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16. Abstract

This report describes the Applications Guide for the microcomputer version of PASSER III-88, a practical computer program designed to assist transportation engineering professionals in the analysis of pretimed or traffic-responsive, fixed-sequence signalized diamond interchanges. The program can evaluate existing or proposed signalization strategies, determine signalization strategies which minimize the average delay per vehicle, and calculate signal timing plans for interconnecting a series of interchanges along continuous one-way frontage roads. In addition, the program can evaluate the effectiveness of various geometric design alternatives, e.g., lane configurations, U-turn lanes, and channelization.

The report describes procedures for applying the program to "real world" diamond interchange operational problems. Procedures for evaluating existing conditions; optimizing phase sequences, green splits, offsets, and cycle lengths; and converting the program's output to controller settings are presented. These procedures provide a consistent approach to diamond interchange analysis. Application of these procedures in conjunction with PASSER III-88 will enable users to evaluate a greater number of alternatives and be more confident in the efficiency of the resultant solution.

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APPLICATION GUIDE FOR THE MICROCOMPUTER VERSION OF PASSER III-88

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

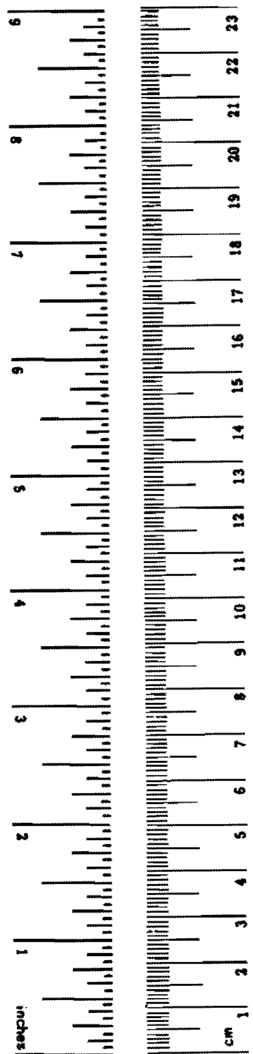
VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac

MASS (weight)

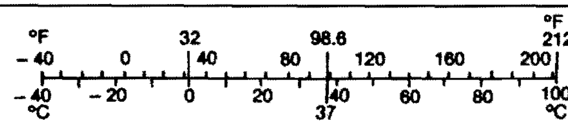
Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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* SI is the symbol for the International System of Measurements

ABSTRACT

This report describes the Applications Guide for the microcomputer version of PASSER III-88, a practical computer program designed to assist transportation engineering professionals in the analysis of pretimed or traffic-responsive, fixed-sequence signalized diamond interchanges. The program can evaluate existing or proposed signalization strategies, determine signalization strategies which minimize the average delay per vehicle, and calculate signal timing plans for interconnecting a series of interchanges along continuous one-way frontage roads. In addition, the program can evaluate the effectiveness of various geometric design alternatives, e.g., lane configurations, U-turn lanes, and channelization.

The report describes procedures for applying the program to "real world" diamond interchange operational problems. Procedures for evaluating existing conditions; optimizing phase sequences, green splits, offsets, and cycle lengths; and converting the program's output to controller settings, are presented. These procedures provide a consistent approach to diamond interchange analysis. Application of these procedures and used PASSER III-88 will enable users to evaluate a greater number of alternatives and be more confident in the efficiency of the resultant solution.

EXECUTIVE SUMMARY

With increasing demands on the urban freeways in Texas, frontage roads are becoming more important as a source of additional capacity for the freeway's main lanes. Additional capacity in the freeway corridor is especially beneficial during rush hour, maintenance, or incident conditions It is essential, however, that the signalized intersections along the frontage roads operate efficiently in order to make the best use of the existing facilities. Toward this goal, several Texas HP&R (Highway Planning and Research) studies have addressed objectives related to improving frontage road-freeway design and operations. This report presents the applications guide for the microcomputer version of the diamond interchange signalization program, PASSER III-88, developed as a part of this research.

PASSER III-88 is a practical computer program designed to assist transportation engineering professionals in the analysis of pretimed or traffic-responsive, fixed-sequence signalized diamond interchanges. The program can evaluate existing or proposed signalization strategies, determine signalization strategies which minimize the average delay per vehicle, and calculate signal timing plans for interconnecting a series of interchanges along continuous one-way frontage roads. In addition, the program can evaluate the effectiveness of various geometric design alternatives, e.g., lane configurations, U-turn lanes, and channelization.

This report contains procedures for evaluation and optimization of "real-world" diamond interchange operational problems. Procedures for evaluating existing conditions; optimizing phase sequences, green splits, offsets, and cycle lengths; and converting the program's output to controller settings are presented. These procedures provide a consistent approach to diamond interchange analysis. Application of these procedures and use of the microcomputer version of PASSER III-88 will allow users to evaluate a larger number of alternatives and be more confident in the efficiency of the resultant solution.

Implementation

The findings of this study should be helpful to Texas State Department of Highways and Public Transportation traffic engineering professionals who plan, design, operate, and maintain signalized diamond interchanges. The microcomputer version of PASSER III-88 developed in this research will be available statewide to SDHPT engineers. Use of the program will result in improved geometrics and timing plans and will substantially reduce delay costs at the signalized diamond interchanges in Texas. The program's use will also improve the efficiency of the state's traffic engineering professionals in that the average turnaround time of the microcomputer version of the program is approximately 10 times faster than the previous mainframe versions of the program. Thus, state personnel should be able to analyze more alternatives in a shorter amount of time. Use of the applications guide will result in a more consistent approach to diamond interchange operation and allow the analyst to be more confident in the resultant solution.

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Disclaimer

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the Texas State Department of Highways and Public Transportation. This report does not constitute a standard, specification, or regulation.

Please be advised that no warranty is made by the Texas Department of Highways and Public Transportation, the Federal Highway Administration, or the Texas Transportation Institute as to the accuracy, completeness, reliability, usability, or suitability of the computer program and its associated data and documentation. No responsibility is assumed by the above parties for incorrect results and/or damages resulting form the use of the PASSER III-88 program package.

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I. INTRODUCTION

Background

PASSER III-88 (Progression Analysis and Signal System Evaluation Routine) is one of a series of signalization programs developed by the Texas Transportation Institute (TTI) for and in conjunction with the Texas Department of Highways and Public Transportation (SDHPT). It was designed to assist traffic engineers in analyzing pretimed or traffic-responsive, fixed-sequence signalized diamond interchanges. The program can evaluate existing or proposed signalization strategies, determine signalization strategies which minimize the average delay per vehicle, and calculate signal timing plans for interconnecting a series of interchanges on one-way frontage roads. In addition, the program can evaluate the effectiveness of various geometric design alternatives (e.g., lane configurations, u-turn lanes, and channelization).

The basic theory of the progression option of the program was developed and tested by TTI in the Dallas Corridor Project sponsored by the Federal Highway Administration and documented in a previous publication (1). PASSER was adapted for off-line processing and analysis purposes in Highway Planning and Research (HPR) Project 165, and a level-of-service evaluation for the approaches to an intersection was undertaken in HPR Project 203. Both projects were sponsored by the SDHPT, and their results have been documented in several reports (2, 3).

The optimization and evaluation portion of PASSER III was developed in HPR Project 178, and the first version of the program was released in August, 1977, (4, 5). Since that time, experience gained by SDHPT personnel and other users have resulted in several suggested modifications and/or improvements to the basic program. In response to these suggestions, enhanced versions of the program, PASSER III-80 and PASSER III-84, were released in 1980 and 1984 (6). The next step in this evolutionary process, a microcomputer version of the program, PASSER III-88, was released in 1988 (7). This report describes procedures for using PASSER III-88 to analyze typical planning, design, and operational problems at signalized diamond interchanges.

Problem Statement and Objectives

The typical diamond interchange has many unique characteristics that can seriously complicate its operation and efficiency. In particular, close intersection spacing and high turning volumes require special phase sequencing and efficient signal timing to minimize vehicular conflict. The possible consequences of poor timing at even moderate volume interchanges are long queues, excessive delays, and poor arterial coordination. Recognizing the infinite number of possible phase sequences and timing patterns, a computer program, PASSER III (5), was developed to automate the analysis process. This program uses a deterministic, macroscopic approach to evaluate each phase sequence and timing alternative. Because of these unique abilities to analyze signalized diamond interchanges, it is expected that the recently released microcomputer version of PASSER III-88 (2) will be an essential analysis tool for transportation engineers.

The objective of this paper is to present a methodology using PASSER III for optimizing and evaluating diamond interchange signal timing. Typical control strategies for diamond interchanges are presented to illustrate the complexity of the problem facing transportation engineers. This discussion is followed by "real-world" examples which illustrate an evaluation of existing conditions and the optimization of green splits, offsets, and cycle length at which the two signals should operate.

Organization

This report is divided into five sections. Section I describes the program's development and the report's objectives. Section II presents a methodology for evaluating the level of service at existing diamond interchanges. Section III presents methodologies for optimizing green splits, cycle length, and phasing alternatives. Conversion of the program's output to controller settings is discussed in Section IV, and Conclusions and Recommendations are presented in Section V.

II. EVALUATION

Diamond Interchange Control

Although there are many variations of diamond interchanges, (see Figure 1), approximately 75 percent of them are full diamonds with or without frontage roads (8). Signal control at diamond interchanges has traditionally been provided by either a 3-phase pretimed signal sequence in which both off-ramp/frontage roads are released simultaneously, (see Figure 2), or by two non-interconnected, full-actuated controllers with one controller at each intersection. The 4-phase, 2-overlap signal phase sequence (see Figure 2) developed by the Texas Transportation Institute in the late 1950's has been used to increase interchange capacity and reduce operational problems and delay under certain circumstances (9). Signal control at most diamond interchanges are typically variations and/or combinations of these two basic phasing sequences using pretimed or actuated controllers.

Pretimed controllers are appropriate where a limited number of traffic patterns are found and when these patterns repeat themselves on a daily basis. These controllers can be easily interconnected with adjacent signalized, controlled intersections. The basic phasing can be modified through changes in the split and offset if two pretimed controllers are used at the interchange (one at each cross street intersection) (10). Actuated controllers are appropriate where a large number of traffic patterns are required and these patterns vary greatly on daily basis. As they are not easily interconnected with adjacent signals, the primary usage of actuated controllers is at isolated diamond interchanges. The California Department of Transportation has developed a diamond interchange software program for the Model 170 controller unit which can provide either 3- or 4-phase actuated control strategies (11). Two standard NEMA full-actuated controller units also can be used to provide either 3- or 4-phase operation. The Texas Diamond Controller uses one NEMA full-actuated controller unit to provide both 3- and 4-phase operation at the same interchange (12). The change from one phasing operation to the other is made by time clock or by external traffic responsive logic.

Split, offset, and cycle length determinations are additional considerations at a diamond interchange. The two intersections at the interchange can either be timed separately to minimize intersection delay or timed together to maximize interchange progression. Neither method is universally better than the other (13), and each will probably result in different optimal cycle lengths (14). Left-turn lane requirements and type of protection (i.e., protected, protected/permitted, permitted) must also be considered. Recognizing this myriad of alternatives, the question arises: How do transportation engineers determine which strategy is most appropriate

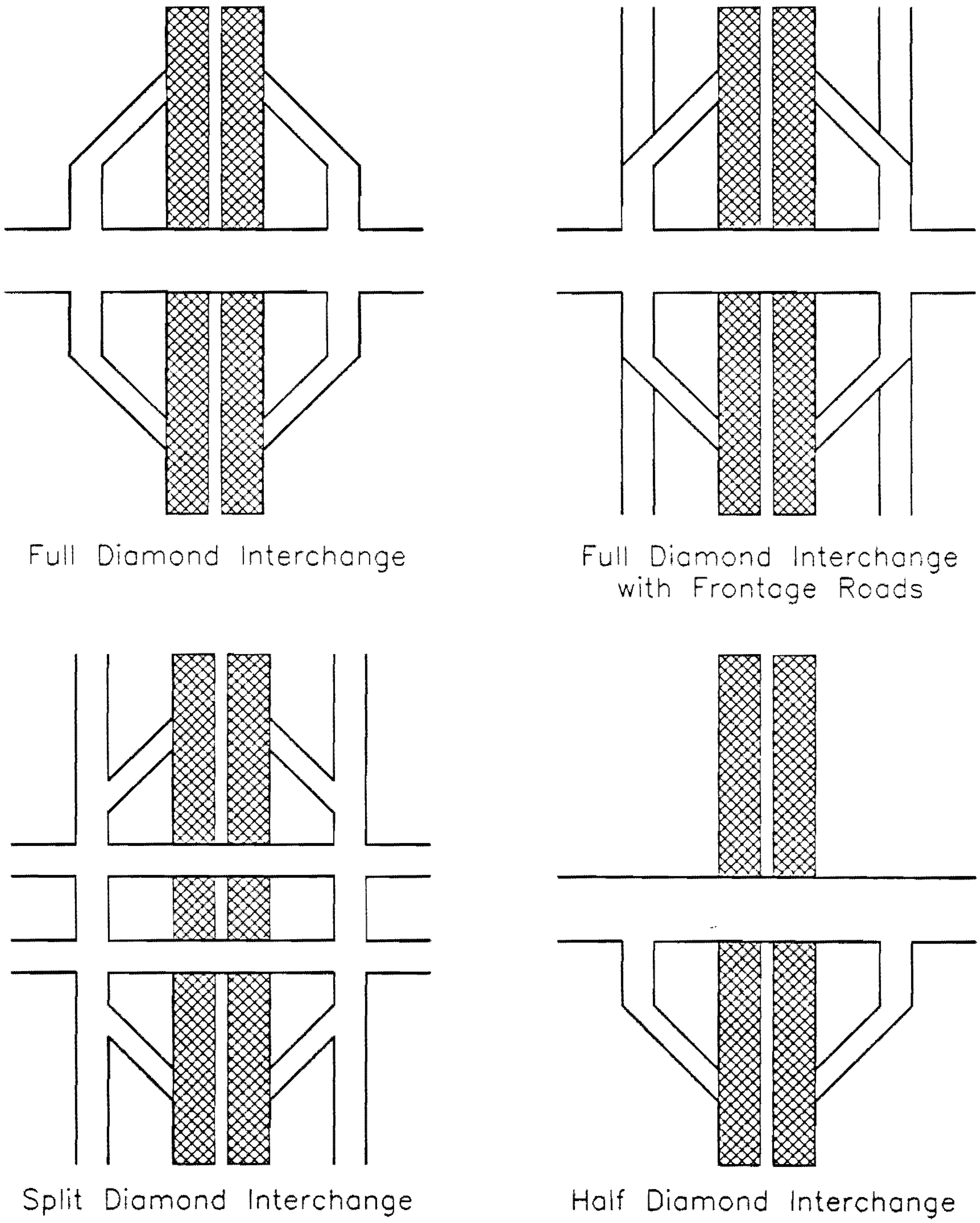


FIGURE 1. COMMON TYPES OF DIAMOND INTERCHANGES.

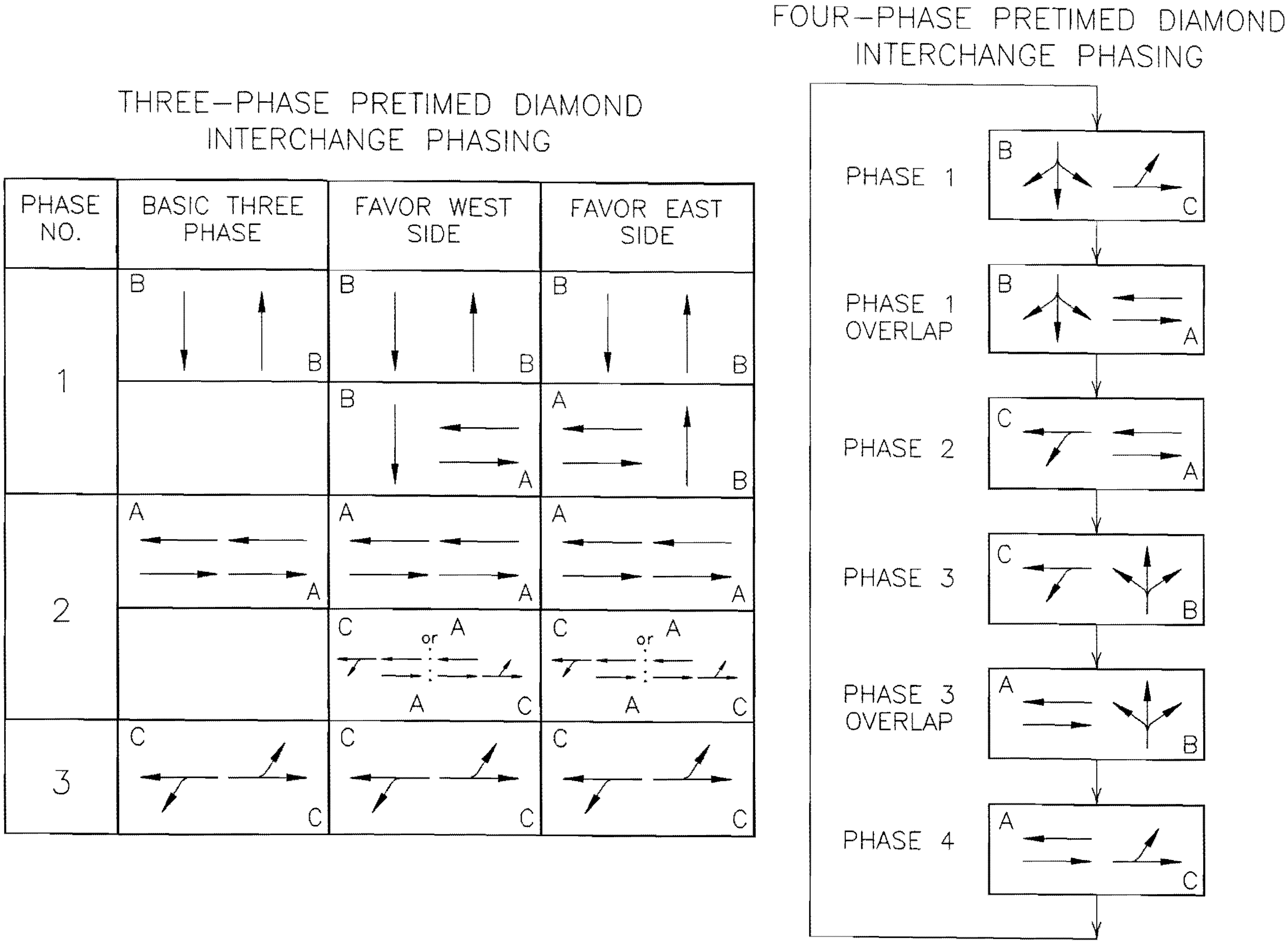


FIGURE 2. COMMON TYPES OF PRETIMED DIAMOND INTERCHANGE SIGNAL PHASING.

at any given signalized diamond interchange? One methodology for assessing the operational efficiency of a control strategy is described below:

a. Check that there is no spillback from one of the ramp intersections through the other intersection or from a left-turn lane back into a through lane. If either condition occurs, gridlock may occur and the control strategy is unacceptable.

b. Check that the queue of vehicles on the off-ramp does not back onto the freeway. If so, the control strategy is probably not acceptable.

c. Check that individual movements are not delayed disproportionately to one another. If so, the green splits need adjustment and/or geometric modifications are required.

d. Check that the overall level of service at the interchange is within acceptable limits. If not, cycle length, phasing sequence, controller type and/or geometric modifications may be appropriate.

Example Interchanges

To demonstrate the procedure for analyzing diamond interchanges, three example interchanges were considered. These interchanges, their traffic volumes, and their signal timings represent actual locations and conditions. Because of this, an element of realism is introduced into the results. It should be noted that all three of the interchanges were operating in the pretimed mode at the time data were collected.

The first example interchange demonstrates the analysis of a "compressed" diamond interchange without frontage roads. This type of interchange typically has only a minimum distance (less than 400 feet between center lines of the ramp terminals) separating the two ramp/cross road intersections and is commonly found in urban areas. The second example interchange demonstrates the analysis of a more conventional diamond interchange, also without frontage roads. This interchange has a relatively large separation between intersections and would typically be found in suburban or rural areas where right-of-way costs are minimal. The third example interchange demonstrates the analysis of a "compressed" diamond interchange with frontage roads. This type of interchange would typically be found on older urban freeways in Texas.

The first example interchange is at the intersection of Arrowood Road and I-77. As shown in Figure 3, Arrowood is a major arterial, having most of its traffic demand generated on the west

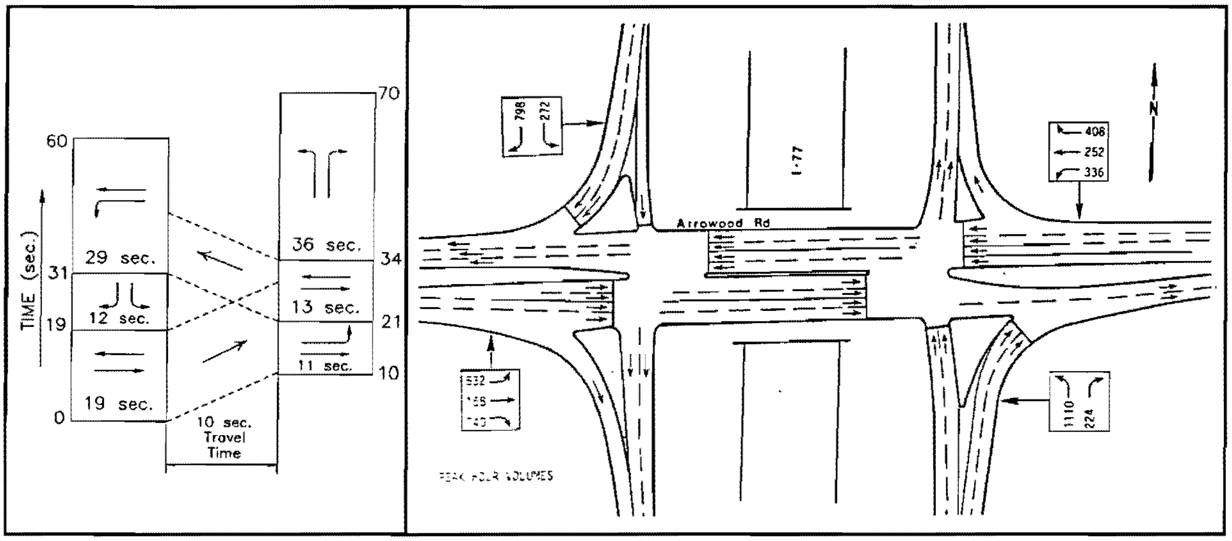


FIGURE 3. EXISTING SIGNAL TIMING, TRAFFIC VOLUMES, AND GEOMETRICS AT THE ARROWOOD AND I-77 INTERCHANGE.

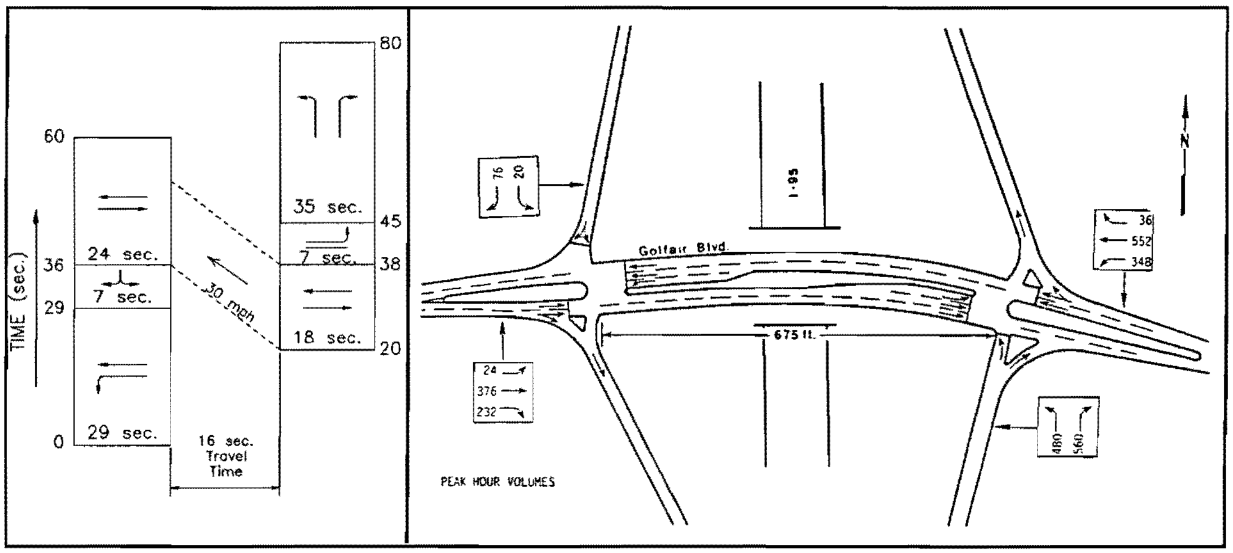


FIGURE 4. EXISTING SIGNAL TIMING, TRAFFIC VOLUMES, AND GEOMETRICS AT THE GOLFAIR AND I-95 INTERCHANGE.

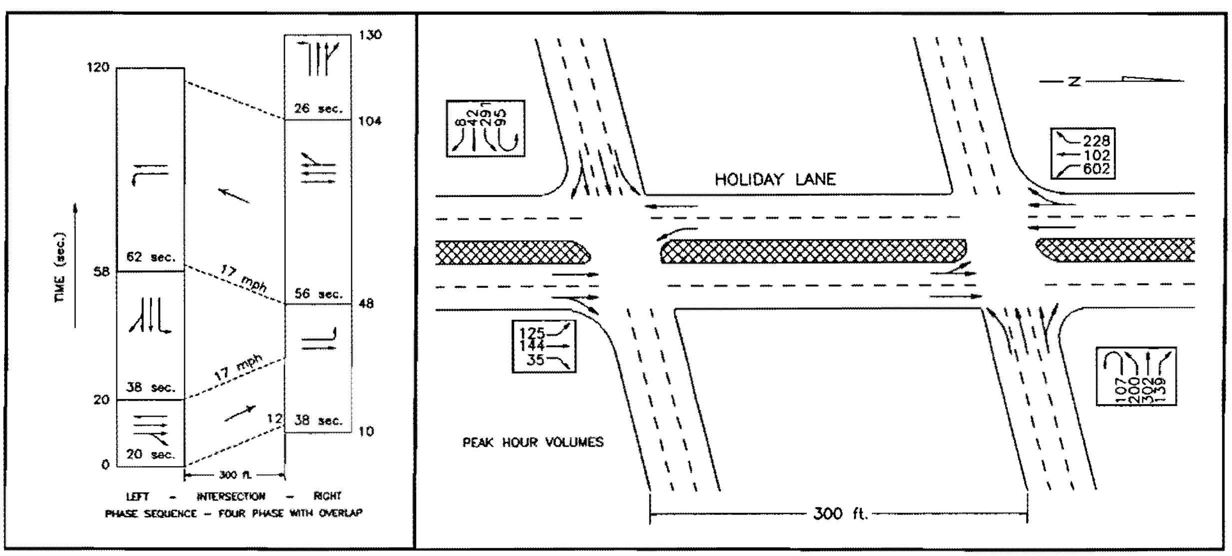


FIGURE 5. EXISTING SIGNAL TIMING, TRAFFIC VOLUMES, AND GEOMETRICS AT THE HOLIDAY LANE AND I-820 INTERCHANGE.

side of the interchange. The signal timing diagram, also shown in Figure 3, identifies the phase sequence and duration at both intersections. The phase sequence shown illustrates the 4-phase, 2-overlap, signal-phase sequence that was discussed previously. At this interchange, a 275-foot separation suggests a 12-second travel time between intersections. The existing timing plan reflects this travel time by using a fixed, 10-second offset in both directions, i.e., the downstream signal will change to green 2 seconds prior to the platoon's arrival.

The second example interchange is at the intersection of Golfair Boulevard and I-95. This interchange, shown in Figure 4, has a much larger separation than the one at Arrowood and I-77. Golfair has relatively light traffic volumes and would be best described as a minor arterial. The signal timing strategy shown would be described as having lagging left-turn movements. Because of heavier traffic demand in the westbound direction, one-way progression has been incorporated into the timing plan. Using a progression speed of 30 miles per hour and given the 675-foot separation distance, the offset from east to west was set at 16 seconds. The resulting offset from the other direction (west to east) was thus set at 44 seconds, i.e., 60-16 = 44.

The third example interchange is at the intersection of Holiday Lane and I-820. As shown in Figure 5, Holiday Lane is a minor arterial with extremely high turning volumes at the two frontage road intersections. The signal timing diagram, also shown in Figure 5, identifies the phase sequence and duration at both intersections. The phase sequence shown also illustrates the 4-phase, 2-overlap, signal-phase sequence that was previously discussed. At this interchange, a 300-foot separation also suggests a 12-second travel time between intersections. As at the Arrowood Road Interchange, the existing timing plan reflects this fact by using a 10-second offset in both directions.

Evaluation of Existing Conditions

To illustrate the evaluation and optimization methodology, it was determined that the method of presentation would be consistent with a traditional traffic engineering study of an isolated intersection. As an initial step, current signal timings, traffic volumes, and interchange geometrics would be obtained. The second step would be to analyze the current interchange operating conditions and relate these conditions to the traditional level-of-service terminology. The third step would be to evaluate the findings and make some judgment as to the type of improvements needed (i.e., reapportioned timings, more lanes, additional phases, etc.) and to use this information in formulating alternative strategies. The fourth step would be to evaluate the improvement alternatives by repeating the third and fourth steps until the best solution has been found. The first step has been described above, step two will be discussed below, and the final "optimization" (steps 3 and 4) discussed in the next section of the report.

PASSER III and HCM Comparison. The evaluation of the Arrowood Road and Golfair Boulevard interchanges was conducted using both the Highway Capacity Software (HCS) and the PASSER III-88 program. The PASSER III-88 analysis was fairly straightforward; the existing signal timing, traffic volumes, and interchange geometrics were input and evaluated by simply running the program. To correctly use the HCS, however, some determination of the platoon arrival type for the interior interchange movements was necessary. As defined in Chapter 9 of the 1985 Highway Capacity Manual (15), Type 5 arrivals represent good traffic progression, while Type 1 arrivals represent poor or nonexistent progression. Type 3 arrivals represent an average condition, such as where vehicles arrive randomly throughout the cycle.

Based on existing offset information and recognizing the intended progression patterns (see Figures 3 and 4), the arrival types for the progressed movements were described as Type 5. These movements included westbound through movements at the west side intersections of both interchanges, and eastbound through movement at the east side of the Arrowood interchange. Because progression was not provided for eastbound traffic at the east side of the Golfair interchange, the arrival type was described as Type 2. All other interchange traffic movements were described as Type 3.

The results of this evaluation, i.e. one PASSER III run and two HCS analyses per interchange, are summarized in Table 1. In general, the results suggest that the PASSER III-88 and HCS methods are capable of arriving at very similar results when given the same input conditions. This conclusion was expected because both methods use the same basic equations for calculating volume to capacity (v/c) ratios and delays. One difference between the two program's capabilities, however, is PASSER III's ability to evaluate the adequacy of available queue storage.

Based on this evaluation it appears that both interchanges are operating at poor levels of service and have several movements with insufficient capacity and queue storage capabilities. Because these results were anticipated (i.e., operational problems were known to exist), the next logical step was to formulate and evaluate alternatives. This process is discussed in Section III. It should also be noted that even though both programs yield the same results, PASSER III was easier to use and provided an additional measure of effectiveness.

Output Interpretation. The evaluation of existing conditions at the Holiday Lane Interchange was conducted using only the PASSER III-88 program and illustrates interpretation of the program's level-of-service criteria. As before, the analysis was fairly straightforward; the existing signal timings, traffic volumes, and interchange geometrics (shown in Figure 5) were input and evaluated by simply running the program. Specifically, existing phasing type and internal offset were entered on the Signal-Phasing Data input screen; minimum phase times were

TABLE 1. COMPARISON OF PASSER III AND HCS EVALUATION RESULTS.

			PASSER III-88			HCS Approach	
Location	Side	Movements	v/c Ratio	Delay[1] (LOS) (sec/veh)	Storage[3] Ratio	v/c Ratio	Delay[1] (LOS) (sec/veh)
Arrowood and I-77	West	EB Through	0.61	16.2 (C)	-	0.60	15.4 (C)
		SB Left	1.17	136.6 (F)[2]	-	1.16	134.7 (F)[2]
		WB Left	0.25	0.2 (A)	0.02	0.25	5.0 (A)
		WB Through	0.53	1.1 (A)	0.10	0.53	2.0 (A)
	East	WB Through	0.64	20.3 (C)	-	0.64	19.0 (C)
		NB Left	0.59	7.7 (B)	-	0.59	7.6 (B)
		EB Left	1.41	336.5 (F)[2]	>1.00	1.41	269.0 (F)[2]
		EB Through	0.38	19.0 (C)	0.28	0.38	6.0 (B)
	Overall			48.1 (E)			40.5 (E)
Golfair and I-95	West	EB Through	0.35	12.3 (B)	-	0.34	11.4 (B)
		SB Lt. & Rt.	1.37	558.2 (F)[2]	-	1.38	372.7 (F)[2]
		WB Left	0.50	16.5 (C)	1.00	0.50	10.0 (B)
		WB Through	0.36	0.5 (A)	0.03	0.36	1.0 (A)
	East	WB Through	1.10	92.4 (F)[2]	-	1.10	75.6 (F)[2]
		NB Left	0.53	7.9 (B)	-	0.53	7.7 (B)
		EB Left	0.43	32.5 (D)	0.11	1.43	23.0 (C)
		EB Through	0.32	15.7 (C)	0.14	0.32	15.0 (B)
	Overall			37.8 (D)			34.2 (D)

[1]Delays represent stopped delay (Approach delay / 1.3).

[2] Movement over capacity; delays may not be realistic.

[3]Ratio of maximum queue per cycle to the available storage capacity for the movement.

set equal to actual phase times on the Movement-Interchange Data input screen; and existing volumes and saturation flow rates were entered on the Movement-Interchange Data input screen. The resulting output is reproduced as Figure 6, and for comparison purposes, PASSER III's level-of-service criteria is reproduced as Table 2.

Based on the results of this evaluation, it is possible to identify the particular signal phases (and the corresponding traffic movements) disadvantaged by the current signal timing and geometric conditions. From the measures of effectiveness reported in Figure 6, it would appear that both frontage roads (phase "B") are operating at poor levels of service. Furthermore, it appears that the southbound left and through movement (phase "C") at the left side intersection also has operational problems; i.e. unsatisfactory volume to capacity and storage ratios.

The poor operations at this interchange can be further explained when the individual movement volumes within each phase are examined. Comparison of movement volumes and lane allocations in Figure 5 suggest that the left-turn movements in the disadvantaged phases are the major cause of the operational problems at the Holiday Lane interchange. In particular, the left-turn movements from the frontage roads and the southbound left turn off of Holiday Lane appear to operate very near their one-lane capacities. The adjacent through and right turn movements, however, appear to have excess capacity. In other words the left-turn volume per lane is much higher than the through and right-turn volume per lane.

```
<GSI01>
* * * INTERCHANGE  1 HOLIDAY LANE                                RUN 1  PAGE  4A

              ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION   *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS         A     B     C     A+C   *   A     B     C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                              *
 PHASE TIME (SEC)  20.0  38.0  62.0  82.0   *  56.0  26.0  38.0  94.0
                                              *
 V/C RATIO            .64   .80   .86   .26  *   .77   .98   .65   .25
  LEVEL OF SERVICE  B     C     E     A     *   C     E     B     A
                                              *
 DELAY (SEC/VEH)   68.71 65.70 22.01   .93  *  33.29 126.12 25.39   .00
  LEVEL OF SERVICE  E     E     C     A     *   D     F     C     A
                                              *
 STORAGE RATIO                   .58   .02  *                 .26   .00
  LEVEL OF SERVICE               E     A     *                 C     A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   TTI -LEAD      TOTAL INTERCHANGE DELAY   40.76 VEH-HRS/HR
   INTERNAL OFFSET  10 SEC      CYCLE LENGTH  120 SEC
```

FIGURE 6. EVALUATION OF EXISTING CONDITIONS AT THE HOLIDAY LANE INTERCHANGE.

TABLE 2. LEVEL OF SERVICE CRITERIA FOR OPERATIONAL MEASURES OF EFFECTIVENESS AT SIGNALIZED DIAMOND INTERCHANGES (7).

	Level of Service					
Measures of Effectiveness	A	B	C	D	E	F
Volume to Capacity Ratio[1]	<.60	<.70	<.80	<.85	<1.0	>1.0
Average Vehicular Delay[2] (sec/veh)	<6.5	<19.5	<32.5	<52.0	<78.0	>78.0
Interior Storage Ratio[3]	<.05	<.10	<.30	<.50	<.80	>.80

1. "Guide for Designing and Operating Signalized Intersections in Texas" (16).
2. "1985 Highway Capacity Manual" Numbers in table represent total delay thresholds; i.e., stopped delay X 1.3.
3. "PASSER III-84 Users Manual" Numbers in table represent the ratio of the average maximum queue to the available storage.

III. SIGNAL TIMING OPTIMIZATION

The next step in the analysis process involves isolating the operational problems at the three interchanges. By inspection of the unequal v/c ratios shown in Table 1 (page 10), it should be apparent that the existing green times at the Arrowood Road and Golfair Boulevard interchanges could be more equitably allocated. In addition, a review of the delays for the progressive internal through movements (i.e., both west side westbound movements and the Arrowood east side eastbound movements) with the other movements suggests that the intended progression may not be as efficient as possible.

Based on these observations, several reasonable alternative solutions can be formulated. In particular, it would seem that different signal timings might lead to improved operation. Hence, a plan of attack might include an initial investigation of different offsets. As a second alternative, the phase duration could be varied (using the best offset found previously). Alternative cycle lengths also should be considered in this process to determine if another cycle length might be more efficient. In addition to signal timing, other operationally related improvement alternatives could include changes in phase sequence, and the use of protected/permitted left-turn phasing.

This section describes procedures for evaluating alternative traffic operational improvements at the example diamond interchanges. Although there are a number of alternatives discussed, their order of presentation should not be interpreted as a step-by-step procedure. That is, each alternative or combination of alternatives is aimed at solving a particular type of problem and not all interchanges have the same problem. Thus, some alternatives may not be appropriate at certain interchanges.

Offset Optimization

As suggested above, the next step might be to investigate a variation in the offset between through movements. In this regard, Figure 7 was generated using one of the optional features provided in PASSER III-88 to illustrate the effects of offset on progression. Basically this option was selected by deleting the existing offset and coding a "Y" in the "Run Delay-Offset" column for the existing phasing sequence at the interchange. The program then evaluated all possible offsets and output the results for the one offset which minimized delay.

As shown in Figure 7, the existing offsets do not coincide with the least delay offset. For the purposes of this analysis, it was assumed that the least delay solution was the best solution in terms of operating efficiency, realizing that in some instances that this assumption may not be true (i.e., minimum queue lengths are not guaranteed by a minimum delay solution). In other words,

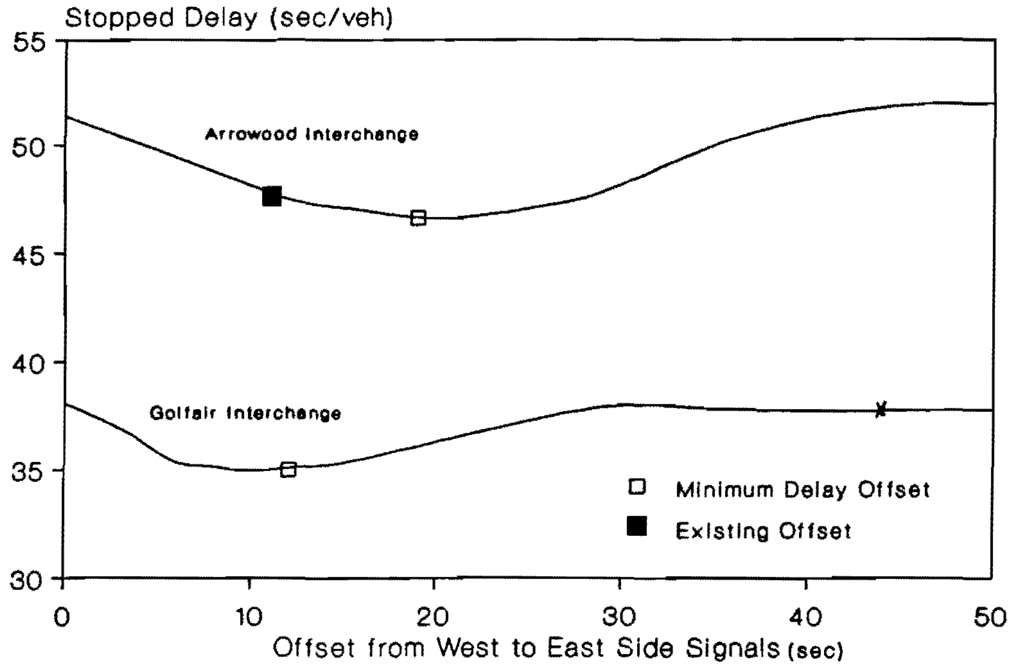


FIGURE 7. EFFECTS OF OFFSET ON DELAY AT SIGNALIZED DIAMOND INTERCHANGES.

the minimum delay solution would not be the best solution if the maximum queue lengths were not acceptable. In fact, if the available queue storage is exceeded, delays predicted by the minimum delay solution are probably unrealistic. PASSER III, as well as most other signal timing optimization models, does not model queue spillback into upstream traffic signals (i.e., the program assumes that traffic departs when the signal changes to green.

The results of the PASSER III-88 analysis of offset for both interchanges are listed in columns 4, 5, and 6 of Table 3. When comparing the individual movement delays (particularly those delays for the progressive movements) and the overall delays, it is obvious that a slight improvement was made at both interchanges. It also is obvious that the demand for several movements still exceeds their capacity, thus it appears that simply changing the offsets at the two interchanges does not solve their operational problems. Note also that queue storage is still a problem at the Arrowood Road interchange.

Split Optimization

The next improvement alternative tried was permitting the program to evaluate all combinations of offset and phase durations (or splits). In only one additional run, PASSER III-88 analyzed all offsets as before, but this time was also permitted the freedom of reallocating the cycle time; i.e., the offsets were evaluated as before and the splits were optimized by simply reducing each of the minimum phase times to 10 seconds before rerunning the program. The results of this analysis are shown in columns 7, 8, and 9 of Table 3.

Comparing these results with those of previous runs, it can be seen how split optimization dramatically improved most operational aspects at the Arrowood and Golfair interchanges. Individual volume to capacity ratios are more balanced and are all within the range of acceptable levels. Individual delays and storage ratios have also been decreased to acceptable levels of service (Level of Service D or better). Thus, further improvements are probably not needed at these interchanges.

The results of offsets and split optimization at the Holiday Lane interchange are illustrated in Figure 8. The top half of this figure represents existing conditions whereas the bottom half of the figure represents optimal offsets and splits. Again, the only change in the input data was that the existing offset and phase times were replaced by "Run Delay Offset Analysis" and minimum phase times, respectively.

TABLE 3. OPTIMIZATION OF SIGNAL OFFSET AND GREEN SPLITS USING PASSER III-88.

Location	Side	Movements	Optimize Offset Only			Optimize Offset and Splits		
			V/C Ratio	Delay[1] (LOS) (sec/veh)	Storage[3] Ratio	V/C Ratio	Delay[1] (LOS) (sec/veh)	Storage Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Arrowood	West	EB Through	0.61	16.2 (C)	-	0.42	11.3 (B)	-
and I-77		SB Left	1.17	136.6 (F)[2]	-	0.80	26.8 (D)	-
		WB Left	0.25	0.2 (A)	0.02	0.42	1.0 (A)	0.03
		WB Through	0.53	1.8 (A)	0.27	0.58	0.0 (A)	0.00
	East	WB Through	0.64	20.3 (C)	-	0.52	17.6 (C)	
		NB Left	0.59	7.7 (B)	-	0.95	27.2 (D)	-
		EB Left	1.41	332.7 (F)[2]	>1.00	0.58	6.4 (B)	0.22
		EB Through	0.38	2.8 (A)	0.15	0.23	1.9 (B)	0.16
	Overall			46.6 (E)			11.2 (B)	
Golfair	West	EB Through	0.35	12.3 (B)	-	0.53	20.7 (C)	-
and I-95		SB Lt. & Rt.	1.37	558.2 (F)[2]	-	0.37	18.5 (C)	-
		WB Left	0.50	0.0 (A)	0.00	0.53	0.0 (A)	0.00
		WB Through	0.36	0.3 (A)	0.03	0.43	1.4 (A)	0.07
	East	WB Through	1.10	92.4 (F)[2]	-	0.85	25.4 (D)	-
		NB Left	0.53	7.9 (B)	-	0.69	13.6 (B)	-
		EB Left	0.43	14.4 (B)	0.10	0.22	6.0 (B)	0.06
		EB Through	0.32	5.7 (B)	0.03	0.24	3.7 (A)	0.03
	Overall			35.0 (D)			10.6 (B)	

[1] Delays represent stopped delay (Approach delay / 1.3).

[2] Movement over capacity; delays may not be realistic.

[3] Ratio of maximum queue per cycle to the available storage capacity for the movement.

```
<GS101>
* * * INTERCHANGE  1 HOLIDAY LANE                                RUN 1  PAGE  4A

                 ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION  *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS         A      B      C     A+C *   A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                              *
 PHASE TIME (SEC)  20.0   38.0   62.0   82.0  *  56.0   26.0   38.0   94.0
                                              *
 V/C RATIO           .64    .80    .86    .26 *   .77    .98    .65    .25
  LEVEL OF SERVICE  B      C      E      A    *   C      E      B      A
                                              *
 DELAY (SEC/VEH)   68.71  65.70  22.01    .93 *  33.29 126.12  25.39    .00
  LEVEL OF SERVICE  E      E      C      A    *   D      F      C      A
                                              *
 STORAGE RATIO                     .58    .02 *                 .26    .00
  LEVEL OF SERVICE                 E      A   *                 C      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   TTI -LEAD      TOTAL INTERCHANGE DELAY   40.76 VEH-HRS/HR
   INTERNAL OFFSET  10 SEC      CYCLE LENGTH  120 SEC
```

```
<GS101>
* * * INTERCHANGE  1 HOLIDAY LANE                                RUN 1  PAGE  4A

                 ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION  *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS         A      B      C     A+C *   A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                              *
 PHASE TIME (SEC)  16.9   37.9   65.2   82.1  *  54.3   30.9   34.8   89.1
                                              *
 V/C RATIO           .80    .80    .81    .26 *   .80    .80    .72    .26
  LEVEL OF SERVICE  C      C      D      A    *   C      D      C      A
                                              *
 DELAY (SEC/VEH)  106.52  66.22  15.79   2.11 *  35.89  68.42  26.51    .53
  LEVEL OF SERVICE  F      E      B      A    *   D      E      C      A
                                              *
 STORAGE RATIO                     .42    .06 *                 .34    .14
  LEVEL OF SERVICE                 D      B   *                 D      C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   TTI -LEAD      TOTAL INTERCHANGE DELAY   36.75 VEH-HRS/HR
   INTERNAL OFFSET  13 SEC      CYCLE LENGTH  120 SEC
```

FIGURE 8. COMPARISON OF EXISTING CONDITIONS AND OPTIMIZED OFFSET AND SPLIT AT THE HOLIDAY LANE INTERCHANGE.

Note that offset and split optimization at Holiday Lane decreased total delay from 40.76 to 36.75 vehicle hours per hour; i.e., total delay was decreased by approximately 10 percent. Delay for several of the individual movements, however, is still relatively high (Level of Service E and F) indicating that additional improvements may be necessary; i.e., optimizing the offsets and splits at the Holiday Lane interchange may not solve all of the interchange's operational problems.

Cycle Length Optimization

The effects of cycle length were also explored as another means of improving interchange operation. Again, a PASSER III-88 feature was used to enter the range of cycle lengths to be considered. Each allowable cycle length between the upper and lower limits is evaluated and the results of each evaluation stored as output. For purposes of comparison with previous results, the program was also allowed to vary both the phase durations and the offset. The results of this analysis for the Arrowood and Golfair interchanges can be summarized as described below.

As shown in Figure 9, the optimum cycle lengths at the Arrowood and Golfair interchanges are 65 and 71 seconds, respectively. These cycle lengths are very close to the current cycle length of 60 seconds used at both interchanges and; therefore, cycle length is probably not a major contributor to the operational problems being experienced there. This result should not be surprising considering that the individual and overall delays were within acceptable limits when the splits were optimized.

The results of cycle length optimization at the Holiday Lane interchange are illustrated in Figure 10. The top half of Figure 10 reflects the existing cycle length of 120 seconds and the bottom half of Figure 10 reflects the minimum delay cycle length of 85 seconds. Note that the minimum delay cycle length is not very close to the existing cycle length. This difference is reflected by a 14 percent difference in total interchange delay. Individual movements, however, are still experiencing excessive delay. These excessive delays indicate that even though optimizing splits, offsets, and cycle length decreases overall delay by 22 percent, the interchange is still not operating satisfactorily.

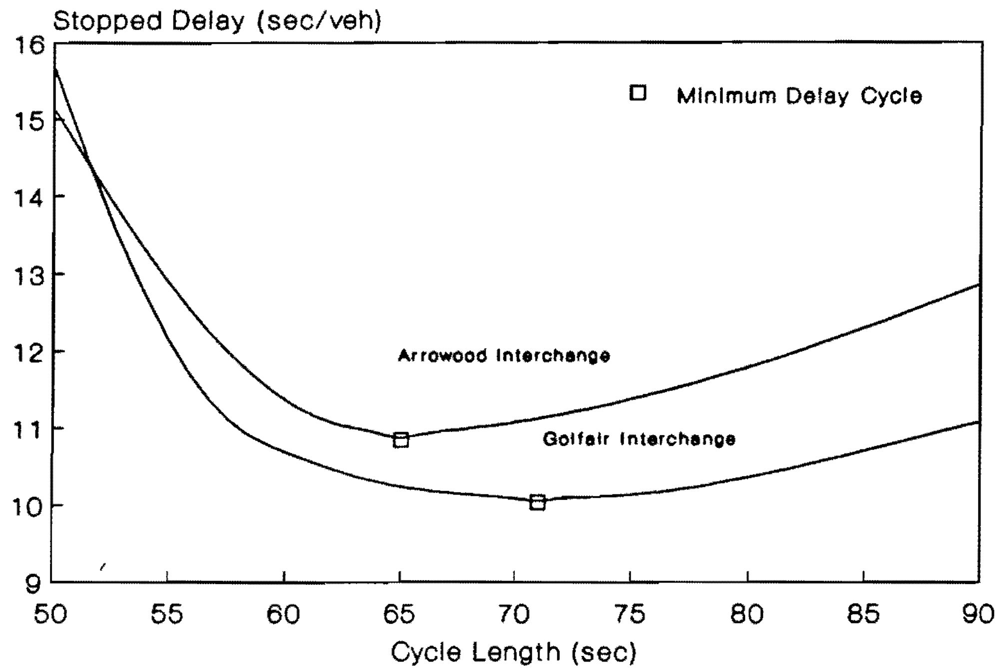


FIGURE 9. EFFECTS OF CYCLE LENGTH ON DELAY AT SIGNALIZED DIAMOND INTERCHANGES.

```
<GSI01>
* * * INTERCHANGE  1 HOLIDAY LANE                                  RUN 1  PAGE  4A

               ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION   *    RIGHT-SIDE INTERSECTION
EFFECTIVENESS         A      B      C     A+C  *    A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                               *
 PHASE TIME (SEC)  16.9   37.9   65.2   82.1   *  54.3   30.9   34.8   89.1
                                               *
 V/C RATIO           .80    .80    .81    .26  *   .80    .80    .72    .26
  LEVEL OF SERVICE  C      C      D      A     *  C      D      C      A
                                               *
 DELAY (SEC/VEH)  106.52  66.22  15.79   2.11  *  35.89  68.42  26.51   .53
  LEVEL OF SERVICE  F      E      B      A     *  D      E      C      A
                                               *
 STORAGE RATIO                     .42    .06  *                .34    .14
  LEVEL OF SERVICE                 D      B    *                D      C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   TTI -LEAD       TOTAL INTERCHANGE DELAY   36.75 VEH-HRS/HR
   INTERNAL OFFSET  13 SEC       CYCLE LENGTH  120 SEC
```

```
<GSI01>
* * * INTERCHANGE  1 HOLIDAY LANE                                  RUN 1  PAGE 11A

               ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION   *    RIGHT-SIDE INTERSECTION
EFFECTIVENESS         A      B      C     A+C  *    A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                               *
 PHASE TIME (SEC)  13.3   28.3   43.4   56.7   *  40.1   23.3   21.6   61.7
                                               *
 V/C RATIO           .79    .79    .89    .27  *   .79    .79    .89    .27
  LEVEL OF SERVICE  C      C      E      A     *  C      C      E      A
                                               *
 DELAY (SEC/VEH)   86.76  52.62  23.79   2.07  *  26.12  53.62  54.96   .00
  LEVEL OF SERVICE  F      E      C      A     *  C      E      E      A
                                               *
 STORAGE RATIO                     .60    .06  *                .38    .00
  LEVEL OF SERVICE                 E      B    *                D      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   TTI -LEAD       TOTAL INTERCHANGE DELAY   31.88 VEH-HRS/HR
   INTERNAL OFFSET   8 SEC       CYCLE LENGTH   85 SEC
```

FIGURE 10. COMPARISON OF OPTIMIZED OFFSET AND SPLIT AND OPTIMIZED CYCLE LENGTH AT THE HOLIDAY LANE INTERCHANGE.

Phasing Optimization

One alternative to improving the efficiency of the Holiday Lane interchange might be to look at an alternative phasing arrangement. Figure 11 presents a comparison of a cycle length optimization for two alternative phasings at Holiday Lane: the existing 4-phase with two overlaps (TTI-lead) and a basic 3-phase (normal lead-lead).

Several interesting findings resulted from this comparison. First, at the shorter cycle lengths, the 4-phase arrangement tended to produce more delay than did the 3-phase arrangement; however, not illustrated in this comparison is the queue storage problems at the interchange with the 3-phase arrangement and cycle lengths less than 50 seconds. In other words, what appears to be an optimal solution from a delay standpoint is in actuality an unacceptable solution from a queue storage standpoint.

A second finding was that minimum delay cycle lengths for 4-phase operation tended to be longer than minimum delay cycle lengths for 3-phase operation. In most cases, the tradeoff between the two phasing alternatives is less delay and more queue storage with 3-phase operation and higher delays and less queue storage with 4-phase operation. For some conditions, however, 4-phase and a longer cycle length can actually produce lower delays than 3-phase operation at a lower cycle length.

Left-Turn Phasing Optimization

One additional signalization alternative that might be considered is that of various types of left-turn phasing--protected, protected/permitted, and permitted. These alternatives can be evaluated by changing the permitted left-turn input on the Signal Phasing Data input screen.

Figure 12 illustrates the impact of changing from protected only to protected/permitted left turns at the Holiday Lane interchange. Notice that the only phases whose delays were affected by this change are the left turns, "C". At both intersections the average delay for left-turning vehicles was decreased. The magnitude of the improvement is smaller than might be expected and may not be significant. One must realize, however, that the Holiday Lane interchange is operating near capacity, and there is not much opportunity for permitted left turn maneuvers to be made. Had the opposing volume-capacity ratio been lower, the impact of a permitted left-turn phase would have been greater.

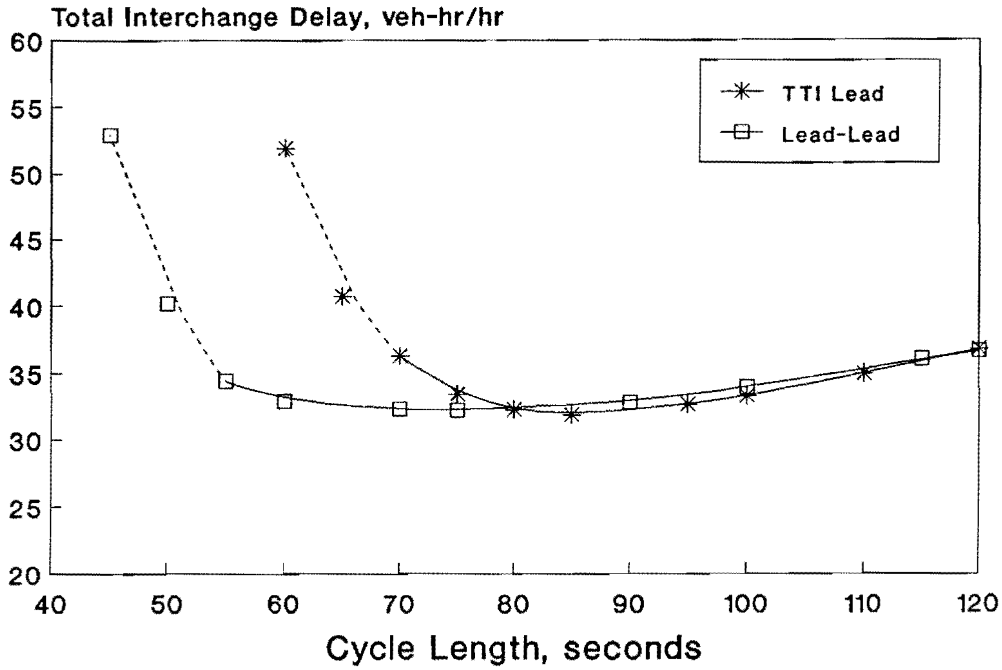


FIGURE 11. COMPARISON OF CYCLE LENGTH OPTIMIZATION FOR TWO ALTERNATIVE PHASING SEQUENCES AT THE HOLIDAY LANE INTERCHANGE.

```
<GSI01>
* * * INTERCHANGE  1 HOLIDAY LANE                                    RUN 1  PAGE 11A

               ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION   *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS          A      B      C    A+C  *   A      B      C    A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                               *
 PHASE TIME (SEC)  13.3   28.3   43.4   56.7   *  40.1   23.3   21.6   61.7
                                               *
 V/C RATIO           .79    .79    .89    .27  *    .79    .79    .89    .27
  LEVEL OF SERVICE  C      C      E      A     *   C      C      E      A
                                               *
 DELAY (SEC/VEH)   86.76  52.62  23.79   2.07  *  26.12  53.62  54.96    .00
  LEVEL OF SERVICE  F      E      C      A     *   C      E      E      A
                                               *
 STORAGE RATIO                     .60    .06  *                  .38    .00
  LEVEL OF SERVICE                E      B     *                 D      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    TTI -LEAD      TOTAL INTERCHANGE DELAY    31.88 VEH-HRS/HR
   INTERNAL OFFSET   8 SEC       CYCLE LENGTH   85 SEC
```

```
<GSI01>
* * * INTERCHANGE  1 HOLIDAY LANE                                    RUN 1  PAGE  4A

               ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF          LEFT-SIDE INTERSECTION   *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS          A      B      C    A+C  *   A      B      C    A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                               *
 PHASE TIME (SEC)  13.3   28.3   43.4   56.7   *  40.1   23.3   21.6   61.7
                                               *
 V/C RATIO           .79    .79    .89    .27  *    .79    .79    .89    .27
  LEVEL OF SERVICE  C      C      E      A     *   C      C      E      A
                                               *
 DELAY (SEC/VEH)   86.76  52.62  22.12   2.07  *  26.12  53.62  50.50    .00
  LEVEL OF SERVICE  F      E      C      A     *   C      E      D      A
                                               *
 STORAGE RATIO                     .56    .06  *                  .36    .00
  LEVEL OF SERVICE                E      B     *                 D      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    TTI -LEAD      TOTAL INTERCHANGE DELAY    31.28 VEH-HRS/HR
   INTERNAL OFFSET   8 SEC       CYCLE LENGTH   85 SEC
```

FIGURE 12. ILLUSTRATION OF THE EFFECTS OF CHANGING FROM PROTECTED ONLY TO PROTECTED/PERMITTED LEFT TURNS AT THE HOLIDAY LANE INTERCHANGE.

IV. CONTROLLER SETTINGS

One of the difficulties in applying the results from PASSER III is conversion of the program's output to controller settings for field implementation. This problem is compounded by the fact that the output format is relatively constant no matter what the hardware configuration; i.e., three- or four-phase, pretimed or actuated, one or two controllers. The rationale for a consistent output format is to facilitate the use of the program for evaluation and interpretation. It is also impossible to produce a format for each of the numerous alternative hardware configurations currently in use. The disadvantage of a consistent output format is that the traffic engineer and/or signal technician must translate the program's output to the needs of their particular piece of equipment.

This section offers some guidance in proper interpretation of PASSER III signal timing output. First, the commonality between the program's output and the phase numbering scheme used in the TSDHPT's solid state diamond interchange controller unit are presented. Second, the procedure for converting PASSER III's phase lengths to green splits and clearance intervals is described. Finally, a methodology for computing actuated controller settings is discussed.

Phase Numbers

Phase numbers for the state's standard diamond interchange controller unit are the same as a standard NEMA 8-phase controller. They are assigned to traffic movements at a diamond interchange as shown in Figure 13. Basically, the arterial movements are assigned to phases 2 and 6, the frontage road or cross-street movements are assigned to phases 4 and 8, and the interior left turn movements are assigned to phases 1 and 5. The interior through movements are assigned to overlap A (OVLA) and overlap B (OVLB). Overlap A is concurrent with both phases 1 and 2 and overlap B is concurrent with both phases 5 and 6. The controller unit operates as two independent 4-phase rings and has the capability of switching between 4-phase and 3-phase diamond operation.

Figure 14 illustrates the relationship between PASSER III's output and this particular phase numbering scheme. Diamond interchange controller phase 2 corresponds to PASSER III's left-side phase A, and has a duration, including clearance, of 50.5 seconds; diamond interchange controller phase 6 corresponds to PASSER III's right-side phase A, and has a duration, including clearance, of 24 seconds; diamond interchange controller phase 4 corresponds to PASSER III's left-side phase B, and has a duration, including clearance, of 19.5 seconds; diamond interchange controller phase 8 corresponds to PASSER's right side phase B, and has a duration, including clearance, of 24 seconds; diamond interchange controller phase 1 corresponds to PASSER III's left-side phase C, and has a duration, including clearance, of 30 seconds; diamond interchange

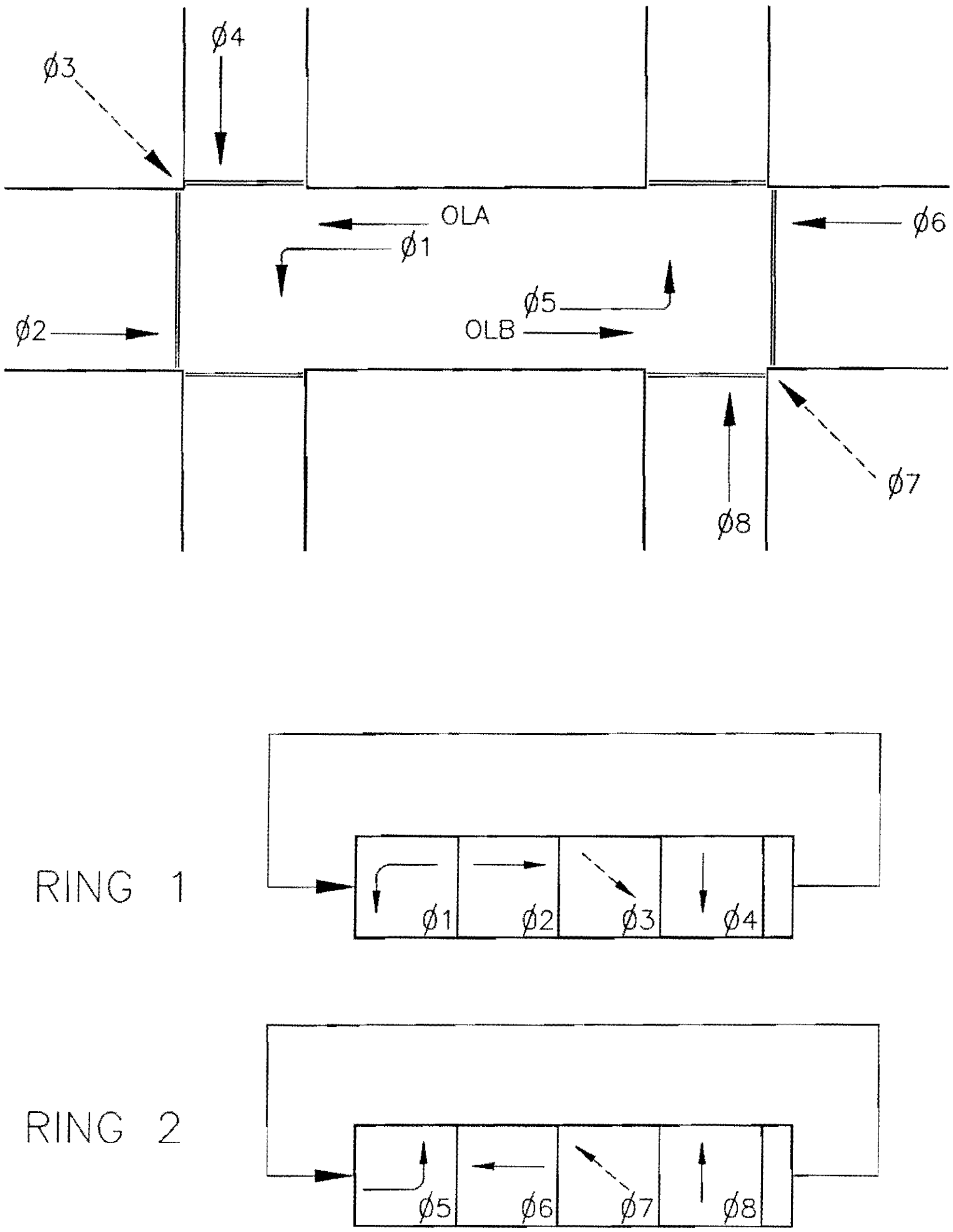


FIGURE 13. SDHPT DIAMOND INTERCHANGE CONTROLLER UNIT.

```
<SPI01>
* * * INTERCHANGE   1 EISENHAUER                                          RUN 1   PAGE  4B

                    ***   SIGNAL PHASING INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                     *                      *
                * LEFT-SIDE SEQUENCE  * RIGHT-SIDE SEQUENCE  *
                *  A      B      C    *  A      B      C     *
PHASE INTERVAL  * <----   |    <----  * <----   ^         ^  *  PHASE INTERVAL
    NUMBER      *         |    |----  *         |     ----|  *   LENGTH (SEC)
                * ---->   V    V      * ---->   |     ---->  *
                *                     *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                     *                      *
       1        *         A           *         B            *     12.00
       2        *         A           *         C            *     38.50
       3        *         B           *         C            *     13.50
       4        *         B           *         A            *      6.00
       5        *         C           *         A            *     18.00
       6        *         C           *         B            *     12.00
                *                     *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   INTERNAL OFFSET     12 SEC                           CYCLE LENGTH   100 SEC
                                                        PHASE ORDER TTI -LEAD
```

FOUR–PHASE PRETIMED DIAMOND
INTERCHANGE PHASING

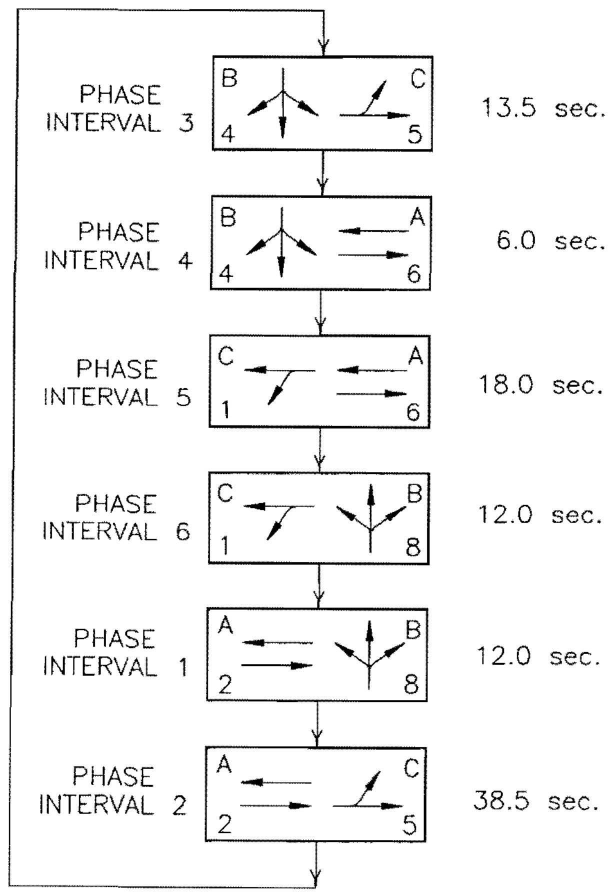


FIGURE 14. RELATIONSHIP BETWEEN PASSER III OUTPUT AND DIAMOND INTERCHANGE PHASING NUMBERS.

controller phase 5 corresponds to PASSER III's right-side phase C, and has a duration, including clearance, of 52 seconds. Overlap A is concurrent with PASSER III's left-side Phases A and C, and has a duration of 80.5 seconds. Overlap B is concurrent with PASSER III's right-side Phases A and C, and has a duration of 76 seconds.

Phase Lengths

As mentioned previously, the phase lengths output by PASSER III represent the total length of the green and change (yellow plus all-red) intervals for that phase. The phase length data appear on two different screens in the program's output (see Figure 15). On the General Signalization Information screen (top half of Figure 15), phase lengths are printed for each individual phase at the two intersections. For example, PASSER III Phases A, B, and C at the left-side intersection correspond to diamond controller phase numbers 2, 4, and 1, respectively, and PASSER III Phases A, B, and C at the right-side intersection correspond to diamond controller phase numbers 6, 8, and 5, respectively. Green interval durations are determined by subtracting the appropriate change interval duration from the length of the phase.

On the Signal Phasing Information screen (bottom half of Figure 15), phase interval lengths are printed for each phase of the interchange's timing plan. This screen shows the interrelationship of the signal phase sequences between the left and right side intersections of the interchange for a 3-phase timing plan. In this case, PASSER III's phase interval 1 corresponds to diamond interchange controller phase 2 + 5, PASSER III's phase interval 2 corresponds to diamond interchange controller phase 4 + 8, PASSER III's phase interval 3 corresponds to diamond interchange controller phase 1 + 6, and PASSER III's phase interval 4 corresponds to diamond interchange controller phase 1 + 5.

Actuated Controller Settings

As mentioned in Section I, PASSER III was developed to analyze pretimed or traffic-responsive fixed sequence signalized diamond interchanges. The phase lengths reported by the program are generally thought of as pretimed settings and no guidance is given as to how to convert these times to actuated settings. The following procedure describes one technique for translating the PASSER III output to reasonable actuated controller settings. It is based, in part, on a procedure developed by Skabardonis for converting PASSER II output into actuated controller settings (17).

The discussion focuses on the three phases that occur at each intersection of the interchange. It is assumed that the relationship between rings and phases is as shown in Figure 13. Other

```
<GSI01>
* * * INTERCHANGE  3 el paso st                                    RUN 04 PAGE 14A

                ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF         LEFT-SIDE INTERSECTION  *   RIGHT-SIDE INTERSECTION
EFFECTIVENESS        A      B      C     A+C *   A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                             *
 PHASE TIME (SEC)  19.0   30.0   16.0   35.0 *  14.0   30.0   21.0   35.0
                                             *
 V/C RATIO           .47    .51    .48    .18 *   .42    .46    .47    .26
  LEVEL OF SERVICE   A      A      A      A  *   A      A      A      A
                                             *
 DELAY (SEC/VEH)   23.81  21.48  13.06   4.29 * 28.45  15.87   7.33   4.74
  LEVEL OF SERVICE   C      C      B      A  *   C      B      B      A
                                             *
 STORAGE RATIO                     .19    .16 *                .32    .20
  LEVEL OF SERVICE                 C      C  *                D      C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   LEAD-LAG       TOTAL INTERCHANGE DELAY    13.77 VEH-HRS/HR
   INTERNAL OFFSET  45 SEC
```

```
<SP101>
* * * INTERCHANGE  3 el paso st                                     RUN 04 PAGE 14B

                   ***   SIGNAL PHASING INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
                * LEFT-SIDE SEQUENCE   * RIGHT-SIDE SEQUENCE  *
                *   A      B      C    *   A      C      B    *
PHASE INTERVAL  * <----    |   <----   * <----       ^   ^    *  PHASE INTERVAL
    NUMBER      *          |   |----   *         ----|   |    *  LENGTH (SEC)
                * ---->    V   V       * ---->  ---->    |    *
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
       1        *          A           *          C           *       19.00
       2        *          B           *          B           *       30.00
       3        *          C           *          A           *       14.00
       4        *          C           *          C           *        2.00
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   INTERNAL OFFSET    45 SEC                        CYCLE LENGTH    65 SEC
   EXTERNAL OFFSET  40.2 SEC                        PHASE ORDER LEAD-LAG
```

FIGURE 15. ILLUSTRATION OF PHASE LENGTHS OUTPUT BY PASSER III.

assumptions include that PASSER III's Phase A, the crossroad through phase, is the sync phase and that a constant cycle length and phase sequence will be used during the control period. Although there is no explicit reference to the left and right side intersection in many of the steps, it is presumed that the analyst will apply the following procedure to both intersections of the interchange.

Minimum Green. The minimum phase duration for each phase (A, B, and C) is based on consideration of three factors. First, if a phase is to exist, it must have a minimum green time of 6 to 10 seconds to satisfy driver expectancy. Second, the minimum phase duration must be long enough to safely allow pedestrians to cross the street. A third consideration applies to the minimum phase duration if only advance detection is used to control the phase (i.e., no stop line detection). In this instance, the minimum phase duration must be sufficient to allow any vehicles stopped between the detector and stop line to clear.

The following relationships describe the calculation of the minimum green and phase durations. It should be noted that the minimum phase duration includes the green, yellow, and all-red intervals.

$$G_{min} = P_{min} - Y - AR$$

For Phases A and B: (Phases 2, 6, 4, and 8)

$$P_{min} = \text{larger of} \begin{cases} \dfrac{D}{L_Q} \times \dfrac{3600}{S} + (l_1 + l_2) \\ W + FDW \end{cases}$$

For Phase C: (Phases 1 and 5)

$$P_{min} = 6.0 + Y + AR$$

where:

G_{min} = minimum green interval duration for phase, sec.;
P_{min} = minimum duration of phase, sec.;
Y = yellow interval duration for phase, sec.;
AR = all-red interval duration for phase, sec.;
D = distance from stopline to nearest edge of detector serving phase, ft.;
L_Q = space occupied by queued vehicle, ft.; (use 25.0 feet/vehicle);
S = saturation flow rate of critical movement in phase, vphgpl;
l_1 = startup lost time in phase, sec. (use 2.0 sec.);
l_2 = end lost time in phase, sec. (use 2.0 sec.);
W = steady WALK interval for phase, sec., see Table 4;
FWD = flashing DON'T WALK for phase, sec., see Table 4.

TABLE 4. WALK AND FLASHING DON'T WALK INTERVAL DURATIONS.

Ped. Demand (peds./cycle)	Ped. Button	WALK interval (seconds)	Flashing DON'T WALK Interval (seconds)
0 - 10	No	5.0	(W - 6)/4.0
>10 [1]	Yes	7.0 x f	(W - 6)/3.5 x f
>10	Yes	7.0	(W - 6)/3.5

W = curb-to-curb width of street being crossed, ft.;

f = fraction of time that pedestrian calls occur. Calculated as: $f = 1 - e^{-P \cdot C/3600}$

P = pedestrian flow rate during the control period, pph;

C = cycle length, sec.

Note: 1 - This value or procedure is used to estimate the average minimum phase duration during the control period and should be used for PASSER III analysis purposes only. The actual minimum phase duration based on pedestrian crossing needs should be calculated using an "f" equal to 1.0.

Vehicle Extension

Stopline Detection. The vehicle extension interval for stopline detection is based on the desired minimum allowable gap that will extend the green interval. In general, the shortest vehicle extension interval that will not result in premature termination of the phase is desired. To prevent termination of the green before queued demand has been served, the maximum allowable gap should be established first and this value used to calculate the vehicle extension.

The following relationship between the maximum allowable gap and the average amount of unused time has been established based on the assumption of random arrivals during the phase receiving green:

$$GAP_{max} = -\left(\frac{3600}{Q}\right) \times \ln\left(\frac{1}{Q/3600 \times D + 1}\right)$$

where:

GAP_{max} = maximum allowable gap for phase, sec. (see Figure 16);

Q = total flow rate on all approaches served during phase, vph;

D = average duration of extended green after the queue dissipates, sec.

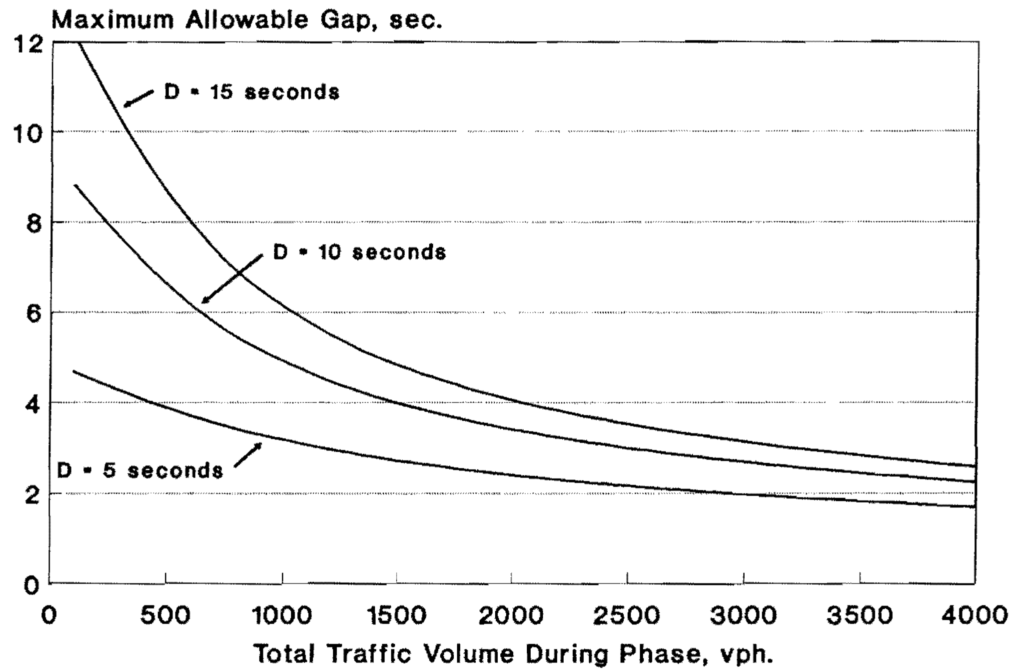


FIGURE 16. MAXIMUM ALLOWABLE GAP VS. TOTAL TRAFFIC VOLUME DURING PHASE.

A reasonable value of D would seem to be about 10 seconds. A lower value would yield a lower maximum allowable gap but would also increase the possibility of early termination of the green. In contrast, as D increases, the delay to traffic in the other phases increases proportionally.

Once the maximum allowable gap has been established, the following equation can be used to calculate the duration of the vehicle extension for Phases B and C:

$$VE = \text{larger of} \begin{cases} GAP_{max} - \left(\dfrac{L_D + L_V}{V}\right) \\ 2.0 / N \end{cases}$$

where:

VE = vehicle extension for phase, sec.;
L_D = length of the detector, ft.;
L_V = detected length of vehicle, ft. (use 14.0 feet);
V = speed of vehicles in transit over loop, fps;
N = total number of lanes served during phase.

Advance Detection. The extension interval used for advance detection is based on the need to provide dilemma zone protection. In this regard, the vehicle extension must be long enough for the driver to travel to the intersection before the yellow interval is presented. Thus, the vehicle extension setting is dictated by safety considerations for advance detection as opposed to performance settings with stop line detection.

In general, the vehicle interval is equal to the travel time from the detector to the stopline. This time, however, can become so large for high speed approaches that it can be quite inefficient. One technique for reducing the vehicle extension calculated in this fashion is based on the probability of drivers stopping as a function of distance from the stopline. Studies have shown that almost all drivers who are less than 2.0 seconds from the stopline at the onset of the yellow indication will proceed through the intersection rather than stop. Thus, to provide dilemma zone protection, the vehicle extension need only be long enough to project the driver from the detector to a point 2.0 seconds, or less, from the stop line. The following equation uses a conservative value of 1.5 seconds as the near boundary of the dilemma zone:

$$VE = \text{larger of} \begin{cases} \dfrac{D - 14}{V} - 1.5 \\ 2.0 \end{cases}$$

Maximum Green. The maximum phase duration and maximum green interval for each of the actuated phases (i.e., Phases B and C) can be calculated using the following procedure:

1. Run PASSER III to get the optimum phasing sequence, phase lengths, and cycle length for the control period. In this step, the average hourly flow rates during the control period and minimum phase durations, P_{min}, must be input to PASSER III. The optimum phase lengths will be used below for calculating the maximum phase durations. The optimum phase sequence and cycle length will be used later for calculating the force-off and yield points.

2. Check the volume-to-capacity condition (X-ratio) for each phase. The X-ratio should be 0.85 or less. If it is not, the phase will operate more nearly as pretimed than actuated during the control period. The critical X-ratio for each phase can be obtained directly from PASSER III's General Signalization Information output screen.

3. If the phase's demand volume to signal capacity ratio (X-ratio) from the PASSER III output is less than 0.85, the optimum phase duration from the PASSER III output (G) can be used to determine the maximum phase duration (G_{max}). In this case, the maximum phase duration is calculated as:

   $$G_{max} = G$$

   If the phase's demand volume to signal capacity ratio (X-ratio) from the PASSER III output is between 0.85 and 0.95, the maximum phase duration is calculated as (17):

   $$G_{max} = G + \frac{X^2}{2 \cdot (1 - X)}$$

   If the phase's X-ratio is over 0.95, the capacity of the interchange may be inadequate or the signal timings may not be efficient. Ideally, a capacity increase or signal timing change would be made to lower the X-ratio to less than 0.95; the maximum phase duration, G_{max}, can be estimated using the above equation and an X-ratio of 0.95.

   At this point, a check should be made to ensure that the maximum phase duration is greater than the minimum phase duration calculated previously. In particular, if the minimum phase durations for Phases A and B have been factored by the percentage of pedestrian actuations, it is possible for the maximum phase duration to be less than the minimum pedestrian requirement. Therefore, if the pedestrian actuation factor (f) was used, the minimum phase duration should be recalculated with f equal to 1.0 and then compared with the maximum phase duration. If P_{min} is greater than G_{max}, then G_{max} should be set equal to P_{min}.

Once the maximum phase duration is calculated, the maximum green interval (g_{max}) can be calculated as:

$$g_{max} = G_{max} - Y - AR$$

Yield and Force-Off Points. Determining the optimum settings for yield and force-off points requires the following sequence of steps:

1. Calculate the effective green for the actuated phases; i.e., Phases B and C (g_b and g_c), using the following equation:

$$g_i = X_i \bullet (G_i - l_i) + VE_i + (Y - l_2)_i \qquad i = b, c$$

where:
g_i = effective green for phase i, sec.;
X_i = volume-to-capacity ratio of critical phase i (from PASSER output);
G_i = optimum duration of phase i, sec. (from PASSER output);
l_i = sum of lost time components during phase i, ($l_i = l_1 + l_2$), sec.

2. Calculate the minimum effective green for the sync phase; i.e., Phase A (g_a), using the following equation:

$$g_a = C - l_T - g_b - g_c$$

where:
C = optimum cycle length, sec. (from PASSER output);
l_T = total lost time, sec.; $l_T = l_a + l_b + l_c$.

3. Check the X-ratio of Phase A to verify that it is less than 0.90 by using the following equation:

$$X_a \bullet \frac{(G_a - l_a)}{g_a} < 0.90$$

If this condition is not satisfied, then recalculate the effective green for Phase A as:

$$g_a = (G_a - l_a) \cdot \frac{X_a}{0.90}$$

Then reduce the effective greens for Phases B and C using the following equation (17):

$$g_i = (C - g_a - l_T) \cdot \frac{Q_i}{Q_b + Q_c} \qquad i = b, c$$

where:

g_i = revised effective green for phase i, sec.;
Q_i = critical flow rate during phase i, sec.

4. Determine the average actuated phase duration for each phase with the following equation:

$$G_i \text{ (Avg.)} = g_i + l_i \qquad i = b, c$$

where:

G_i (Avg.) = average duration of phase i, sec.;
l_i = lost time of phase i, sec.

5. To determine the optimum offset relationship between the two intersections of the interchange and the appropriate yield and force-off points, use the optimum cycle length, phase sequence, and average phase durations, G_i (Avg.), in a second PASSER III analysis; i.e., allow PASSER III to optimize offset only.

6. Establish the offset to yield point for the left-side intersection (O_l). If the interchange is coordinated with other signals, the offset will be predetermined based on the coordination of Phase A with the system master intersection. If the interchange operates in an isolated mode, the offset to yield point can be established as zero seconds.

7. Use the optimum offset reported by PASSER III to determine the offset to yield point for the right-side intersection (O_r). The offset to yield point is not equal to the offset reported by PASSER III. The offset reported by PASSER III is defined as the time measured from the start of Phase A on the left to the end of Phase B on the right. In contrast, the yield point is referenced to the end of Phase A on both the left- and right-side intersections.

The PASSER III optimum offset and actual green splits can be used to determine the yield point of the right-side intersection relative to the system reference (and thus, the left-side intersection). The calculation of the right-side offset is dependent on the phase sequencing of the right-side intersection.

Right-side intersection with leading left-turn phasing (A B C):

$$O_r = O_l - G_a\,(\text{left}) + O_p + G_c\,(\text{right}) + G_a\,(\text{right})$$

Right-side intersection with lagging left-turn phasing (A C B):

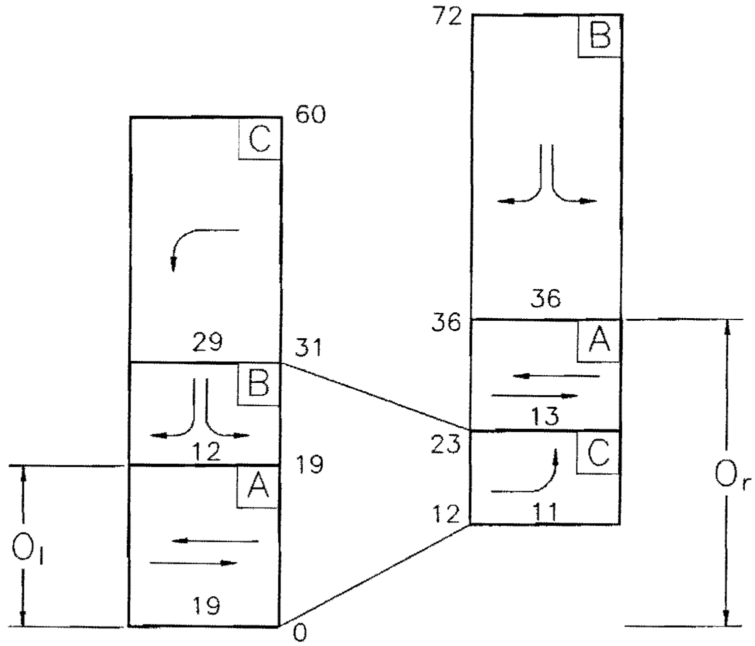
$$O_r = O_l - G_a\,(\text{left}) + O_p + G_b\,(\text{right}) + G_a\,(\text{right})$$

where:

O_r = offset to yield point for right-side intersection, sec.;
O_l = offset to yield point for left-side intersection, sec.;
O_p = optimum offset from PASSER III output in Step 1, sec.;
G_i = optimum duration of phase i for left or right intersection (from PASSER III output) sec.

It should be noted that all offsets are expressed in terms of one cycle length. Thus, if the calculated offset exceeds the cycle length, then one cycle should be subtracted from the value of the offset. For example, if the calculated offset to yield point for the right-side intersection (O_r) is 140 seconds and the cycle length is 100 seconds, then the actual relative offset is 40 seconds (i.e., $O_r = 140 - 100 = 40$ sec.).

8. At this point, the force-off points can be determined using the optimum phase durations, G, (from PASSER III) and the yield points established in the preceding task. The force-off points are calculated by adding the optimum phase durations to the previous yield point or force-off point based on the phasing sequence of the left- or right-side intersection. The example shown in Figure 17 demonstrates this technique for an interchange with a 60-second cycle length and lead-lead left-turn phasing.



Left Side Intersection (O_1 = 19 seconds, given)

Phase	Optimum Phase (seconds)	Yield Point (seconds)	Force-Off Point (seconds)
A (φ 2)	19	19	-
B (φ 4)	12	–	31 (19 + 12)
C (φ 1)	29	–	60 (31 + 29)

Right Side Intersection O_p + 12 (from second PASSER run)

$O_r = O_1 - G_a\text{ (left)} + O_p + G_a\text{ (right)}$

$O_r = 19 - 19 + 12 + 11 + 13 = 36$ seconds

Phase	Optimum Phase (seconds)	Yield Point (seconds)	Force-Off Point (seconds)
A (φ 6)	13	36	-
B (φ 8)	36	–	72 (36 + 36)
C (φ 5)	11	–	23 (72 + 11 – 60)

FIGURE 17. INTERCHANGE PHASING SEQUENCE.

V. CONCLUSIONS

As shown in the preceding analysis, PASSER III-88 is an extremely powerful tool for evaluating and optimizing traffic operations at signalized diamond interchanges. At present, no other methodologies or computer programs are available that easily deal with all the complexities found at diamond interchanges.

The results of the evaluation and optimization of the interchanges presented in this report illustrated the nature and magnitude of potential improvements that could typically be found in the field. In addition, the comparison of the HCS and PASSER III-88 evaluations indicates agreement between the analytical approaches used by each. PASSER III, however, evaluates additional measures of effectiveness (i.e. queue storage) that are especially important at diamond interchanges.

The benefit of using PASSER III-88 is its ability to analyze complex traffic interactions at diamond interchanges better, more thoroughly, and more accurately than any other computer program. Not only will PASSER III-88 provide a consistant approach to diamond interchange analysis, but its timesaving capabilities will also enable transportation engineers to make a greater number of analyses and be more confident in the efficiency of the solution reached.

REFERENCES

1. "Dallas Corridor Study: Final Report." Report No. FHWA RD-77-15. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, August 1976.

2. 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." Report No. FHWA/TX-74/165-14. College Station, Texas: Texas Transportation Institute, Texas A&M University, August 1974.

3. 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." Report No. FHWA/TX-75/203-2F. College Station, Texas: Texas Transportation Institute, Texas A&M University, August 1975.

4. Messer, C.J., Fambro, D.B., and Turner, M.J. "Analysis of Diamond Interchange Operation and Development of a Frontage Road Level of Service Evaluation Program - PASSER III -- Final Report." Report No. FHWA/TX-76/178-2F. College Station, Texas: Texas Transportation Institute, Texas A&M University, August 1976.

5. Fambro, D.B., Putman, D.L., Haenel, H.E., Cervenka, L.W., and Messer, C.J. "A Report on the User's Manual for Diamond Interchange Signalization-- PASSER III." Report No. FHWA/TX-76/178-1. College Station, Texas: Texas Transportation Institute, Texas A&M University, August 1977.

6. "PASSER III-84," Texas Department of Highways and Public Transportation, November 1985.

7. Fambro, D.B., Chaudhary, N.A., Messer, C.J., and Garza, R.U. "A Report on the User's Manual for the Microcomputer Version of PASSER III-88." Report No. FHWA/TX-88/478-1, College Station, Texas: Texas Transportation Institute, Texas A&M University, May 1988.

8. "Signal Control Strategies for Signalized Diamond Interchanges." Report No. FHWA-IP-80-5. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, December 1980.

9. Woods, D.L. "Limitations of Phase Overlap Signalization for Two-Level Diamond Interchanges." Traffic Engineering, Vol. 39, No. 12, September 1969, pp. 38-41.

10. Guyton, J.W., Yarbrough, J.W., and Arey, T.M. "Control Strategies for Signalized Diamond Interchanges." Report No. FHWA-TS-78-206. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, April 1978.

11. "Diamond Interchange Program User's Manual." Report No. FHWA-IP-80-4. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, December 1980.

12. Haenel, H.E., Kosik, A.H., and Marsden, B.G.. "Innovative Uses of Traffic Responsive Control in Texas." presented at the Engineering Foundation Conference in Henniker, New Hampshire, July 1983.

13. Messer, C.J., Fambro, D.B., and Richards, S.H. "Optimization of Pretimed Signalized Diamond Interchanges." Transportation Research Record 644. Washington, D.C.: Transportation Research Board, National Academy of Sciences, 1977, pp.78-84.

14. Messer, C.J. and Chang, M.S. "Traffic Operations of Basic Traffic Actuated Control Systems at Diamond Interchanges." Transportation Research Record 1114. Washington, D.C.: Transportation Research Board, National Academy of Sciences, 1988, pp. 54-62.

15. Highway Capacity Manual. TRB Special Report 209. Washington, D.C.: Transportation Research Board, National Academy of Sciences, 1985.

16. Messer, C.J., and Fambro, D.B. "Guide for Designing and Operating Signalized Intersections in Texas." Report No. FHWA/TX-75/203-1. College Station, Texas: Texas Transportation Institute, Texas A&M University, August 1976.

17. Skabardonis, A. "Progression Through a Series of Intersections with Traffic Actuated Controllers." Report No. FHWA-RD-89-133. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, October 1988.

APPENDIX A

SATURATION FLOW RATE ASSISTANT

SATURATION FLOW RATE ASSISTANT

One of the enhancements that was added to PASSER III-88 is an optional assistance screen for calculating saturation flow rates for the 18 possible movements at a diamond interchange. The manual procedure that is recommended for calculating saturation flow rates is described in the appendix of the "User's Manual for the Microcomputer Version of PASSER III-88". This procedure is based on a proportioning technique and Equation 9-8 in the 1985 Highway Capacity Manual (8). The optional assistance screen is simply an automation of this manual procedure. This appendix uses an example interchange to illustrate the use of the automated procedure.

Turning movements, interchange geometrics, and lane assignments or an estimate thereof, are required to determine the input saturation flow rates for PASSER III-88. Basically, whenever individual movements are made from exclusive lanes, the saturation flow rate for movement should be equal to the saturation flow rate for the exclusive lanes. If more than one movement shares a common lane, however, the saturation flow rates for the individual movements should have the same proportion as the volumes for the individual movements. The exception to this latter rule is that whenever the volume proportion exceeds the saturation flow rate of the shared lane, the shared lane should be treated as an exclusive lane for the higher volume movement.

Figure A-1 illustrates the AM peak hour volumes, interchange geometrics, and lane assignments that will be used to illustrate the saturation flow rate calculation, and Figure A-2 illustrates the Interchange Movement Data input screen prior to any data being entered. The user may enter volumes and saturation flow rates directly on this screen by using the arrow and numeric keys, or indirectly on the screen by using the optional assistance screen. To access the assistance screens, the user should press the F3 key, and a window similar to the one shown in Figure A-3 will appear on the screen. The exact wording in the box depends upon the cursor position on the input screen when the F3 key was pressed. Specifically, there are six different windows for the six blocks of data pertaining to the six approaches at the interchange--four external approaches and two internal approaches.

Upon entering the saturation flow rate adjustment screen, the cursor will be positioned in the data entry field for the "Number of Approach Lanes." The values shown in Figure A-3 are default values, any of which can be changed by the user. Each of the data entry fields in this screen has its own error checking routines to minimize entry of unreasonable or incorrect data. The number of approach lanes at the left-side cross-street of the example interchange is two, thus, the default value does not need changing. Had the number of approach lanes been greater than two, entry of the appropriate value would have caused "Middle Lane(s)" to have been inserted between the "Right Lane" and the "Left Lane" in the bottom third of the window.

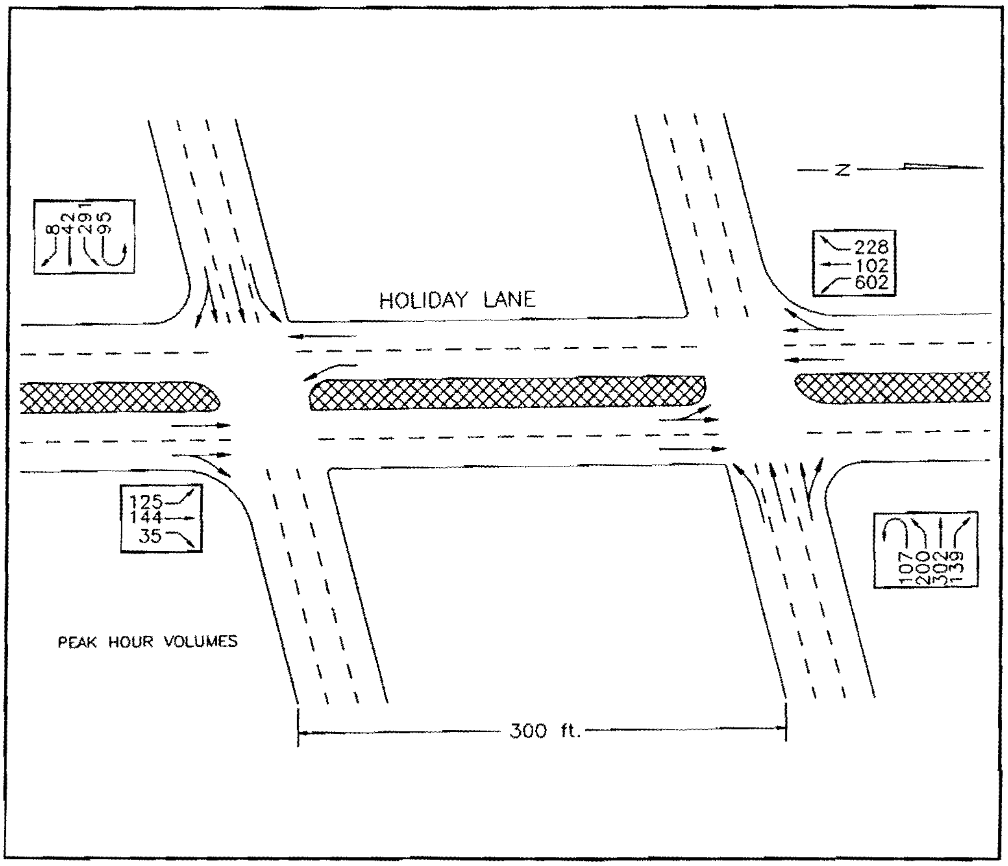


FIGURE A-1. EXAMPLE INTERCHANGE VOLUME AND GEOMETRICS.

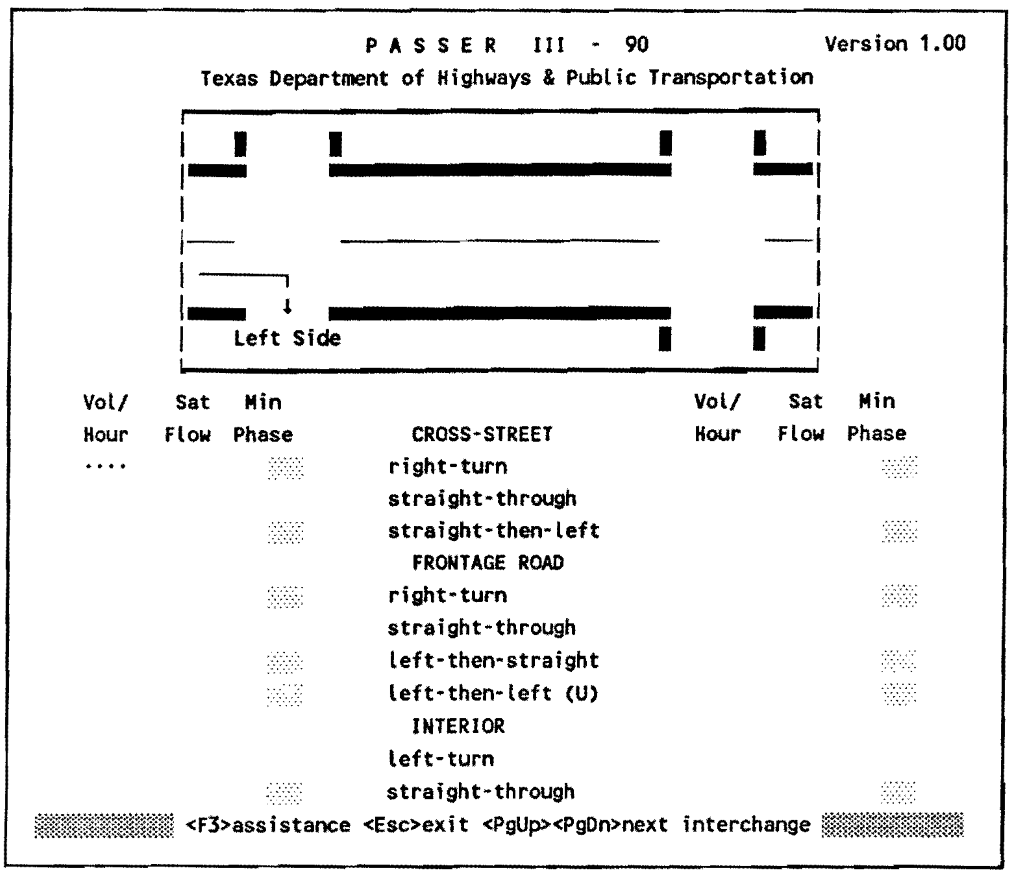


FIGURE A-2. INTERCHANGE MOVEMENT DATA INPUT SCREEN.

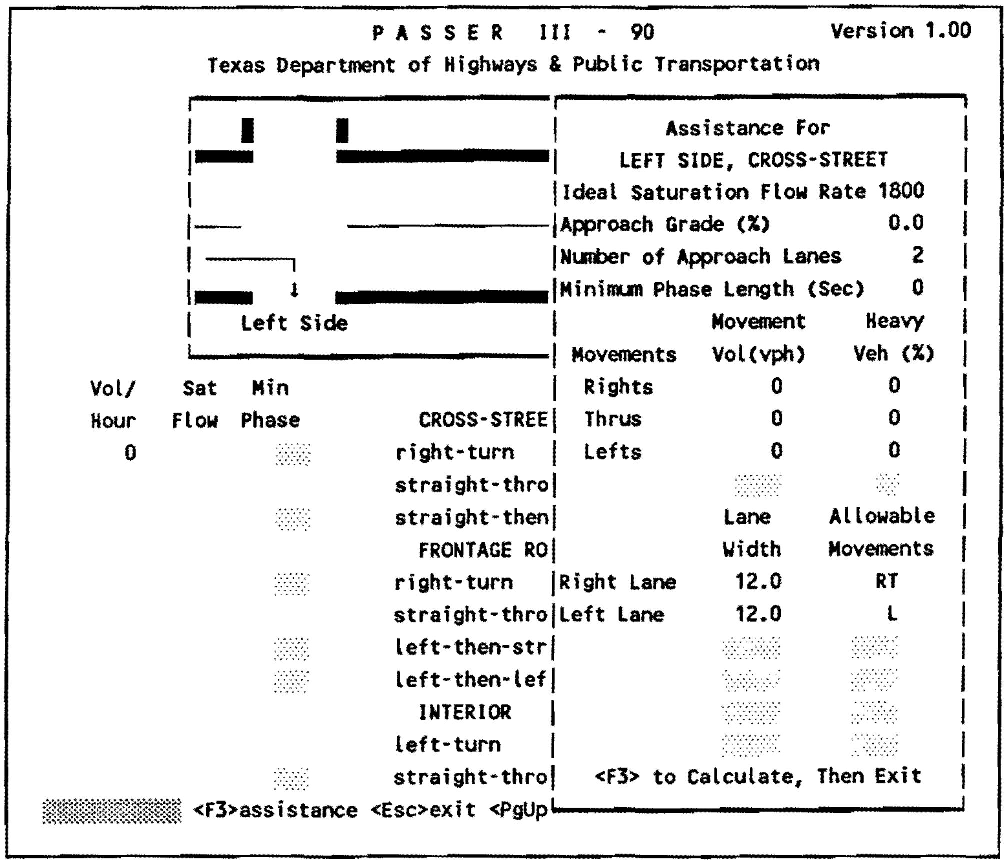


FIGURE A-3. SATURATION FLOW RATE ASSISTANCE WINDOW.

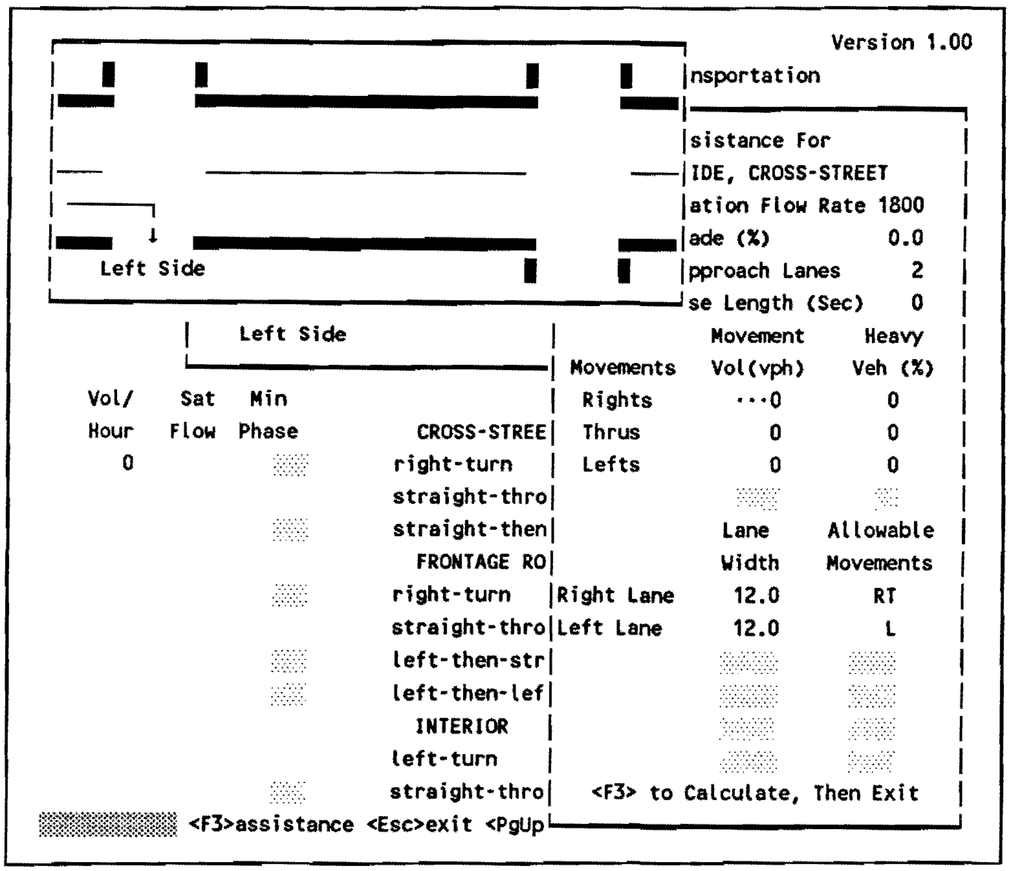


FIGURE A-4. INTERCHANGE MOVEMENT REFERENCE WINDOW.

Once the ideal saturation flow rate, approach grade, and number of approach lanes have been correctly entered on the first three data entry lines, the cursor should be moved to the movement volume and heavy vehicle percentage data entry lines. Note that positioning the cursor on the various lines in this portion of the window will cause a second window to appear in the upper left-hand corner of the movement screen (see Figure A-4). This second window illustrates the individual movement at the interchange for which data is being entered and will change as the cursor is moved up or down within this block. It should be noted that had volumes been entered on the input screen, those values would have appeared as defaults in the assistance window. The user can either accept or modify the default volumes, but upon exiting the assistance window, the volumes in the window will automatically be placed on the movement screen.

The bottom third of the saturation flow rate assistance window allows the user to specify the individual lane widths and the allowable movements from each of the lanes. Allowable movements per lane are specified by entering the first letter of the desired movement. At least one movement must be specified from each lane, more than one movement, however, may be specified from a single lane, and all movements for which non-zero volumes were entered, must be assigned to at least one lane. Figure A-5 illustrates the completed data entry for calculating saturation flow rates at the left-side cross-street. After pressing either the F3 or Esc key, the Interchange Movement Data input screen will appear as shown in Figure A-6. The user has the option of changing any of the calculated values at this point.

Figures A-7 through A-16 illustrate the saturation flow rate assistance window and the resultant Interchange Movement Data input screen for the remaining five blocks of data. One item that should be mentioned is the saturation flow rate calculations for the right-side cross-street movements. In Figure A-7, the left lane at the right-side cross-street is marked as an optional straight-through/straight-then-left lane; however, the straight-then-left traffic is high enough that straight-through vehicles will probably avoid this lane. Thus, the left lane will probably operate as an exclusive lane for straight-then-left traffic, and the saturation flow rate for this movement will be the saturation flow rate for the left lane. Note, as shown in Figure A-8, that the saturation flow rate assistant checks for these de facto lanes and calculates appropriate saturation flow rates automatically.

In summary, existing volumes from the Interchange Movement Data input screen are automatically read into the saturation flow rate assistance window each time it is accessed by the user. Saturation flow rates are automatically calculated each time the user exits the assistance window, and the calculated values placed in the appropriate data entry fields in the Interchange Movement Data input screen. The data used in the saturation flow rate calculations is saved as a permanent part of the data file.

```
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```
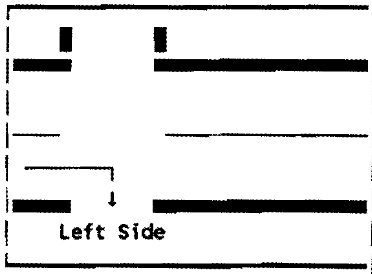
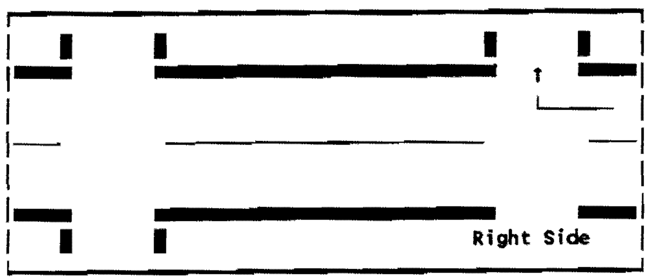




```
                                          Assistance For
                                      LEFT SIDE, CROSS-STREET
                                     |Ideal Saturation Flow Rate 1800
                                     |Approach Grade (%)          0.0
                                     |Number of Approach Lanes      2
                                     |Minimum Phase Length (Sec)    0
                                     |              Movement   Heavy
                                     | Movements    Vol(vph)   Veh (%)
Vol/    Sat   Min                    | Rights          35         0
Hour    Flow  Phase      CROSS-STREE | Thrus          144         0
 0                       right-turn  | Lefts          125         0
                         straight-thro|
                         straight-then|              Lane     Allowable
                           FRONTAGE RO|              Width    Movements
                         right-turn  |Right Lane     12.0        RT
                         straight-thro|Left Lane     12.0        LT
                         left-then-str|
                         left-then-lef|
                           INTERIOR   |
                         left-turn    |
                         straight-thro|  <F3> to Calculate, Then Exit
            <F3>assistance <Esc>exit <PgUp
```

FIGURE A-5. SATURATION FLOW RATE DATA FOR LEFT-SIDE CROSS-STREET.

```
                    P A S S E R   III  -  90          Version 1.00
            Texas Department of Highways & Public Transportation
```




```
Vol/    Sat   Min                              Vol/    Sat    Min
Hour    Flow  Phase     CROSS-STREET           Hour    Flow   Phase
 35     407            right-turn              ....
144    1676     0      straight-through
125    1455            straight-then-left
                          FRONTAGE ROAD
                       right-turn
                       straight-through
                       left-then-straight
                       left-then-left (U)
                          INTERIOR
                       left-turn                125      0      0
                       straight-through         144      0
            <F3>assistance <Esc>exit <PgUp><PgDn>next interchange
```

FIGURE A-6. INTERCHANGE MOVEMENT DATA FOR LEFT-SIDE CROSS-STREET.

```
                         P A S S E R   III  -  90          Version 1.00
                  Texas Department of Highways & Public Transportation

      Assistance For
   RIGHT SIDE, CROSS-STREET
|Ideal Saturation Flow Rate 1800
|Approach Grade (%)          0.0
|Number of Approach Lanes      2
|Minimum Phase Length (Sec)    0
|              Movement    Heavy
| Movements    Vol(vph)    Veh (%)
|  Rights        228        0      |                     Vol/    Sat    Min
|  Thrus         102        0      |S-STREET             Hour    Flow   Phase
|  Lefts         602        0      |turn                   0
|                                  |ht-through
|               Lane     Allowable |ht-then-left
|               Width    Movements |TAGE ROAD
|Right Lane     12.0        RT     |turn
|Left Lane      12.0        LT     |ht-through
|                                  |hen-straight
|                                  |hen-left (U)
|                                  |RIOR
|                                  |urn                  125      0      0
|  <F3> to Calculate, Then Exit    |ht-through           144      0
                                    t <PgUp><PgDn>next interchange
```

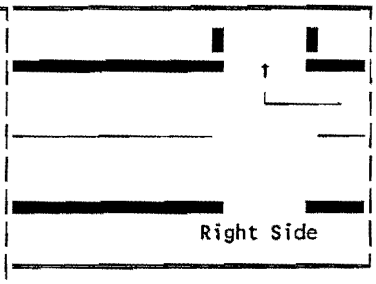


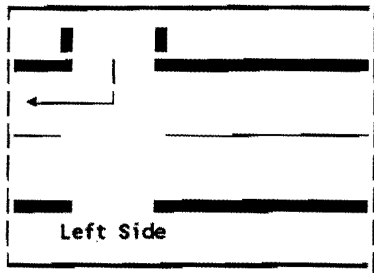
FIGURE A-7. SATURATION FLOW RATE DATA FOR RIGHT-SIDE CROSS-STREET.

```
                         P A S S E R   III  -  90          Version 1.00
                  Texas Department of Highways & Public Transportation
```


```
 Vol/   Sat   Min                               Vol/   Sat   Min
 Hour   Flow  Phase        CROSS-STREET         Hour   Flow  Phase
  35    407              right-turn              228   1114
 144   1676     0        straight-through        102    499     0
 125   1455              straight-then-left      602   1800
                           FRONTAGE ROAD
 ....                    right-turn
                         straight-through
                         left-then-straight
                         left-then-left (U)
                           INTERIOR
 602     0      0        left-turn               125      0     0
 102     0               straight-through        144      0
          <F3>assistance <Esc>exit <PgUp><PgDn>next interchange
```

FIGURE A-8. INTERCHANGE MOVEMENT DATA FOR RIGHT-SIDE CROSS-STREET.

```
                    P A S S E R   III  -  90            Version 1.00
              Texas Department of Highways & Public Transportation
```




```
                                    Assistance For
                                 LEFT SIDE, FRONTAGE ROAD
                             Ideal Saturation Flow Rate 1800
                             Approach Grade (%)          0.0
                             Number of Approach Lanes      3
                             Minimum Phase Length (Sec)    0
                                               Movement    Heavy
                              Movements        Vol(vph)    Veh (%)
Vol/   Sat   Min              Rights              8          0
Hour   Flow  Phase  CROSS-STREE  Thrus           42          0
 35    407          right-turn   Lefts           291         0
144    1676   0     straight-thro  U-Turns        95         0
125    1455         straight-then            Lane     Allowable
                    FRONTAGE RO              Width    Movements
  0                 right-turn    Right Lane   12.0      RT
                    straight-thro Middle Lane  12.0      T
                    left-then-str Left Lane    12.0      LU
                    left-then-lef
                    INTERIOR
602     0     0     left-turn
102     0           straight-thro   <F3> to Calculate, Then Exit
        <F3>assistance <Esc>exit <PgUp
```

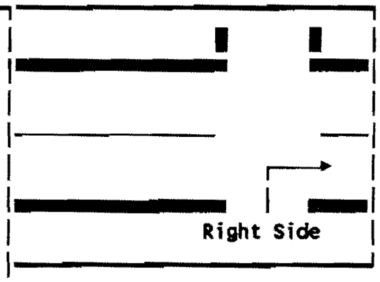
FIGURE A-9. SATURATION FLOW RATE DATA FOR LEFT-SIDE FRONTAGE ROAD.

```
                    P A S S E R   III  -  90            Version 1.00
              Texas Department of Highways & Public Transportation
```


```
Vol/   Sat   Min                            Vol/   Sat   Min
Hour   Flow  Phase   CROSS-STREET           Hour   Flow  Phase
 35    407           right-turn              228   1114
144    1676   0      straight-through        102    499    0
125    1455          straight-then-left      602   1800
                     FRONTAGE ROAD
  8    562           right-turn             ....
 42    2952   0      straight-through
291    1289          left-then-straight
 95    421           left-then-left (U)
                     INTERIOR
602     0     0      left-turn               220     0     0
102     0            straight-through        435     0
        <F3>assistance <Esc>exit <PgUp><PgDn>next interchange
```

FIGURE A-10. INTERCHANGE MOVEMENT DATA FOR LEFT-SIDE FRONTAGE ROAD.

```
                P A S S E R   III  -  90             Version 1.00
         Texas Department of Highways & Public Transportation
```

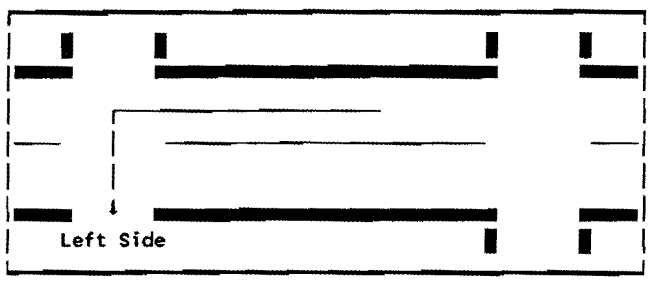



```
         Assistance For
     RIGHT SIDE, FRONTAGE ROAD
Ideal Saturation Flow Rate 1800
Approach Grade (%)          0.0
Number of Approach Lanes      3
Minimum Phase Length (Sec)    0
              Movement     Heavy
 Movements    Vol(vph)    Veh (%)
  Rights        139         0
  Thrus         302         0
  Lefts         200         0
  U-Turns       107         0
               Lane     Allowable
               Width    Movements
Right Lane     12.0        RT
Middle Lane    12.0        T
Left Lane      12.0        LU

   <F3> to Calculate, Then Exit
```

```
                      Vol/    Sat    Min
                      Hour    Flow   Phase
S-STREET
turn                  228     1114
ht-through            102      499     0
ht-then-left          602     1800
TAGE ROAD
turn                    0
ht-through
hen-straight
hen-left (U)
RIOR
urn                   220        0     0
ht-through            435        0
t <PgUp><PgDn>next interchange
```

FIGURE A-11. SATURATION FLOW RATE DATA FOR RIGHT-SIDE FRONTAGE ROAD.

```
                P A S S E R   III  -  90             Version 1.00
         Texas Department of Highways & Public Transportation
```



Vol/ Hour	Sat Flow	Min Phase		Vol/ Hour	Sat Flow	Min Phase
			CROSS-STREET			
35	407		right-turn	228	1114	
144	1676	0	straight-through	102	499	0
125	1455		straight-then-left	602	1800	
			FRONTAGE ROAD			
8	562		right-turn	139	1081	
42	2952	0	straight-through	302	2349	0
291	1289		left-then-straight	200	1114	
95	421		left-then-left (U)	107	596	
			INTERIOR			
-709	0	0	left-turn	220	0	0
302	0		straight-through	435	0	

<F3>assistance <Esc>exit <PgUp><PgDn>next interchange

FIGURE A-12. INTERCHANGE MOVEMENT DATA FOR RIGHT-SIDE FRONTAGE ROAD.

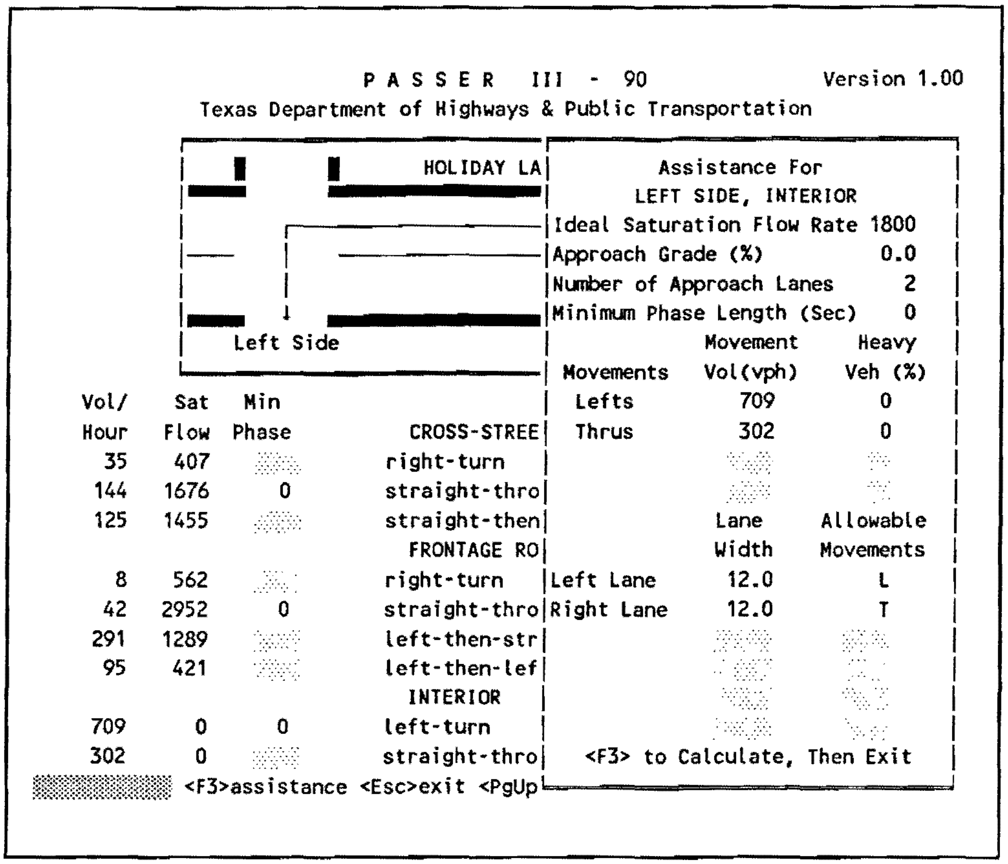


FIGURE A-13. SATURATION FLOW RATE DATA FOR LEFT-SIDE INTERIOR MOVEMENTS.

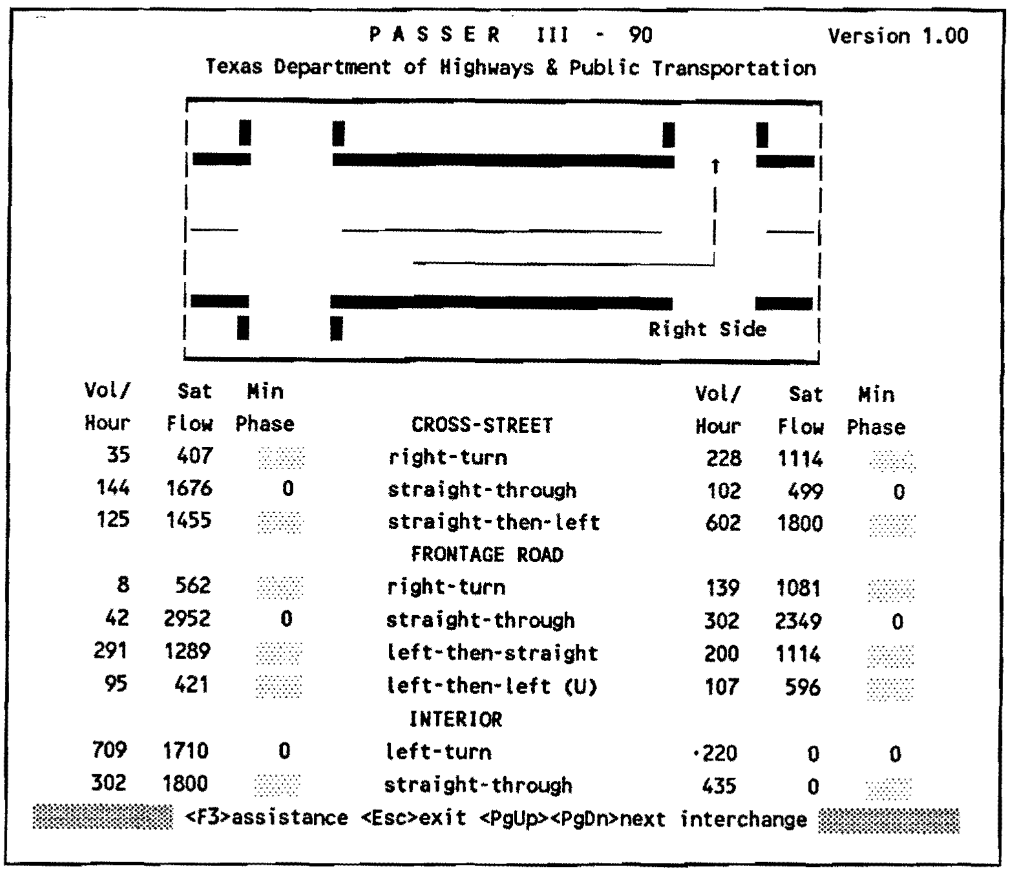


FIGURE A-14. INTERCHANGE MOVEMENT DATA FOR LEFT-SIDE INTERIOR MOVEMENTS.

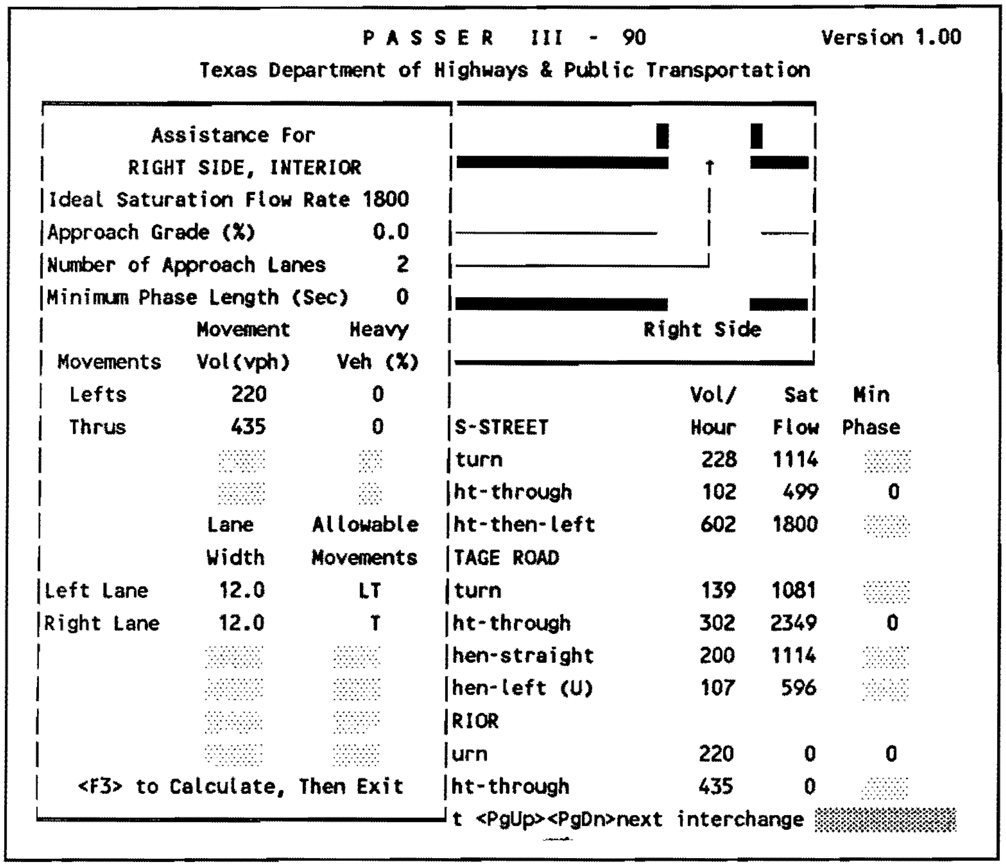


FIGURE A-15. SATURATION FLOW RATE DATA FOR RIGHT-SIDE INTERIOR MOVEMENTS.

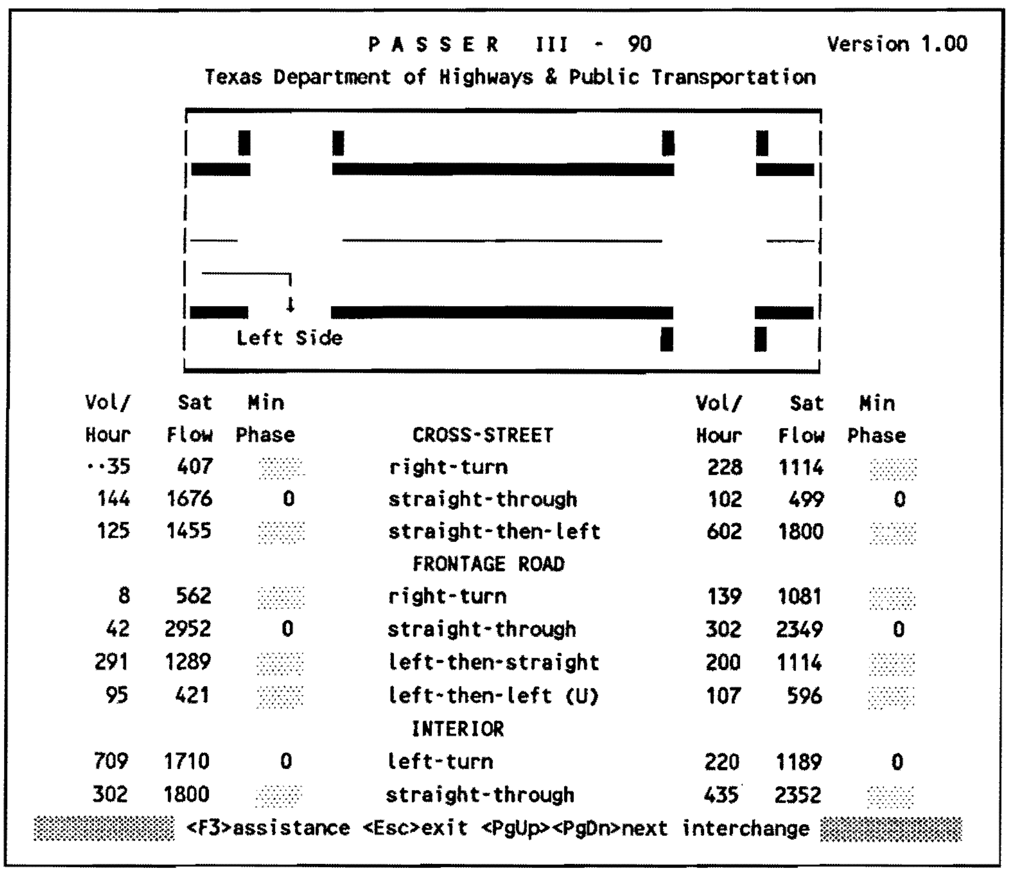


FIGURE A-16. INTERCHANGE MOVEMENT DATA FOR RIGHT-SIDE INTERIOR MOVEMENTS.