDIATE TRAVEL TIMES IN SECONDS
      WRITE(3,40) KK,(TTS(J),J=1,I)
   40 FORMAT( I17,17F6.0)
    5 CONTINUE
   35 CONTINUE
      WRITE(3,70)
   70 FORMAT('1')
```

```
      SUBROUTINE RSPED(LINM)
C SUBROUTINE TO TEMPORARILY REPORT ALL SPEED STATION DATA FROM BEGINNING
C MINUTE (LINM) TO 22 MINUTES HENCE
C
      INTEGER AVSP
      DIMENSION ISPD(41,23)
C DEFINE LAST MINUTE
      K=LINM+22
C FETCH ALL SPEEDS
      DO 5 J=1,23
      DO 5 I=1,41
      ISPD(I,J)=AVSP(I,J+LINM-1)
    5 CONTINUE
C HEAD NEW PAGE
      WRITE(3,10) (J,J=LINM,K)
   10 FORMAT('1SPEED TABLE DATA(FPS)'/' ST'23I5)
C OUTPUT TABLE
      WRITE(3,20) (I,(ISPD(I,J),J=1,23),I=1,41)
   20 FORMAT(I3,23I5)
      RETURN
      END
```

DALLAS NORTH CENTRAL EXPRESSWAY NORTHBOUND TRAVEL TIMES

ST	DIST	MIN	FPS MPH	TTM TTS							
2		540	63								
			42								
4	1030	540	66	0.27							
			44	16.							
6	2880	541	77	0.99	0.72						
			52	59.	43.						
8	2490	542	79	1.53	1.26	0.53					
			53	92.	75.	32.					
10	3470	542	63	2.27	1.99	1.27	0.73				
			42	136.	119.	76.	43.				
41	2450	543	70	2.91	2.64	1.91	1.38	0.64			
			47	175.	158.	115.	82.	38.			
12	1060	543	62	3.17	2.89	2.17	1.63	0.90	0.25		
			42	190.	173.	130.	97.	54.	15.		
14	3680	544	61	4.16	3.88	3.16	2.62	1.88	1.24	0.98	
			41	249.	233.	189.	157.	113.	74.	59.	
16	2050	545	65	4.72	4.44	3.72	3.18	2.44	1.80	1.54	0.56
			44	283.	266.	223.	190.	146.	108.	92.	33.