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16. Abstract

This document provides guidelines and procedures for the retiming of diamond interchanges, including procedures for data collection, and the types and amount of data to be collected; the analytical procedures and software packages that are available for signal retiming projects; and examples of step-by-step applications for each type of signal timing projects.

Section One, Introduction, provides background information and discusses how to determine if a diamond interchange should be retimed. Section Two, Diamond Interchanges, discusses the characteristics of a diamond interchange, including the types of phasing, controllers, measures of effectiveness, and signal timing methods. Section Three, Data Requirements, discusses the procedures and requirements for data collection at a diamond interchange.

Section Four, Evaluation, provides information needed for the simulation of existing interchange operations, including the use of currently available software programs. Section Five, Optimization, provides the guidelines to optimize timings at a typical diamond interchange. Section Six, Implementation, explains the interpretation of optizimized signal plans and implementation in the field. Section Seven, Documentation, explains how the benefits of signal retiming can be calculated, and provides examples of possible benefits.

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IMPLEMENTATION GUIDELINES FOR RETIMING DIAMOND INTERCHANGES

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	2.54	centimeters	cm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	6.452	centimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.395	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	millimeters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0328	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

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

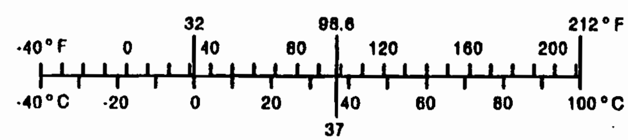
Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

These factors conform to the requirement of FHWA Order 5190.1A

*SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
yd	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
yd^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.53	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	millimeters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



IMPLEMENTATION STATEMENT

The objective of this study is to place in a single set of documents, implementation guidelines for traffic signal retiming projects in Texas. These documents include the types and amounts of data to be collected, and the procedures for doing so; the analytic procedures and software packages that are available and the types of projects for which they are suited; and examples featuring step-by-step applications for several typical signal retiming projects in Texas. This set of documents also includes field implementation and evaluation guidelines. Specific types of signal retiming projects addressed by this study are as follows:

1164-1 Implementation Guidelines for Retiming Isolated Intersections;

1164-2 Implementation Guidelines for Retiming Arterial Streets;

1164-3 Implementation Guidelines for Retiming Diamond Interchanges;

1164-4 Implementation Guidelines for Retiming Arterial Networks; and

1164-5 Implementation Guidelines for Retiming Freeway Corridors.

The objective of this document is to provide implementation guidelines and procedures for retiming signalized diamond interchanges.

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 Texas Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation and is NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES.

ACKNOWLEDGEMENTS

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1.0 INTRODUCTION

1.1 Background

With both urban congestion and the available funding continuing to worsen in Texas cities, Texas Department of Transportation (TxDOT) engineers face a growing problem of developing low-cost solutions for increasing the capacity of their signalized intersections and arterial streets. The State's assumption of the maintenance of those traffic signals in cities between 15 and 50 thousand in population and at freeway interchanges, together with the initiation of the Primary Arterial Street System (PASS) program for larger cities, adds to the magnitude of the problem.

Some of the lowest cost methods of dealing with capacity problems are traffic signal retiming projects. Signal optimization and retiming projects have received increased attention as cost-effective and transportation systems management (TSM) measures. Results from several studies have demonstrated that one can achieve substantial energy savings through the development of improved timing plans on existing signal systems. Also, unnecessary delays and stops at traffic signals are eliminated, resulting in travel time savings for the public.

The development of efficient signal settings requires detailed data collection of traffic and geometric conditions, application of improved methods to optimize the signal timing plan, and field implementation and evaluation of the improved signal timings. Several techniques and computer programs have been developed, and are available to traffic signal analysts to analyze existing conditions and optimize signal timing, thus minimizing delays, fuel consumption, and stops and improving traffic progression.

Because of the diversity of retiming projects and the number of techniques and tools available, no single procedure or set of guidelines applies to all projects. Field implementation and evaluation guidelines also are virtually nonexistent in the literature. In addition, most districts do not undertake such projects on a routine basis. For these reasons, it would benefit traffic signal analysts if a set of guidelines and procedures for several types of typical traffic signal retiming projects were available to each district. These guidelines should cover not only the development of new timing plans, but also their subsequent implementation and evaluation.

1.2 Objectives

This study places implementation guidelines for traffic signal retiming projects in a single set of documents. These documents would include the types and amounts of data to be calibrated and the procedures for collecting them; the analytic procedures and software packages available and the types of projects to which they apply; and examples featuring step-by-step applications for several typical traffic signal retiming projects in Texas. This set of documents also includes field implementation and evaluation guidelines. Specific types of retiming projects addressed are as follows:

1164-1	Implementation Guidelines for Retiming Isolated Intersections;
1162-4	Implementation Guidelines for Retiming Arterial Streets;
1164-3	Implementation Guidelines for Retiming Diamond Interchanges;
1164-4	Implementation Guidelines for Retiming Arterial Networks; and
1165-5	Implementation Guidelines for Retiming Freeway Corridors.

This document provides implementation guidelines and procedures for retiming signalized diamond interchanges. It includes the types and amounts of data to be collected, and the procedures for collecting them; the analytic procedures and software packages available for a particular project; and examples of step-by-step applications for each type of diamond interchange signal timing project.

1.3 Organization

This document provides guidelines and procedures for developing and implementing traffic signal retiming plans at signalized diamond interchanges. Separate documents address other types of traffic signal retiming projects. The guidelines and procedures for diamond interchange traffic signal retiming projects are organized as follows:

1.4 When to Retime Signals

Public complaints are usually the first signs of traffic signal operational problems. Signal retiming cannot address all complaints, but several complaints may indicate a need for at least a field observation or study. Some common complaints include: excessive approach delay, left turn delay, poor progression and excessive queues. Traffic signal analysts make field observations to determine the legitimacy of the complaints. Major problems will be obvious to the observer, such as long queues, ineffective use of green times and excessive cycle lengths (greater and 150 seconds). In some cases, equipment, such as detectors, may need repair. If one rules out these problems, retiming may improve the signal's operational efficiency. As a rule of thumb, one should make field observations or studies every three to five years to determine if signal retiming is necessary.

Changes in traffic flow caused by land use and population changes, addition or deletion of signals in the area, change in major traffic generators, and changes in geometrics of the roadway or diamond interchange may create the need for retiming signals. Some jurisdictions recommend a yearly inspection and documentation (by field data and/or video) of signal operations. This documentation will help identify operational problems before they become severe.

2.0 DIAMOND INTERCHANGES

2.1 Characteristics

Several types of interchanges can be classified as diamonds; however, their common feature is two closely spaced intersections that connect either entrance and exit ramps or parallel frontage roads to an arterial or cross-street. Although exit ramps typically occur prior to the cross-street and entrance ramps typically occur after the cross-street, their order of occurrence can be reversed in what is commonly referred to as a reverse diamond. If the distance between successive interchanges is short, the entrance and exit ramps between interchanges are sometimes braided or grade separated (x-ramps).

The most common type of diamond interchange found in Texas cities is the conventional full diamond interchange with parallel one-way frontage roads. Figure 2-1 shows examples of full and other common types of diamond interchanges. Some full diamond interchanges with one-way frontage roads have U-turn (turnaround) lanes for heavy left-then-left-turn traffic from the frontage roads. One may operate diamond interchanges in the pretimed or actuated mode and as isolated interchanges or in coordination with other interchanges to allow progression along frontage roads. Guidelines in this document however, only address isolated diamond interchanges.

Some of the operational problems that occur at diamond interchanges as the result of increasing traffic demand include:

1. Queue spillback from one of the intersections at the interchange, which may result in the blockage of the upstream intersection by queued vehicles;

2. The left-turn lane in the interior section of the interchange overflows and spills into the through lane;

3. Off-ramp queue spillback, which occurs when a long queue of vehicles backs up into the freeway; and

4. Weaving problems on the frontage road between the ramp termini and the arterial cross-street.

The following sections describe diamond interchange phasing, terminology, signal timing philosophy, control strategies, and measures of effectiveness.

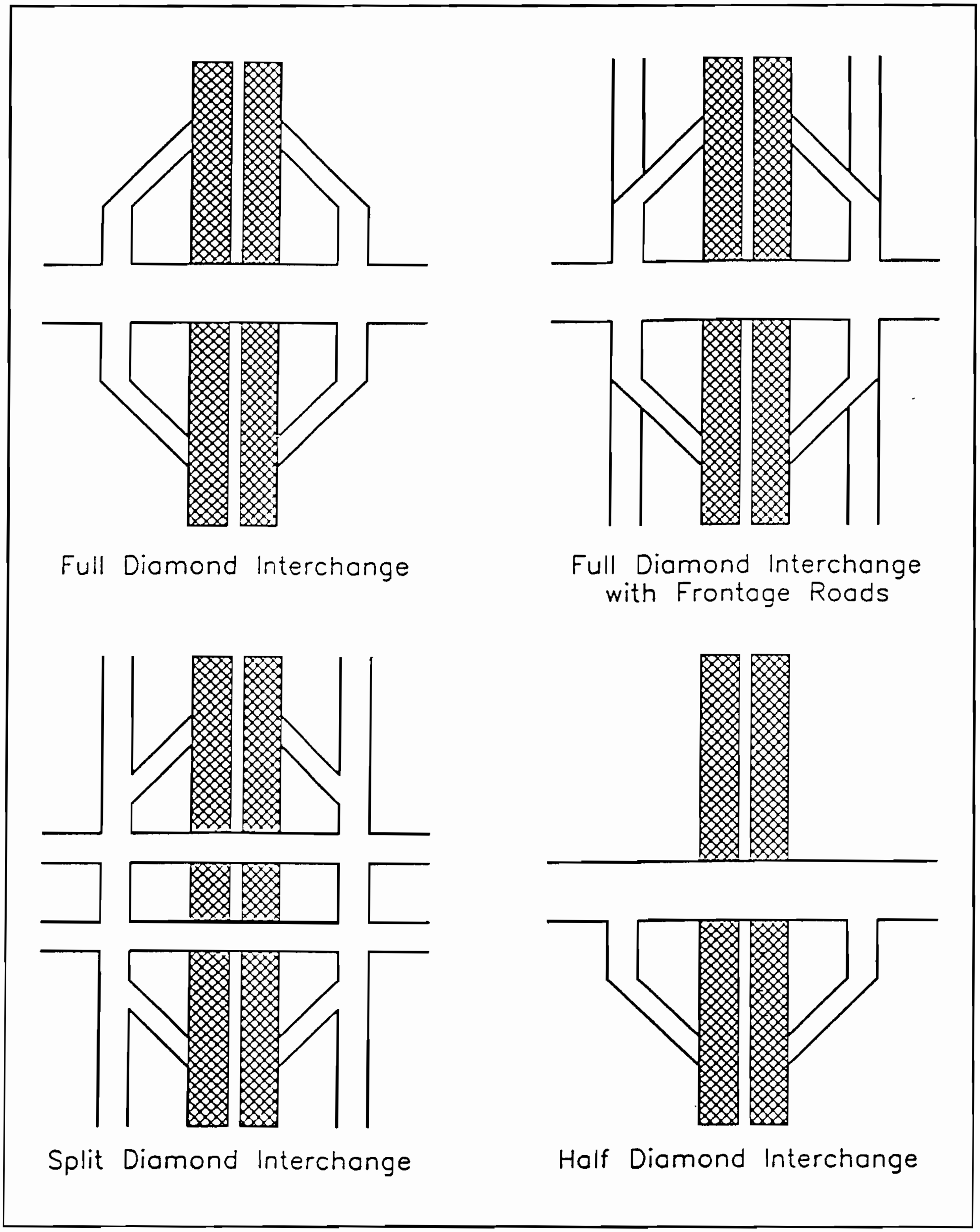


Figure 2-1. Types of Diamond Interchanges

2.2 Diamond Interchange Phasing

Several phasing strategies exist which one may use at signalized diamond interchanges. One may classify each phase pattern by the number of basic phases and the sequence of movements at the diamond interchange. The basic phase configurations are two-phase, three-phase, and four-phase. The following sections discuss phasing types, phase sequences, and left-turn treatment alternatives.

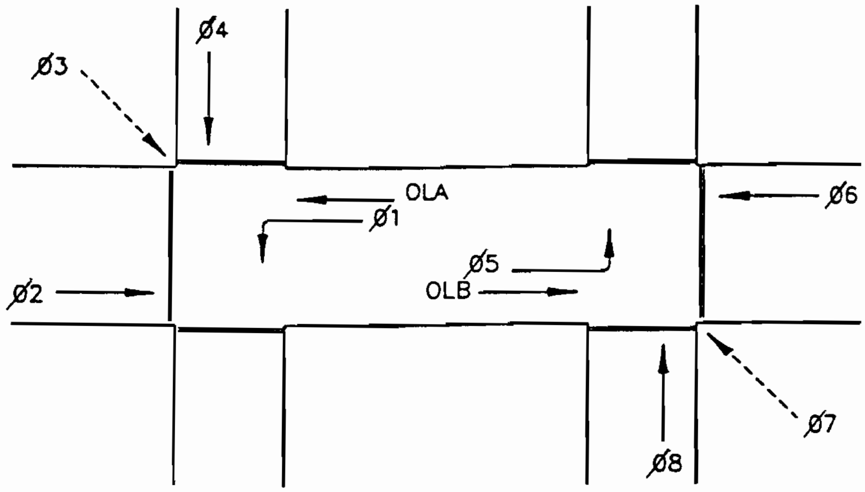
Phasing Types. The number of basic phases and the method by which one calculates green splits, differentiates between two-phase, three-phase and four-phase control at a diamond interchange. Figure 2-2 shows the basic movements and corresponding PASSER III and NEMA phase designations at a diamond interchange.

For two-phase control, one treats the diamond interchange as two separate intersections, each having two basic phases. These two phases are the arterial or cross-street phase (Phase 2 or 6) and the ramp or frontage road phase (Phase 4 or 8). Protected left-turn phases for the interior movements are not provided. Although not required, the arterial phase lengths at both intersections usually are the same duration, which means that the ramp phase lengths at both intersections are also the same duration.

For three-phase control, one treats the diamond interchange as two separate intersections, each having three basic phases. The three phases are the arterial or cross-street phase (Phase 2 or 6), the ramp or frontage road phase (Phase 4 or 8), and the interior left-turn phase (Phase 1 or 5). Protected left-turn phases for the interior movements (Phase 1 or 5) are provided. As with two-phase control, the arterial phase lengths at both intersections usually are the same duration, and the ramp phase lengths at both intersections usually are the same duration, which means that the additional left-turn phase lengths are also the same duration.

For four-phase control, one treats the diamond interchange as a single intersection which has four basic phases. The four basic phases are two exterior movements on the arterial or cross-street (Phases 2 and 6) and two exterior movements on the ramp or frontage road (Phases 4 and 8). Protected left-turn phases for the interior movements (Phases 1 and 5) are provided; however, their duration is determined by subtracting the sum of the two exterior phases at the intersection from the desired cycle length. The remaining green time is allocated to the interior left-turn movement at each intersection.

A subset of four-phase control is "four-phase with two overlaps," or TTI lead-lead phasing, and is discussed further in the following sections. Basically, however, the length of the two overlap phases is set so as to provide progression between the two interchange signals, thus minimizing interior delay, stops, and queuing. The interior movements are generally afforded a better level of service than the exterior movements, and slightly longer cycle lengths are needed for four-phase with two overlaps to work well.



Left-Side	PASSER Desg.	NEMA Desg.	Right-Side	PASSER Desg.	NEMA Desg.
Cross-Street (Arterial)	A	2	Cross-Street (Arterial)	A	6
Frontage Road (Ramp)	B	4	Frontage Road (Ramp)	B	8
Interior Lefts	C	1	Interior Lefts	C	5

Figure 2-2. Movements at a Diamond Interchange

Phase Sequence. One may further classify phasing patterns by the order in which the interior left turn movement proceeds in relation to the arterial street movement on the same side of the interchange. Four basic phasing patterns are possible at a diamond interchange:

- Lead-lead: protected left-turn movements from the interior lanes lead the opposing arterial phase at both intersections;
- Lead-lag: protected left-turn movements from the interior lanes lead the opposing arterial phase at the left intersection and lag the opposing arterial phase at the right intersection;
- Lag-lead: the mirror image of the lead-lag phasing pattern; and
- Lag-lag: protected left-turn movements from the interior lanes lag the opposing arterial phase at both intersections.

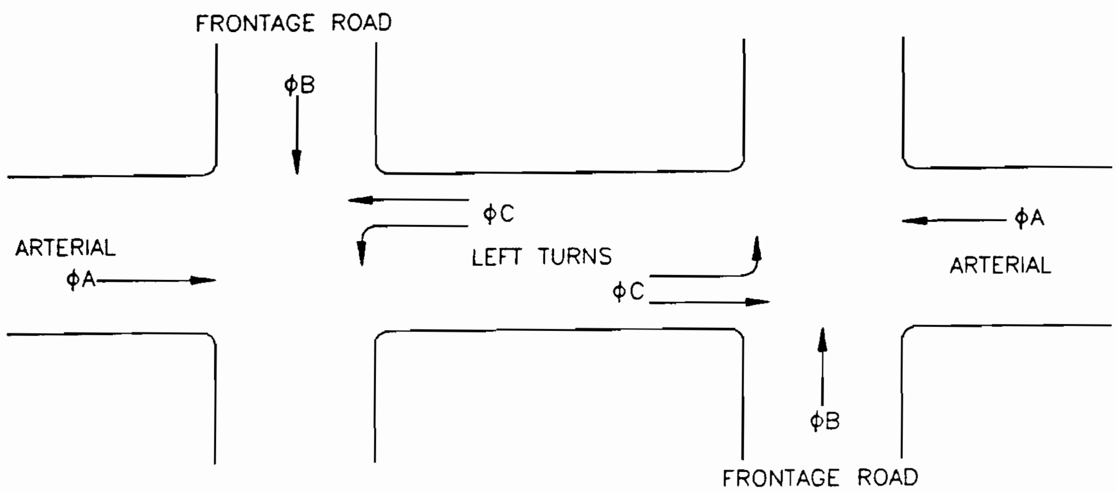
Figure 2-3 illustrates each of the possible phase sequences. Note that two of the phase sequences shown in the figure are lead-lead, the difference being that one calculates the splits for the latter sequence using the four-phase with two overlap timing philosophy.

Left-Turn Treatments. The interior left turn phases (Phase 1 or 5) may be protected only, protected plus permitted, or permitted only. In the permitted only case, these phases would not exist; i.e., their duration would equal zero. This alternative is desirable if a large number of acceptable gaps in the opposing traffic stream and adequate sight distance exist. By allowing permitted left turns, one may increase the overall capacity of the interchange by allowing some green time normally allocated to left-turning vehicles to be allocated to the other movements.

2.3 Terminology and Application

In Texas, a special terminology for describing phasing patterns has become popular. The term "Figure XX" denotes these patterns, where "XX" is the number 3, 4, 6, or 7 (1). "Figure 3" refers to all lag-lag phasing patterns. "Figure 4" refers to all "lead-lead" phasing patterns, of which four-phase with overlap is a subset. "Figure 6" refers to lead-lag phasing patterns, and "Figure 7" refers to lag-lead phasing patterns.

The terms "Figure 3" and "Figure 4" originated because the most popular phase patterns implemented were the "three-phase" pattern with simultaneous left turns and the "four phase with overlaps" phasing plan. The names "Figure 6" and "Figure 7" are arbitrary and do not refer to six-phase or seven-phase patterns. The following sections discuss each phasing pattern in more detail:



LEFT SIDE PHASE SEQUENCE			LEFT TURN SEQUENCE	RIGHT SIDE PHASE SEQUENCE		
A	B	C	LEAD–LEAD	A	B	C
A	C	B	LAG–LEAD	A	B	C
A	B	C	LEAD–LAG	A	C	B
A	C	B	LAG–LAG	A	C	B
A	B	C	TTI–LEAD	A	B	C

Figure 2-3. Possible Phase Sequences

Lead-Lead or "Figure 4." This phasing pattern has become the preferred phasing plan for most diamond interchanges because, when one selects proper splits and offsets, it allows almost all traffic movements to proceed through the intersection without additional stops; however, several methods exist in which lead-lead phasing can be implemented.

The most common subset of the "Figure 4" pattern is the "four-phase with overlaps" pattern, also referred to as "TTI-lead." This pattern provides progression for all movements, eliminating storage of left-turn vehicles in the center of the interchange except frontage road U-turn vehicles. Calculations of the exterior and interior phase times are based on travel time between intersections and lost time per phase. See Section 2.4, "Diamond Interchange Signal Timing Philosophy," for split timing equations.

Figure 2-4 illustrates a pretimed four-phase sequence and an actuated four-phase sequence. Phase 1 of four-phase with overlaps is one frontage road leading followed by the overlap phase. Phase 2 is the inbound cross-street movement clockwise from Phase 1. Phase 3 is the other frontage road movement followed by Phase 3 overlap and Phase 4 is the arterial inbound movement. Overlap phases refer to the two normally conflicting movements receiving a green indication at the same time (i.e., an arterial phase at one intersection and a ramp phase at the other intersection, (Phases 4 and 6 or 2 and 8) both having a green indication). The overlap phases are of fixed length, and their sum usually equals one to two seconds less than twice the travel time between the two intersections.

Because four-phase with overlaps minimize the number of vehicles stopping within the interchange, they are generally recommended for isolated diamond interchanges, with one-way ramps or frontage road intersections spaced less than 200 feet apart (i.e., closely spaced intersections). For intersections spaced 200 to 400 feet apart, volume levels and turning movement patterns dictate whether four-phase control with overlaps will work well. Generally, however, four-phase with overlap control works well for heavy unbalanced ramp (frontage road) traffic and intersection spacing up to 400 feet.

Lag-Lag or "Figure 3." Three-phase operation tends to produce less overall delay when adequate interior storage is available. Thus, traffic signal analysts generally recommend this type of phasing for diamond interchanges with moderate to high traffic volumes, wide spacings between the two intersections, and high through volumes on either the cross-street or the frontage road. Unlike four-phase operation, one treats the two sides of the interchange as separate intersections each having three basic phases. The three phases that exist at each intersection are the arterial or cross-street phase, the frontage road or ramp phase, and the interior left-turn phase. Left-turns are made during a protected phase (green arrow indication).

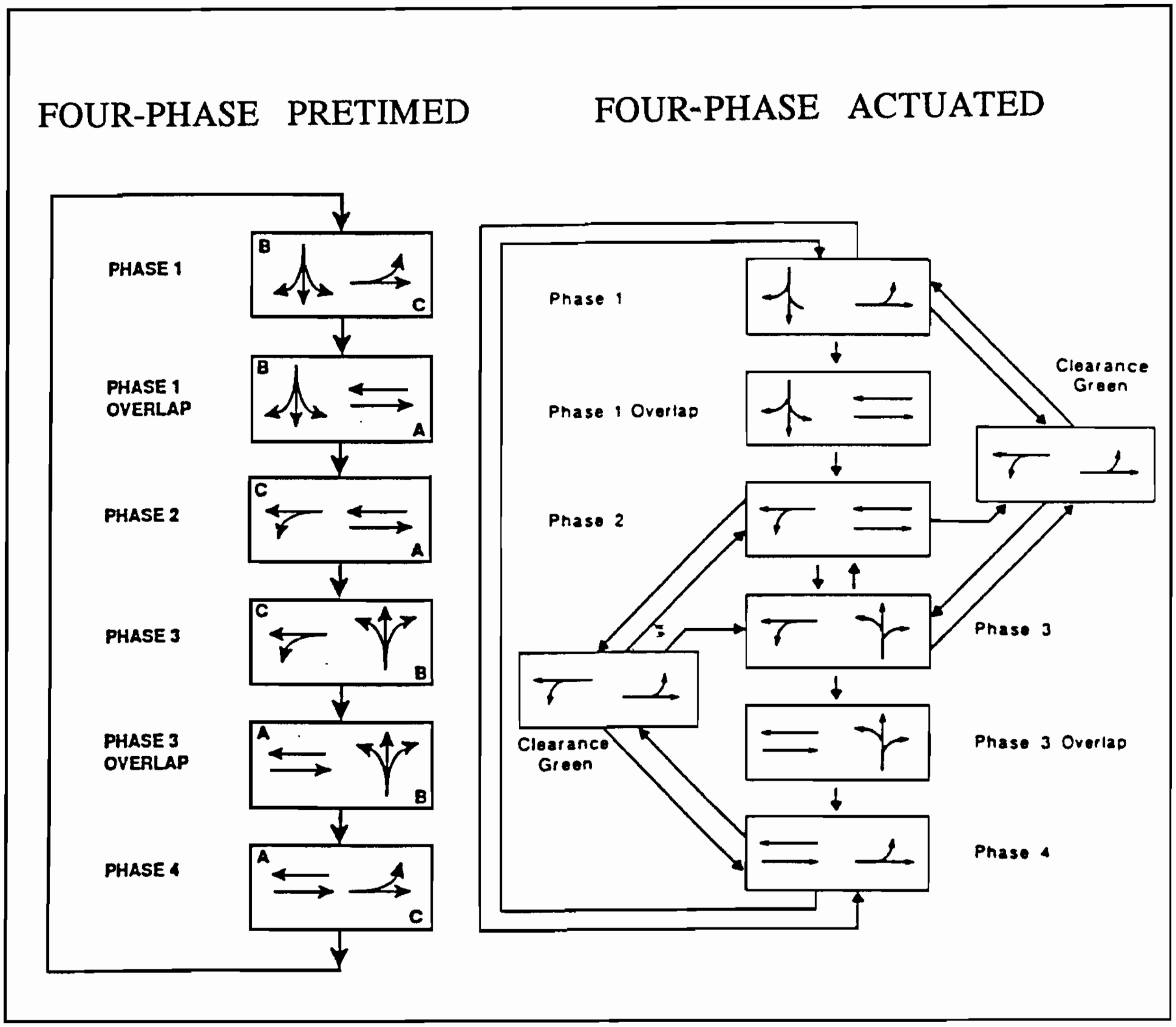


Figure 2-4. Four Phase Pretimed and Actuated Signal Phasing

Analysts may implement three-phase operation with either pretimed or actuated controllers. Although only one controller is necessary for this type of phasing, one may use two controllers (one at each intersection). The new generation of diamond interchange controllers, however, have programmable offsets and overlaps, which add a great deal of flexibility to a single controller design and eliminate the need for a two controller design. With either type of controller arrangement, three primary phases exist, with six possible subordinate phases (see Figure 2-2). During Phase 1, both frontage roads proceed at the same time, followed by Phase 2, the cross-street phase (inbound-outbound phase without protected left turns). Phase 3 is the simultaneous display of protected turn signals for the internal movements. Figure 2-5 shows a three-phase pretimed system and the basic three-phase configuration for actuated control.

When using the symmetrical phasing plan, care must be taken in allocating time to the frontage road phase. Otherwise, it is possible that an unexpected stoppage of vehicles in the interior of the intersection may occur, increasing the potential for accidents (1).

Lead-lag or "Figure 6." This phasing pattern favors heavy unbalanced traffic flow on the leading left turn or lagging frontage road side of the interchange. Heavy left-turn traffic from the right-side frontage road proceeds through the interior of the interchange without stopping.

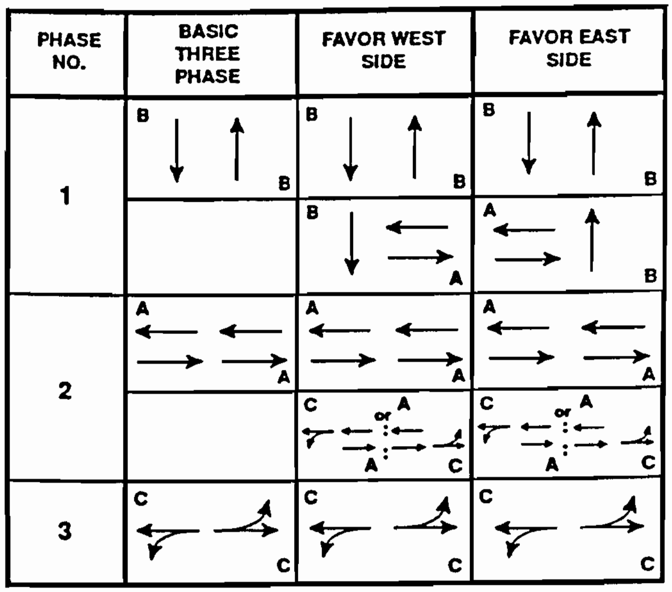
Lag-Lead or "Figure 7." This phasing pattern is the mirror image of "Figure 6." Heavy left-turn traffic from the left-side frontage road proceeds through the interior of the interchange without stopping.

Two-Phase. For signalized diamond interchanges operating under low volume conditions, two-phase operation may be a desirable option. As mentioned previously, the two-phases that exist at each intersection are the arterial or cross-street phase and the frontage road or ramp phase. The interior left-turn movements do not have a protected phase (green arrow indication), but proceed permissively during the arterial phase (green ball indication). The benefits of this type of phasing are reduced delay due to fewer phases, and correspondingly shorter lost time and cycle lengths.

Two-phase operation is beneficial when the left-turn and/or opposing through traffic volumes are light; however, sufficient sight distance **must** be available to the left-turning vehicles to determine whether it is safe to make the turn. The protected left-turn phase is generally actuated, and will only be provided if left-turn vehicles are waiting in the left-turn lane; i.e., left-turn vehicles cannot receive green arrow without having to wait.

Each of the above mentioned phasing strategies prove advantageous for different traffic conditions and geometric conditions. Analysts should take advantage of these different phase sequences as traffic conditions fluctuate. An example of a system which addresses the ability to switch phase sequences based on traffic conditions is the Arlington phasing scheme.

THREE-PHASE PRETIMED



THREE-PHASE ACTUATED

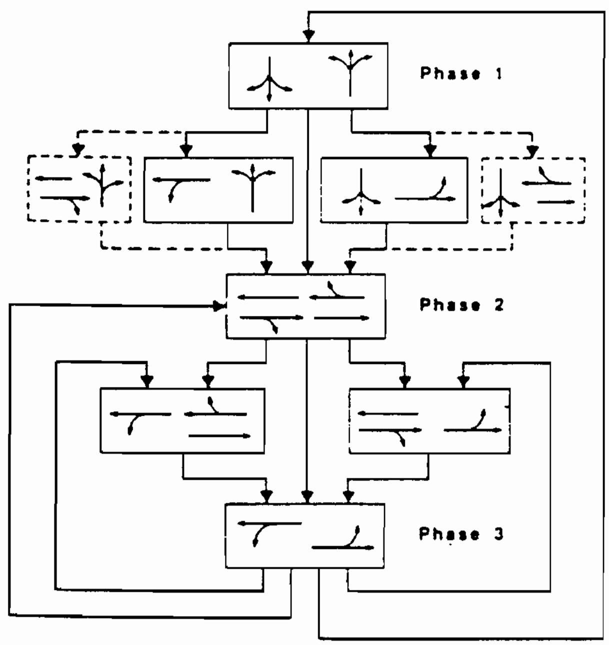


Figure 2-5. Three Phase Pretimed and Actuated Signal Phasing

Arlington Phasing. The Arlington phasing scheme (Arlington, Texas) (2) attempts to minimize delay for the system by minimizing the service to the interior left-turn movements by providing protected lefts only when substantial demand is present (use of protected-permitted left turns); thus, this phasing scheme usually operates at shorter cycle lengths. The detector configuration utilizes several sets of queue detectors, and the system can operate as either a three-phase or four-phase controller, depending on the length and location of the queues. Figure 2-6 illustrates the Arlington phasing configuration. Disadvantages of this phasing scheme are the tendency to have extremely long cycle lengths during periods of heavy traffic volumes. Such long cycle lengths occur because the queue detection may result in two or more phases being extended to maximum at the same time.

Interchange Spacing and Recommended Phasing. The following table summarizes diamond interchange widths and corresponding desirable phase sequences. Note that three-phase control can be either Figure 3, Figure 6, or Figure 7, depending upon the traffic and geometric conditions at the interchange.

Table 2-1 Interchange Width and Recommended Phasing[1]

Interchange Width (ft.)	Phasing
< 200	four-phase with two overlaps
200 - 400	three-phase or four-phase with two overlaps; depends on traffic volume distribution
> 400	three-phase

[1]Recommended phasing for various traffic volumes has been previously discussed.

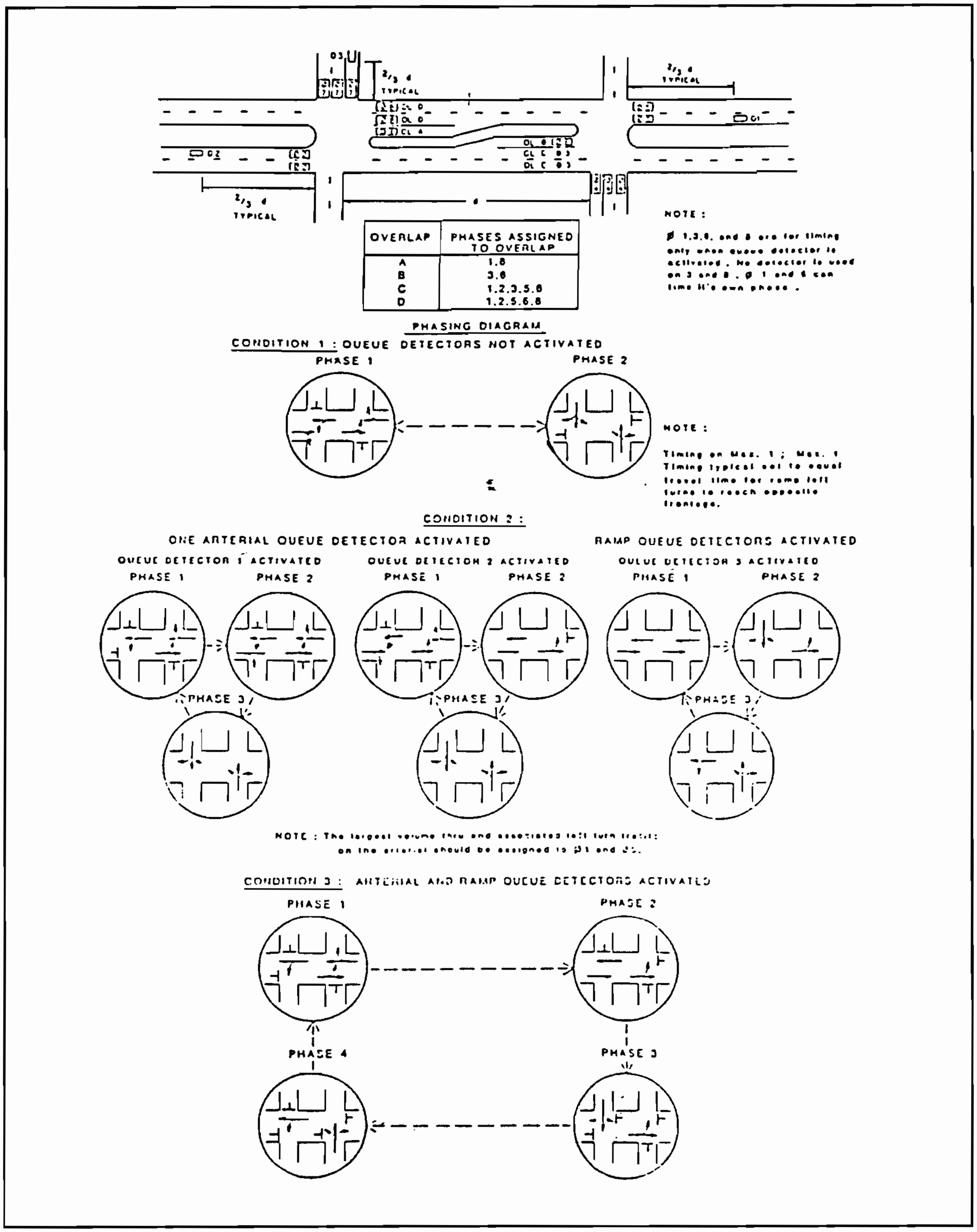


Figure 2-6. Arlington Phasing

2.4 Diamond Interchange Signal Timing Philosophy

The diamond interchange operates similar to two closely spaced signalized intersections. The distance between the two frontage roads determines the efficiency and phasing strategy of the signal system and the number of vehicles storable before spillback or gridlock occurs. The travel time to get from one frontage road to the next is the function of the distance between frontage roads. The problem with diamond interchanges, and at many other intersections, is that a high number of left-turning vehicles exist which, in most cases, require a protected left-turn phase. This additional phase reduces the time available for through vehicles.

Three-Phase Control. For three-phase control, one determines the green times for phases A (Phases 2 and 6), B (Phases 4 and 8), and C (Phases 5 and 1) for each side of the intersection by using Webster's formula (3):

$$G = (y/Y) * (C - \Sigma L) + L$$

where:

- G = phase green on approach, in seconds;
- y = critical flow ratio on the approach, q/s [(approach volume, veh/sec)/(approach saturation flow, veh/sec)]
- Y = sum of the three flow ratios at the intersection;
- C = cycle length, in seconds;
- ΣL = sum of intersection phase lost times, in seconds; and
- L = phase lost time, in seconds.

Four-Phase Control. The following equations relate the flow ratio, volume to capacity (v/c) ratio, cycle length, total lost time, and travel time for four-phase with overlap phasing:

For the four external movements:

$$Y_x = [X_x (C + 2\Phi - L_x)] / C$$

where:

- Y_x = Σy_i for the four external movements;
- X_x = critical v/c ratio for the four exterior movements;
- C = cycle length, in seconds;
- L_x = total lost time, usually 4 seconds per phase times the number of phases, in seconds; and
- Φ = overlap or interior travel time, in seconds.

For four-phase with dual overlaps, the following relationships between travel time and lost time hold true in terms of average delay per vehicle:

$2\Phi = L_x$	efficiency insensitive to cycle length;
$2\Phi > L_x$	more efficient as cycle length decreases; and
$2\Phi < L_x$	more efficient as cycle length increases.

For the two interior movements:

$$Y_n = [X_n (C - 2\Phi - L_n)] / C$$

where:

- Y_n = Σy_i for the two internal left-turn movements;
- X_n = critical v/c ratio for the internal movements;
- L_n = total lost time for the internal phases, in seconds;
- C = cycle length, in seconds; and
- Φ = overlap or interior travel time, in seconds.

For a given or desired v/c ratio, one may calculate the capacity or flow ratio using the above equations. Likewise, for a given flow rate and cycle length, one may calculate the resulting interior or exterior v/c ratio by rearranging the above equations to solve for X.

The following relationships have been established for the interior phase lengths, Φ, and cycle length:

$$G_1 + G_5 = C - 2\Phi$$

where:

- G_1 and G_5 = interior left-turn phase times, in seconds;
- C = cycle length, in seconds; and
- Φ = overlap or interior travel time, in seconds.

Four exterior movements feed the interchange and the relationship of their phase times to cycle length and interior travel time (Φ) are as follows:

$$G_2 + G_4 + G_6 + G_8 = C + 2\Phi$$

where:

- G_4 and G_8 = phase times for the ramp or frontage roads, in seconds;
- G_2 and G_6 = phase times for the cross-street, in seconds;
- C = cycle length, in seconds; and
- Φ = overlap or interior travel time, in seconds.

Therefore, as the distance between intersections increases, the capacity available for the interior movements decreases, and the green time available for the exterior movements increases. This relationship illustrates the reason that four-phase operation does not work well for long spacing between intersections. One should also note that in order to maximize the operational efficiency of the four-phase strategy, one generally needs slightly longer cycle lengths than those required for three-phase timing strategies.

2.5 Webster's Minimum Delay Cycle

One may consider each side of the interchange an isolated intersection each of which has a minimum delay cycle length. The side of the interchange with the largest minimum delay cycle controls the cycle length for the entire interchange; i.e., the cycle length must be long enough to handle traffic at the higher volume intersection. The equation shown below is used to calculate a minimum delay cycle length.

$$C_o = (1.5L + 5) / (1 - \Sigma Y)$$

where:

- C_o = minimum delay cycle length, in seconds;
- L = total lost time, usually 4 seconds per phase times number of phases, in seconds;
- ΣY = sum of the critical flow ratios, $y_1 + y_2 + \dots\dots\dots + y_i$ (where y_i = volume for critical movement i divided by the saturation flow rate for critical movement i).

Analysts calculate the splits for the three basic phases at each of the intersections using Webster's method as discussed in the previous section.

2.6 Control Strategies

Pretimed Control. One typically uses pretimed control strategies when a limited number of traffic patterns exist and no significant changes in these patterns are expected to occur. One or two controllers (one at each cross-street intersection) may be used to control the interchange. Pretimed control can be used with three-phase or four-phase configurations. One should note, however, that the Texas Department of Transportation (TxDOT) does not purchase pretimed controllers. Rather, they **only** purchase actuated equipment and then use them as pretimed controllers when conditions warrant.

Full-Actuated Control. One may use two standard NEMA full-actuated units to implement three-phase or four-phase configurations. Actuated controllers are typically used for isolated diamond interchanges, where traffic demands and/or traffic patterns vary significantly during the day.

Texas Diamond Controller. This controller configuration is a special full-actuated controller developed by TxDOT to provide phasing that changes with changing traffic demands. A single software-modified eight-phase NEMA controller unit with special internal programming logic is used to provide a combination of either four-phase or three-phase operation at the diamond interchange. The controller makes the change from one type of phasing to the other by time clock or by external traffic-responsive logic (4).

2.7 Measures of Effectiveness

As discussed, signal timing considerations at a diamond interchange include splits, offsets and cycle length. To determine whether a particular set of signal timing parameters provide an acceptable timing plan for implementation, analysts must calculate measures of effectiveness (MOE's). To be useful, these measures should be easily calculated and related to items important to both the driver and the analyst.

Volume-to-Capacity Ratio. According to the *Highway Capacity Manual* (HCM), the v/c ratio equals the actual or projected rate of flow on an approach or designated group of lanes during a peak 15-minute interval divided by the capacity of the approach or designated lanes (5). Capacity at intersections is defined as the maximum rate of flow (for the subject approach), which may pass through the intersection under prevailing traffic, roadway, and signalization conditions. Capacity at signalized intersections is based on saturation flow rates and available green times. Saturation flow is defined as the maximum rate of flow that can pass through a given intersection approach or lane group under prevailing traffic and roadway conditions assuming that the approach or lane group had 100 percent of real time available as effective green time. One can compute volume-to-capacity ratio (v/c) as follows:

$$X_i = v_i / c = v_i / [(g_i/C) * S_i]$$

where:

X_i	=	volume-to-capacity ratio of lane group or approach i;
v_i	=	volume of approach i or lane group, in veh/hr;
g_i	=	effective green time for lane group or approach i, in seconds;
C	=	cycle length, in seconds; and
S_i	=	saturation flow for lane group or approach i, in vphg.

Volume-to-capacity ratios greater than 1.0 indicate over-capacity conditions (i.e., more vehicles than capacity), whereas volume-to-capacity ratios less than 1.0 indicate under-capacity conditions. These conditions should be noticeable from field observations.

Delay. One measure of effectiveness typically used to describe the level of service at signalized intersections is delay. Total system delay and individual delay are a concern for diamond interchanges, with the objective being to minimize both types of delay. Delay at signalized intersections is delay caused by uniform arrival of vehicles, and delay caused by random and overflow arrivals.

The *Highway Capacity Manual* (5) contains the most widely used method to compute stopped delay. Two parts make up the equation: delay due to uniform arrivals and delay due to random and overflow arrivals. Delay for uniform arrivals is based on the assumption that the vehicles arrive at a constant rate and are fully discharged during the cycle. Thus, no vehicles wait for more than one cycle to pass through the intersection. The first part of the equation for stopped delay due to uniform arrivals is as follows:

$$d_1 = \frac{0.38C[1 - (g/C)]^2}{[1 - (g/C)(Min(X,1.0))]}$$

where:

d_1	=	uniform delay in seconds/vehicle;
C	=	cycle length in seconds;
g	=	green time per phase in seconds; and
Min (X,1)	=	the lesser value of either X (v/c ratio for lane group) or 1.0;
X	=	volume to capacity ratio for that phase.

Vehicle arrival patterns, however, are not uniform. They are more likely to be random in nature and may exceed the phase capacity in some instances. The second part of the equation for delay due to random arrivals and queue overflow (incremental delay) is as follows:

$$d_2 = 173X^2 \left[(X - 1) + \sqrt{[(X - 1)^2 + mX/c]}\ \right]$$

where:

d_2	=	incremental delay in seconds/vehicle;
X	=	volume to capacity ratio for that phase;
m	=	a calibration term representing the effect of arrival type and degree of platooning; and
c	=	capacity of lane group, in vehicles/hour.

Intersection stopped delay is computed as follows:

$$d = d_1 * DF + d_2$$

Total delay can be related to stopped delay as follows (6):

$$D = 1.3 * d$$

where:

D	=	total delay, in seconds/vehicle;
d	=	average stopped delay, in seconds/vehicle; and
DF	=	delay adjustment factor for either quality of progression or control type.

Total delay includes the time stopped plus the time lost slowing to a stop and accelerating back to the desired travel speed. Thus, the average total delay is always greater than the average stopped delay. One calculates total delay using the same equations used to calculate stopped delay except that one multiplies the constants 0.38 in the uniform delay equation and 173 in the incremental delay equation by 1.3. This change results in a constant for the uniform delay equation and the incremental delay equation of 0.5 and 225, respectively. Table 2-2 shows the relationship between different levels of service and average stopped and average total delay.

Table 2-2 Level of Service Criteria for Delay

Level of Service	Average Stopped Delay (sec/veh)		Average Total Delay (sec/veh)
A	< 5.0	x 1.3	< 6.5
B	5.1 to 15.0	x 1.3	6.6 to 19.5
C	15.1 to 25.0	x 1.3	19.6 to 32.5
D	25.1 to 40.0	x 1.3	32.6 to 52.0
E	40.1 to 60.0	x 1.3	52.1 to 78.0
F	> 60.0	x 1.3	> 78.0

For a diamond interchange, one may estimate delay for the exterior movements by using the HCM equation. Interior delay is calculated based on a deterministic delay-offset technique. Interior delay varies as the offset between the two intersections varies; with geometric, volume and signalization inputs remaining constant. The total interchange delay at the diamond interchange equals the sum of the exterior and interior movement delays; i.e., seconds per vehicle and multiplied by vehicles per hour and this product is divided by 3600 to convert to vehicle hours of delay per hour of operation.

When evaluating the overall operation of the diamond interchange, the overall interchange delay, as well as the average delay and v/c ratios of the individual movements, should be considered. "Acceptable" levels of service may vary depending on the importance of the movement. For example, one may be willing to tolerate poorer levels of service for low volume turning movements to provide better levels of service for high volume through movements.

Queue Lengths. The two parts of the diamond interchange where queuing is a major concern are on the frontage road and the interior lanes of the interchange. If the distance from the intersection to the freeway exit ramp is short, vehicles may back up into the exit ramp and onto the freeway. If the width of the intersection is insufficient for storing left-turning vehicles, adjacent through lanes or the frontage road approaches may be blocked.

One should note that if queue spillback or queue blockages happen, reduction in the capacity of other phases will occur. Queue spillback creates a serious problem and must be eliminated if the interchange is to operate at an acceptable level. Diamond interchanges are particularly susceptible to queue spillback during periods of oversaturation. During these time periods, one should select special phasing strategies that meter the amount of traffic entering the interchange.

Storage Ratio. This particular measure of effectiveness is a method of quantifying queuing potential in the interior of the interchange and is calculated by the expression shown below:

$$S_r = Q_{max} / S_{cap}$$

where:
- S_r = storage ratio;
- S_{cap} = the available storage capacity between interchange intersections and includes all lanes available for queuing, in vehicles; and
- Q_{max} = the maximum number of queued vehicles on an approach for an average cycle, in vehicles.

A large storage ratio (greater than 0.8) indicates a potential queuing problem in the interior of the interchange. This problem generally occurs at diamond interchanges with closely spaced intersections and heavy turning volumes. Table 2-3 shows the level of service suggested for storage ratios based on the previous equation.

Table 2-3 Level of Service Criteria for Storage Ratio

Level of Service	A	B	C	D	E	F
Storage Ratio	<0.05	<0.10	<0.30	<0.50	<0.80	>0.80

3.0 DATA REQUIREMENTS

To analyze an existing signal timing plan or develop an optimal signal timing plan for implementation, one needs complete and accurate input data. Without accurate field data, a less than acceptable signal timing plan will result, and existing conditions will not be simulated accurately. Knowing what data is needed before going to the field will save the analyst time and extra trips to the project site.

The following sections discuss guidelines and suggestions for complete and accurate data collection needed for retiming signalized diamond interchanges. Analysts use these data in the development of timing plans for both the pretimed and traffic actuated environments. Section 4.0, "Evaluation," Section 5.0, "Optimization," and Section 6.0, "Implementation" describe the recommended use of this data.

The first question one must ask is how many timing plans need retiming? Timing plans may differ for the a.m., p.m., and off-peak periods due to the change in traffic patterns. The number of timing plans may be limited due to the type of controller equipment or signal hardware. One should collect data during the periods of interest. For example, analyst should collect data for analyzing the a.m. peak timing plan during the a.m. peak period, for the off-peak timing plan during the off-peak time period, etc.

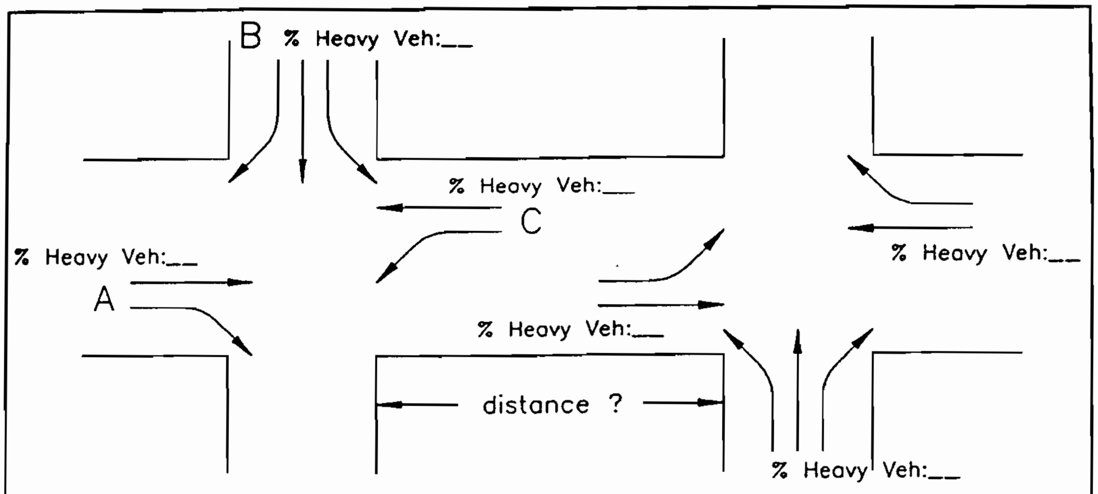
Three types of data should be collected:

1. Traffic Data;
2. Signalization Data; and
3. Geometric Data.

A worksheet showing a sketch of the diamond interchange is useful for recording and organizing data. Figure 3-1 shows an example data collection sheet for recording some of this information.

3.1 Traffic Data

24-Hour Volumes. Volumes are needed for the peak period of interest (a.m., p.m., or off-peak). As a first step in this process, one should make a 24-hour count to determine the peak hour (or 15 minute period). One may make such a count by placing tube counters on the frontage road and cross-street approaches or by dumping detector counts from the controller. Appendix A shows an example of a 24-hour count and a method of determining the peak period from the count.



LEFT SIDE					RIGHT SIDE				
		Green	Yellow	All Red			Green	Yellow	All Red
Phase Times	A				Phase Times	A			
	B					B			
	C					C			

Type of Phasing:__________ Left Turn Sequence:________

Figure 3-1. Example of Data Collection Sheet

Turning Movement Volumes. After determining the peak period, manual counts are necessary to record volumes for the turning movements and the interior movements in the interchange. Eighteen movements should be counted at a diamond interchange as shown in Figure 3-2. Note that the origin and destination of these movements define them.

For example, straight through movements on the exterior approaches to the interchange are subdivided into those that travel straight through both intersections and those that turn left at the downstream intersection. Generally, one should make turning movement counts in 15 minute intervals during the two hour a.m. or p.m. peak periods and for one hour during the off peak period. One adds the highest 4 consecutive 15 minute volumes to determine the highest peak or off-peak hour traffic volumes. These volume levels represent actual demand, and one should use them to compute benefits. The peak 15 minute volumes represent critical demand during the peak hour, and one should multiply them by four and use the critical demand when computing signal timings.

Figure 3-3 shows a turning movement data sheet with the various movements illustrated and time divided into 15 minute intervals. When counting turning volumes, the interior movements are shown separately from the other turning movements, even though they have been previously counted on the frontage road or cross-street because of signal timing considerations. As mentioned previously, one should make these counts in 15 minute intervals within the peak period.

During congested periods, one should count the demand volume rather than discharge volume; i.e., the measured discharge volume will be less than the true demand volume if the queue fails to clear during the green indication. If this situation occurs, the actual volume counted should be those vehicles arriving at the back of the queue rather than those vehicles that depart when the signal is green. One should note, however, that this procedure is for counting and not always a recommended signal timing strategy; i.e., trying to clear the queue each and every cycle results in extremely long cycle lengths during congested conditions.

Peak hour factor (PHF). After making these counts, adjustments may be necessary to account for the peak period. According to the HCM, peak rates of flow relate to hourly volumes through the use of peak hour factors. The peak hour factor is defined as the ratio of total hourly volume to the maximum 15 minute rate of flow within the hour. If 15 minute counts are used, then:

$$PHF = \frac{V}{(V_{15} * 4)}$$

where: PHF = peak hour factor;
V = highest hourly volume, in veh/hr; and
V_{15} = highest 15-minute count within that hour in veh/15 min.

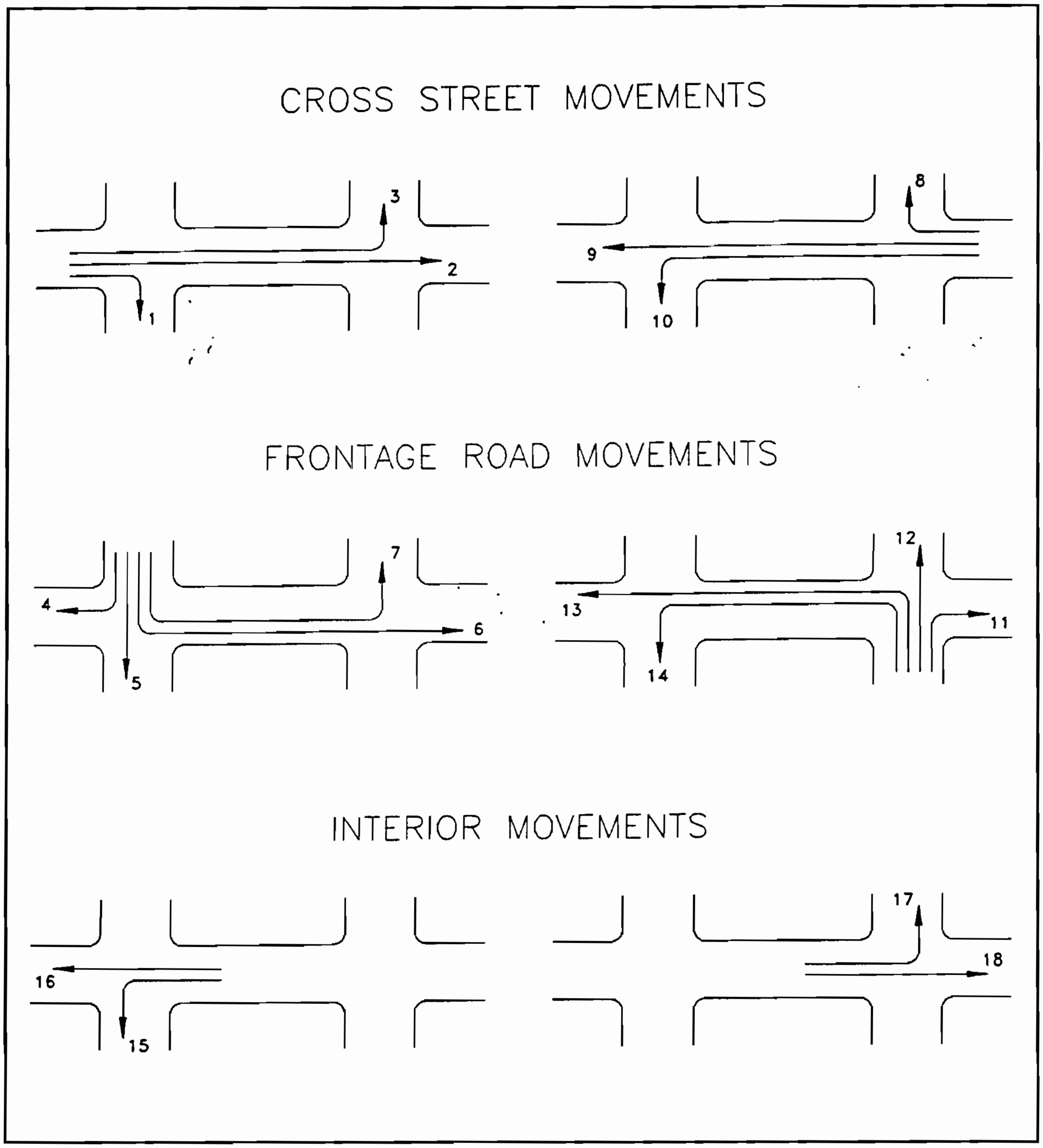


Figure 3-2. Turning Movements Needed for Analysis

Date: / / Counter:______ TIME	Arterial			Frontage				Interior		Arterial			Frontage				Interior	
	Rt 1	St 2	Lt 3	Rt 4	St 5	Lt 6	U 7	Lt (12+16) 8	St (11+15) 9	Rt 10	St 11	Lt 12	Rt 13	St 14	Lt 15	U 16	Lt (3+7) 17	St (2+6) 18
(15 min Interval)	Volumes			Volumes				Volumes		Volumes			Volumes				Volumes	
TOTAL																		

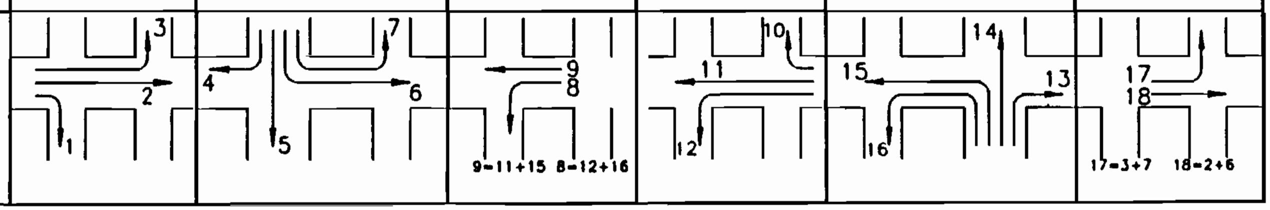


Figure 3-3. Turning Movement Data Sheet

Analysts generally report demand volumes in terms of vehicles per hour for a peak hour. For analysis, one normally adjusts peak hour volumes to flow rates in vehicles per hour for a 15 minute period. For example, after making a 24-hour count, analysts determined the a.m. peak hour to exist from 7:00 to 8:00 with a total volume during this hour of 900 vehicles. The peak 15 minute count within that hour equalled 300 vehicles between 7:30 and 7:45. Thus, one can calculate the resulting peak hour factor as follows:

$$PHF = \frac{900}{300 * 4} = 0.75$$

For timing purposes, one can calculate the peak hour flow rate for timing purposes as either the hourly volume divided by the peak hour factor (900/0.75), or the peak 15 minute volume multiplied by 4 (300 x 4). In either case, the calculated peak hour flow rate equals 1200 vehicles per hour. One should note that, if the peak 15-minute flow rate was multiplied by four to arrive at a peak hour flow rate, the correct peak hour factor equals 1.0; i.e., the adjustment for peak flows within the hour have already been accounted for.

Saturation Flow Rate. The saturation flow rate equals the maximum flow rate at which vehicles pass through the intersection. One expresses this rate as in vehicles per hour of green per lane during an hour with continuous demand and subject to prevailing roadway conditions. For example, adjustment factors for roadway and traffic conditions, such as lane width and truck percentages, are used to reduce the ideal saturation flow rate to an adjusted rate appropriate for the location. The following data items are used for calculating restrictions of flow to adjust the saturation flow rate using the HCM method.

Percent Heavy Vehicles - The analyst should count the number of heavy vehicles within the total volume of traffic as a percentage of the total traffic. A heavy vehicle is characterized as having at least six wheels in contact with the roadway. One may classify heavy vehicles as three types: trucks, recreational vehicles, and buses. This heavy vehicle count is necessary to account for the additional space occupied by the larger vehicles. Heavy vehicles also operate differently than passenger vehicles, which contributes to a decrease in the saturation flow rate and capacity; i.e., heavy vehicles accelerate from a stop at a slower rate and physically occupy more space than passenger. Thus, heavy vehicles occupy more time and space than do passenger cars and reduce the saturation flow rate and capacity accordingly.

Pedestrians - The number and type of pedestrians crossing the intersection should be noted. Children and the elderly may need special consideration for crossing time. This requirement proves true for interchanges both with and without pedestrian push buttons or signals. Right-turn conflicts with pedestrians also should be noted. If the number of conflicts is high, the right turn saturation flow rate may need reducing. One can record pedestrian volumes as the actual number counted or as a general range (less than 50, 50 to 200, or greater than 200). Either option is acceptable; however, it is important that one base the data on field observations.

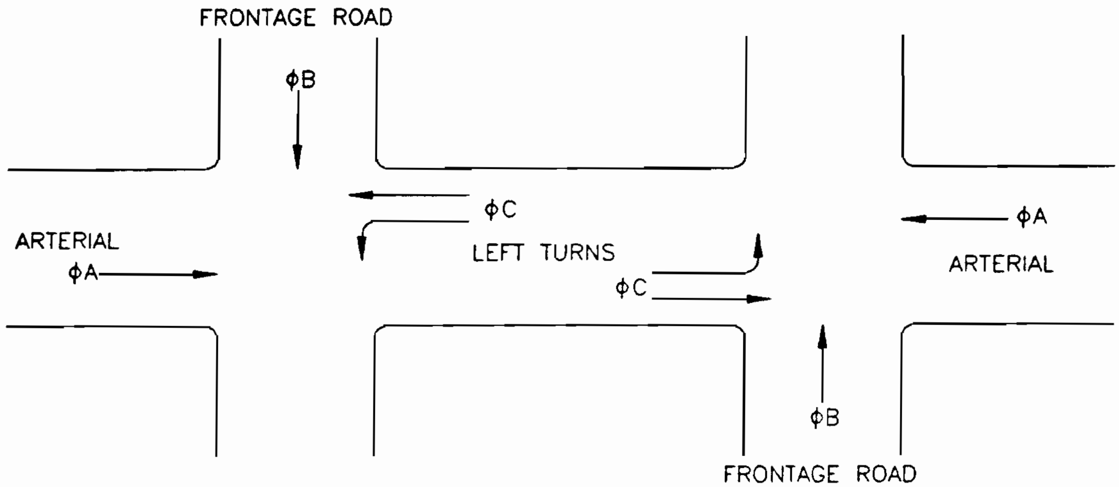
Saturation flow rate is extremely important to determine the capacity and splits for specific movements. For example, if one overestimates a particular movement's saturation flow rate, less green time than needed will be allocated to that movement, and if one underestimates a particular movement's saturation flow rate, more green time than needed will be allocated to that movement. Neither condition is desirable.

3.2 Signal Data

Cycle Length. The analyst should record the cycle length for the timing plan being analyzed. For pretimed control, the cycle length will remain constant. One may obtain the cycle length from old timing plans, the controller, or by measuring the length of the cycle with a stop watch. Signals controlled by actuated controllers will have varied cycle lengths. One should measure an average cycle length by averaging between 10 and 30 stop-watch measurements.

Phase Sequence. The analyst should record the existing phasing at the diamond interchange. Diamond interchanges typically operate as three-phase or four-phase with overlap. One may further classify these phase types by the order in which the left-turn movements proceed in relation to the cross-street phase. Figure 3-4 shows the different phase sequence combinations possible. Four-phase with overlap, also known as TTI-lead, follows the sequence of frontage road, overlap phase, cross-street, frontage road, overlap phase, and cross-street.

The analyst may obtain the phase sequence information for existing conditions from timing plans or field observation. Generally, if the two ramp or frontage road phases start at the same time and/or the two arterial or cross street phases start at the same time, the interchange operates in a three-phase mode, and if the start of the two ramp or frontage road phases are offset by about one-half of the cycle length, the interchange operates in the four-phase with overlaps mode.



PHASING CODE	LEFT SIDE PHASE SEQUENCE			LEFT TURN SEQUENCE	RIGHT SIDE PHASE SEQUENCE		
1	A	B	C	LEAD-LEAD	A	B	C
2	A	C	B	LAG-LEAD	A	B	C
3	A	B	C	LEAD-LAG	A	C	B
4	A	C	B	LAG-LAG	A	C	B
1A	A	B	C	TTI-LEAD	A	B	C

Figure 3-4. Possible Phase Sequences

Left-Turn Treatment. The interior phases (Phase C or Phase 1 and 5) may include protected, protected plus permitted, or permitted left-turn movements. One can identify these alternatives in the field by observing the following characteristics:

Protected -	Left-turn movements are protected with a green arrow. Left-turning vehicles are not allowed to proceed otherwise.
Protected/Permitted -	Left-turn movements are protected with a green arrow and may also proceed during the green ball indication after yielding to opposing traffic.
Permitted -	Left-turn movements proceed during green ball indication after yielding to opposing traffic.

Green Splits. The analyst needs the existing green time plus yellow plus any red clearance time for each phase. For pretimed controllers, the green splits will remain constant and one can obtain them from the signal timing plans or measure them with a stop watch. If the signal is actuated, the green time will vary, and one should determine an average green time for each phase. It is recommended that analysts make field measurements using a stop-watch for between 10 and 30 consecutive cycles and that the average phase lengths and cycle lengths be used as input.

Type of Controller. As mentioned in the previous chapter, diamond interchanges may operate in either a pretimed or actuated mode. The Texas Diamond controller is a special type of actuated controller that allows one to use three-phase or four-phase operation depending on the traffic demand. One should record the type and/or number of controllers as the hardware's capabilities may place limitations on the signal timing plans that can be implemented. This limitation is becoming less and less of a concern with the newer traffic signal control equipment that is available. It should be noted that this limitation is not a recommendation to change traffic signal equipment, but rather to identify the available options prior to signal timing plan development.

3.3 Geometric Data

Number of Lanes and Lane Movements. The analyst should record the number of lanes on the ramp or frontage road, arterial or cross-street, as well as the interior of the interchange. The number of lanes is counted at the stop bar, not downstream of the intersection. One should also note the types of movements allowed for each lane, including exclusive turning lane or shared lanes. Some diamond interchanges have U-turn lanes, and they should be noted also.

On the right side of the interchange, 411 vehicles are turning left and 568 vehicles are going through the interior. The total number of vehicles equals 979. Forty-two percent of vehicles are turning left and 58 percent are going through. Again, the interior has storage for 27 vehicles total. Using proportions:

(27 veh. total) * (0.42 left turns) = 11 vehicles;
(27 veh. total) * (0.58 throughs) = 16 vehicles;

Because left turns form more than a third of the total volume, left turns share part of the optional through plus left lane. The remaining storage of 16 vehicles in the optional and through only lanes is for the through movements.

Signal Phasing Data. The existing phase sequence is three-phase, lead-lead (ABC/ABC). Analysts determined the internal offset to equal 5 seconds; i.e., the time from when the arterial phase A (left side) starts until the end of the frontage road phase B (right side). To simulate existing conditions, one answers 'N' for the delay-offset optimization column for all phase sequences, and enters the internal offset for the existing phasing pattern only. Figure 4-9 shows calculations for determining the internal offset.

Interchange Movement Screen. The design hourly volumes (peak 15 minute counts times 4) were entered for each movement using the engineer's assistant key <F3>. Analysts also calculated the saturation flow rates by entering the appropriate lane assignment information. No U-turns existed during the peak period, and there were five percent heavy vehicles.

For evaluating existing conditions, analysts entered the existing phase times (green plus yellow plus red clearance) for the arterial phase, the frontage road phase, and the interior left-turn phase for both sides of the interchange. The phase times on each side of the interchange must add up to the existing cycle length (in this case, 90 seconds).

Running the Program. After entering the data, the user returns to the *Main Menu* and selects the *Run* command. Analysts noted no apparent coding errors, and the input data was carefully checked to ensure that it was coded correctly. The program produced the following output for the evaluation of existing conditions at the example urban diamond interchange.

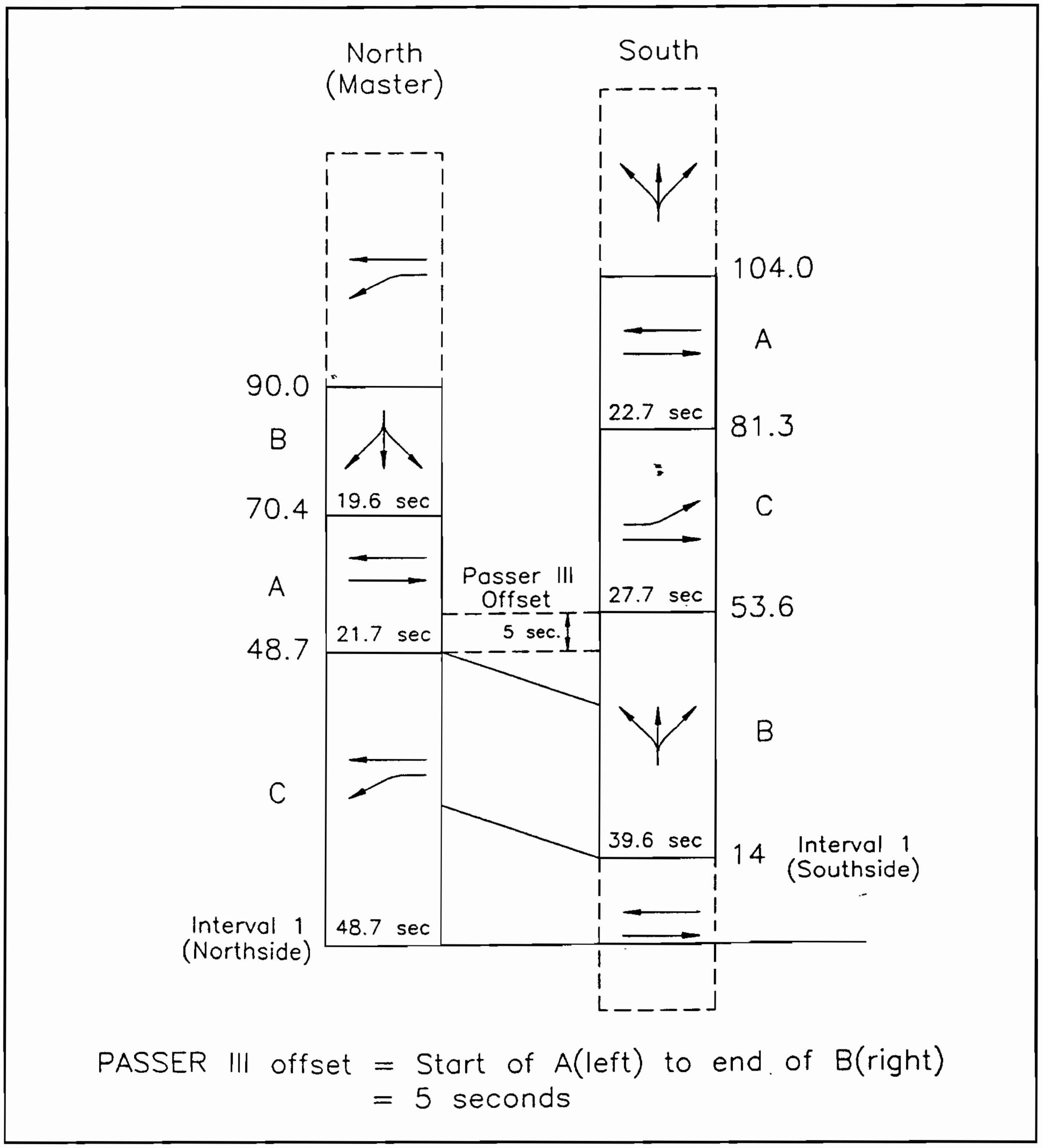


Figure 4-9. Calculation for PASSER III Internal Offset - Example

Interpreting the Output. The program printed the *General Signalization Information* and the *Signal Phasing Information* for the existing conditions, as shown in Figure 4-10. The top part of the figure (*General Signalization Information*) indicates the level-of-service for delay, v/c ratio, and the queue storage provided by the existing signal timing plan. The volumes exceed capacity for the arterial on the left side of the interchange and a v/c ratio of 0.97 is reported for the frontage road on the right side of the interchange. The program reports level-of-service F and E for delay for the arterial on the left side, as well as level-of-service E for the frontage road on the right side. The storage ratio for the left and right side of the interchange, however, is good. These findings remain consistent with field observations of the interchange's operation during the a.m. peak hour.

Possible reasons for these problems include that the interchange is relatively narrow (243 feet between signals), but was operated with a three-phase, lead-lead sequence; and the green splits appear disproportionate to the traffic volumes. The latter problem is illustrated by the fact that the interior movements each have a very good level-of-service, whereas the exterior movements have much worse levels of service.

The next section of these guidelines addresses possible optimization strategies to improve this signal timing plan.

```
<GSI01>
* * * INTERCHANGE  1 BINGLE                                          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)  22.0   20.0    48.0    70.0    *  23.0    39.0    28.0   51.0
                                                 *
V/C RATIO          1.14    .75     .29     .19   *   .76     .97     .69    .35
 LEVEL OF SERVICE  F       C       A       A     *   C       E       B      A
                                                 *
DELAY (SEC/VEH) 156.91  59.18     .36     .05    *  43.12 166.94   3.91    .00
 LEVEL OF SERVICE  F       E       A       A     *   D       F       A      A
                                                 *
STORAGE RATIO                     .04     .03    *                  .04    .00
 LEVEL OF SERVICE                 A       A      *                  A      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
  PHASE ORDER   LEAD-LEAD      TOTAL INTERCHANGE DELAY    64.12 VEH-HRS/HR
  INTERNAL OFFSET   5 SEC      CYCLE LENGTH   90 SEC
```

```
<SPI01>
* * * INTERCHANGE  1 BINGLE                                          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            *       5.00
      2         *         A            *         C            *      17.00
      3         *         B            *         C            *      11.00
      4         *         B            *         A            *       9.00
      5         *         C            *         A            *      14.00
      6         *         C            *         B            *      34.00
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
  INTERNAL OFFSET     5 SEC                          CYCLE LENGTH    90 SEC
                                                     PHASE ORDER LEAD-LEAD
```

Figure 4-10. Printout of Existing Conditions.

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5.0 OPTIMIZATION

The next step in the retiming process is to try to optimize the existing signal timing plan thus improving diamond interchange operation. As discussed earlier, traffic engineers use measures of effectiveness, such as delay and volume to capacity ratios, to monitor the efficiency and operation of the diamond interchange. When traffic volumes increase or operating conditions degrade, one must increase the capacity of the interchange. Methods used to increase the interchange capacity include modifications to the cycle length, green splits, phasing and/or interchange geometry.

This report will discuss the use of PASSER III for optimizing signalized diamond interchange operations, along with optimization strategies. PASSER III users should be aware that some control strategies produced by the program may not work with the existing equipment in the field; i.e., a limit to the number of offsets implementable may exist for diamond interchanges controlled with one controller. With the new generation of Texas Diamond Controller, however, analysts can program offsets and/or overlaps using several different diamond configuration schemes adding a great deal of flexibility to a single controller.

The following sections discuss guidelines for optimizing signal timing using PASSER III. In most cases, the user will have previously entered existing volumes and saturation flow rates for the interchange. Before optimizing, the user should have checked these data for accuracy and calibrated them for local conditions. The other data will be edited depending on the type of optimization needed.

5.1 Cycle Length Optimization

The cycle length at which the signals operate must measure long enough to provide acceptable volume to capacity ratios while also minimizing overall interchange delay. One may determine the minimum delay cycle length by analyzing a range of cycle lengths using PASSER III and by manual inspection, selecting the optimal cycle length. The minimum green times entered by the user constrain the minimum allowable cycle length. The sum of the minimum green times must be less than the lower cycle of the range to be analyzed.

To optimize the cycle length, the user selects the *EDIT* command from the *Main Menu* and selects the *General Freeway Identification Screen*. PASSER III can be used to select the optimum cycle length for a wide range of cycle lengths.

1. **Lower Cycle** - The user should set the lower cycle length equal to the smallest permissible cycle length based on the sum of the minimum conflicting greens as determined using the Poisson or Webster technique. Each side of the

intersection may require a different cycle length. One should use the larger of the two cycle lengths as the lower limit.

2. **Upper Cycle** - The lower cycle length will generally constrain the upper cycle length. Because a 15 to 20 second range is generally long enough to find a suitable cycle length, the upper cycle limit should not lie greater than 20 seconds longer than the lower cycle limit; however PASSER III will run even if the range is greater than 20 seconds. The maximum allowable value for the upper cycle length is 150 seconds for an optimization run.

3. **Increment** - An increment of 5 seconds is recommended; however, one may use other increments.

5.2 Phase Split Optimization

PASSER III allocates green times to movements based on an equal-degree-of saturation approach. That is, the program allocates green times in proportion to the percentage of the intersection's total critical lane volume served by the phase. Although this method does not guarantee minimal delay, it does provide adequate green time for the critical movements. The critical movements are those with the highest volume to saturation flow ratio per phase. Extremes in volume to capacity ratios for existing conditions (i.e., some movements with high v/c ratios and others with low v/c ratios) may indicate poor green (phase) split allocation. In this case, phase split optimization may improve the operating conditions at the interchange.

To optimize the phase splits, the user accesses the *Movement Interchange Data Screen* and edits the minimum phase times used as inputs when evaluating existing conditions. The minimum green times that should be entered for an optimization run are based on pedestrians considerations or driver expectancy. Normally, a minimum phase time should not be less than 10 seconds; however, if pedestrians are present, minimums may be longer than this value. The sum of the minimum green times should be less than or equal the lower cycle length.

5.3 Phasing Optimization

The phasing used at a diamond interchange will depend on the width of the interchange and the level and distribution of traffic volumes. PASSER III can analyze the phase sequence combinations shown in Figure 5-1. To optimize the phase sequence using PASSER III, the user accesses the *Signal Phasing Data Screen* and enters 'Y' beside the phase sequences to be considered. This data entry will also cause the optimization of the internal offset for that sequence and specified cycle length. Normally, the optimum cycle length is used during optimization of the phase sequence; i.e., cycle lengths and phase sequence/offsets are generally not optimized in a single run of the program. The sum of minimum green times should be less than the specified cycle length.

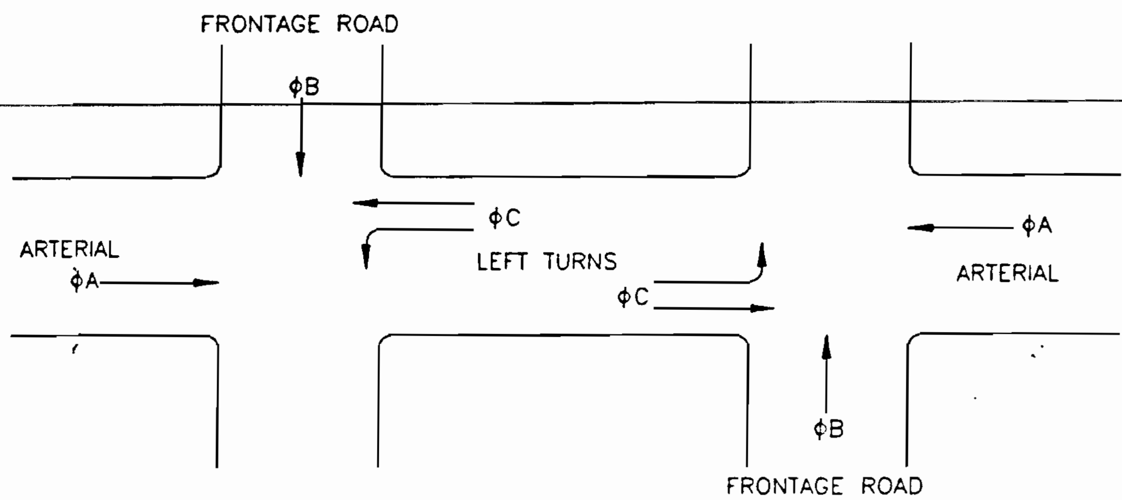
Generally four-phase with overlap (TTI-Lead) works best for closely spaced intersections where heavy interior movements cause storage problems. Three-phase control generally works best for widely spaced intersections with light turning movements, and heavy through movements either on the frontage road or the arterial. At intermediate spacing, the type of phasing that works best depends on traffic volume levels and the distribution of turning movements.

The optimum sequence of the interior left-turn phase will depend on whether the predominant interior movements originate from the frontage road or the cross street. For example, if the left side of the interchange has heavy left turns from the frontage road, a lead-lag phasing would be preferable to accommodate the heavy turning movement from the frontage road. One would probably use a lead-lead or lag-lag phase sequence for heavy interior turns originating from the arterial.

5.4 Internal Offset Optimization

The progression of movements on the interior approaches of the diamond interchange are essential to minimizing vehicular delays. The internal offset may be evaluated and optimized by PASSER III based on the phase configuration, volumes and cycle length. The internal offset is defined as the time from the beginning of the arterial or cross-street phase on the left side of the interchange (Phase A or Phase 2) to the end of the frontage road phase on the right side of the interchange (Phase B or Phase 8). The offset which produces minimum delay and adequate interior storage ratios is desirable.

To optimize the internal offset, the user accesses the *Signal Phasing Data Screen* and enters a 'Y' beside the phase sequence of interest. One may run a delay-offset optimization for one cycle length or a range of cycle lengths in combination with one or more phase sequences. PASSER III will report the internal offset for the cycle length and allowable phase sequence which produces the least system delay as the best solution.



PHASING CODE	LEFT SIDE PHASE SEQUENCE			LEFT TURN SEQUENCE	RIGHT SIDE PHASE SEQUENCE		
1	A	B	C	LEAD–LEAD	A	B	C
2	A	C	B	LAG–LEAD	A	B	C
3	A	B	C	LEAD–LAG	A	C	B
4	A	C	B	LAG–LAG	A	C	B
1A	A	B	C	TTI–LEAD	A	B	C

Figure 5-1. PASSER III Phasing Codes

One should note, however, that generally a large number of other solutions exist which are almost optimal; i.e., numerically, one solution is the best, but practically, several different solutions can produce acceptable operations. To optimize the internal offset without optimizing the phasing sequence, the user enters a 'Y' beside the existing phase sequence, and then enters the existing cycle length and green splits in the required screen.

5.5 Other Improvement Strategies

PASSER III has the capability of evaluating several left-turn phasing alternatives, and the analyst may consider geometric conditions to optimize or improve signal timing.

Left-Turn Protection. The interior left-turn movements may be either protected only, protected plus permitted, or permitted. Depending on the opposing traffic volumes, permissive left-turn movements may significantly increase the capacity of the interchange. Permissive left-turn movements will improve operations significantly only if adequate gaps in the opposing traffic stream exist.

Protected-plus-Permitted - To simulate the effects of allowing permitted-plus-protected left-turn movements, the user accesses the *Signal Phasing Data Screen* and enters 'Y' for permitted turns. Because protected left-turn movements are PASSER III's default condition, allowing permitted left turns is in addition to the protected phase that already exists.

Permitted Only (2-Phase) - To model permissive left turns only or no Phase C, the user sets the minimum phase times for the arterial phase and frontage road phase so that their sum equals the desired cycle length. The upper and lower cycle length must equal the desired cycle length. The user should set the minimum phase times for the interior left-turn movements to 0. The user also must enter 'Y' for permissive left turns on the *Signal Phasing Data Screen*.

U-turn Lanes. If heavy left turns from one frontage road to the other frontage road exist, U-turn lanes may be a feasible solution for reducing the numbered left-turning vehicles in the interior of the interchange. The number of turning vehicle removed and the magnitude of delay reduction as a result of the U-turn lanes will determine the cost effectiveness of their construction. Note that this improvement is often justified at moderate volume levels (more than 150 to 200 U-turns per hour) (7).

To simulate the addition of adding a U-turn lane, the user accesses the *Movement Interchange Data Screen* and enters 0 for the left-then-left-turn volumes. It is recommended that one change the volumes using the assistant key <F3> so that the saturation flow rates are adjusted as well. The user should also edit the allowable movements in the lane that previously accommodated the U-turn movement.

When collecting geometric data, one may observe lanes which share movements (throughs and lefts) for determining the percentages of left turns and through movements utilizing the shared lane. These percentages will prove helpful when calculating the amount of vehicle storage available in the interior of the interchange.

Lane Widths. The lane width for each lane of the cross-street, frontage road and the interior portion of the interchange is required. One may make measurements directly in the field or obtain them from existing plan sheets. Lane widths affect the saturation flow rate on the approach. Lanes less than 12 feet will begin to reduce capacity.

Percent Grade. The analyst should record the percent grade at each approach. This information should be obtainable from existing plans or be measured in the field. Percent grade will also affect the saturation flow rates and possibly the lost-time due to longer start-up times. Grades may also affect the travel times between intersections. Generally, upgrades decrease saturation flow rates and increase travel time, and downgrades increase saturation flow rates and decrease travel time.

Interchange Width. Analyst should record the distance between the right side and the left side of the interchange. This information is necessary for calculating the interior storage capacity and travel time for overlap phasing. One measures the width for travel time determination from stop-line to stop-line. The width for queue storage capacity is measured from stop-line to the most distant point that vehicles can stop and not block the upstream intersection. This distance will always be less than from stop-line to stop-line.

Travel Time. Interior travel time is a function of the geometrics of the interchange. One can estimate travel time from the left (right) to right (left) sides by interchange widths as discussed earlier in Section 4.2, or it be measured in the field using a stopwatch. The travel time is the time for a vehicle to get from the exterior stop line of the left (right) side intersection to the interior stop line of the right (left) side intersection. Travel times can differ in the two directions if a grade exists between the two intersections or if one of the intersections is wider than the other one. It is important that these times be as accurate as possible, as they are used to determine when the downstream signal should change to green.

4.0 EVALUATION

After collecting the data, the next step toward developing a new signal timing plan for a diamond interchange is evaluating the existing conditions. Evaluation of an existing control strategy requires field observation as well as analysis. A recommended methodology for assessing the operational efficiency of a control strategy is summarized below (7).

Field Evaluation

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

2. 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.

3. Check that the queue of vehicles at the frontage road signal phases does not back into adjacent signalized intersections on the arterial. If so, the control strategy is probably not acceptable.

Analysis

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

2. Check that the overall level of service at the interchange falls within acceptable limits. If not, cycle length, phasing sequence, mode of operation and/or geometric modifications may be appropriate.

4.1 Evaluation Software

Analysts may perform evaluation or simulation of a signalized diamond interchange's operation using computer programs, such as the Highway Capacity Manual Software (8) or PASSER III-90 (9). Analysts can use the measures of effectiveness calculated by these programs to locate operational problems within the interchange and pinpoint areas that need improvements.

Highway Capacity Manual Software (HCS). This program uses the Highway Capacity Manual method for calculating saturation flow rates, and intersection capacity and delay. The HCM procedure forms the basis for most signal analysis methods. The program requires some manipulation to simulate a diamond interchange in that one must estimate progression adjustment factors for the coordinated movements. One weakness of the HCS is that the program can only evaluate signal timing plans; it cannot optimize cycle lengths, green splits, or phase sequences. For further information, consult the *Highway Capacity Software User's Manual* (8).

PASSER III-90. The Progressive Analysis and Signal System Evaluation Routine, (PASSER) III is a fixed-time based optimization model. The Texas Transportation Institute developed the program for TxDOT to determine and evaluate the optimal signal timing plan at diamond interchanges. PASSER III analyzes isolated diamond interchanges (with or without frontage roads) and/or progression for a series of diamond interchanges connected by frontage roads. The program analyzes different phasing patterns and varies the offset between the two signals to minimize delay within the interchange.

The optimal cycle length in the isolated mode is the cycle length that provides adequate queue storage and minimizes delay progression mode to maximize the frontage road bandwidth. TTI designed PASSER III to analyze fixed time and fixed sequence control, but the program has provisions for analyzing actuated control using the built in delay-offset analysis. Input requirements include turning movements, distance between intersections, average link speeds, queue clearance interval, phasing sequence, and minimum green times. PASSER III-90 has a built in assistant function to calculate saturation flow rates based on the Highway Capacity Manual methodology. One may obtain further information for running PASSER III from the PASSER III-90 User's Manual (9).

Other Programs. One should note that other programs can be used to analyze diamond interchanges. These programs require additional data and computer requirements, and thus, prove difficult to use; however, they are useful for analyzing complex geometry and oversaturated conditions and the following paragraphs briefly describe them.

The Transportation Road Research Laboratory (TRRL) developed TRANSYT-7F in Great Britain to optimize traffic signal settings for an arterial street. The University of Florida modified the program for the FHWA to reflect U.S. conditions and terminology. TRANSYT-7F searches for signal timings (splits, cycle lengths, and offsets) that minimize some combination of stops and delay. Saturation flow rates must be determined external to the program. Measures of effectiveness include v/c ratio, delay, stops, queue lengths, and fuel consumption. One can use TRANSYT-7F to analyze signalized diamond interchanges; however, the data coding scheme is much more complex than PASSER III. For further information, see *An Application Manual for Evaluating Two and Three-Level Diamond Interchange Operations Using TRANSYT-7F* (10).

The University of Texas at Austin developed the TEXAS model for diamond interchanges for the TxDOT. This version (Version 3.0) has the capability of performing detailed computer simulation of diamond interchanges as well as single intersections. A graphics display illustrates the speed, location, and time relationship for every simulated vehicle. This program simulates pretimed, semi-actuated, and fully actuated control, and evaluates emissions of air pollutants from vehicles at the intersection. This model does not optimize signal timings. One enters the data through two computer based data-entry programs called GDVDATA (Geometry, Driver, Vehicle) and SIMDATA (Simulation). Measures of effectiveness obtainable either directly from printout or from post-processor analysis of output data files include delay, queue lengths, probability of clearing a queue in one signal cycle, and travel times. For further information, refer to *TEXAS Model Version 3.0 (Diamond Interchanges)* (1).

TRAF NETSIM is a microscopic simulation model developed by the Federal Highway Administration. This model can simulate traffic control systems in great detail; however, corresponding detail in the input data is required. The TRAF NETSIM model can handle both isolated intersections, coordinated networks, and diamond interchanges. The model can simulate uncontrolled, stop/yield controlled, pretimed and semi-actuated systems. Fully actuated signals can also be simulated in isolated mode. The output includes detailed statistics on delay, stops, queues, emissions, and other measures of effectiveness. For further information, refer to *TRAF User Reference Guide* (11).

4.2 Input Requirements

Both PASSER III and HCS may be used with IBM-PC compatible microcomputers. Both programs give similar results for exterior movements, but sometimes very different results for the coordinated movements (7). These guidelines will address the use of PASSER III-90 for the analysis of signalized diamond interchanges due to the program's capability of both simulating and optimizing signal timing plans.

The following steps compose guidelines for data requirements to simulate (evaluate) existing conditions at a diamond interchange using PASSER III. For the isolated mode, the user codes using three data screens: *Freeway Identification*, *Interchange and Signal Phasing Data*, and *Interchange Movement Data*. To begin entering data, the user must first access the *File Screen*. The *File Screen* allows the user to set up the file name and path for the new data set. After entering the proper path and file name, the user hits <ESC> to the *Main Menu* and then selects the *Edit Command* to enter new data.

Freeway Identification. Most of the input data requirements on this screen are self explanatory. Those items having special requirements for evaluating existing conditions are noted.

1. **Run Number;**
2. **Freeway Name;**
3. **District (TxDOT District Number);**
4. **City Name;**
5. **Number of Interchanges** (For isolated intersections, enter 1); and
6. **Cycle Lengths** (Set the lower and upper cycle lengths equal to the existing cycle length).

The remainder of the screen does not apply to the isolated mode and will be blanked out when the user enters 1 for the Number of Interchanges. Figure 4-1 illustrates a filled-in *Freeway Identification Screen* for evaluating existing conditions.

Interchange and Signal Phasing Data. Instructions for entering data on the second of the three input screens are described below.

1. **Cross Street.** Enter the cross street name.

2. **Permitted Left Turns.** Input the left turn treatment existing at the interchange. If the existing phasing allows any unprotected left turns, enter 'Y'; if left turns are protected only, enter 'N.'

3. **Interior Travel Time.** The interior travel time is the running time from the left (right) side of the intersection to the right (left) side. One may measure the travel time in the field or estimate it based on the width of the interchange. Table 4-1 illustrates the corresponding travel times for various interchange widths.

4. **Interior Queue Storage.** The interior queue storage equals the number of storable vehicles in the interior of the interchange. The user enters storage separately for the through and left movements. An estimate of queue storage can be obtained by assuming a vehicle occupies 25 feet of lane space. One must add multiple lane storage, and remember that a single lane may store both left and through vehicles. The storage for left-turn and through vehicles sharing a lane may be based on proportions of left-turn and through vehicles to total volume or by field observation.

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

                    FREEWAY IDENTIFICATION

        Run Number: 01   Freeway Name: USERS MANUAL EXAMPLE 1
        District  : 15      City Name: SAN ANTONIO
        Number of Interchanges  (1 for isolated analysis) :  1
               --------CYCLE LENGTHS----------
               Lower:100  Upper: 100 Incr:

        Calculate Band Split Proportional to Traffic?(Y/N):
        "A" direction Percentage              ( 0 to 100 ):
        Speed Search?                                (Y/N):


               Time/Space Diagram (Y/N):
        ------------PLOT SCALING / INCH-------------------
        Horizontal (Seconds):       Vertical (Feet):
           Note: Set to 0 for program selected values



             <Esc>exit screen <F10>Cancel <PgUp><PgDn>next screen
```

Figure 4-1. Freeway Identification Screen

Table 4-1 Estimated Travel Times for Various Diamond Interchange Widths (9)

Distance (ft)	Travel Time (sec)	Overlap (sec)
67	6	4
94	7	5
125	8	6
160	9	7
200	10	8
244	11	9
288	12	10
332	13	11
376	14	12
420	15	13

5. **Signal Phasing Data.** For simulating existing conditions, answer 'N' to the optimized delay-offset for all phase sequences, but enter the existing internal offset on the line for the existing phase sequence. Figure 4-2 shows PASSER III phase sequence codes. One may easily calculate the existing internal offset to be coded into PASSER III if the existing phase times are known. The PASSER III definition of internal offset is defined as the time at which Phase A (arterial or cross-street phase) on the left side begins to the time that Phase B (frontage road phase) on the right side ends. Figure 4-3 shows examples of two different phase sequences and calculations for determining the internal offset for input to PASSER III. Figure 4-4 illustrates the signal data screen and hints for simulating existing conditions.

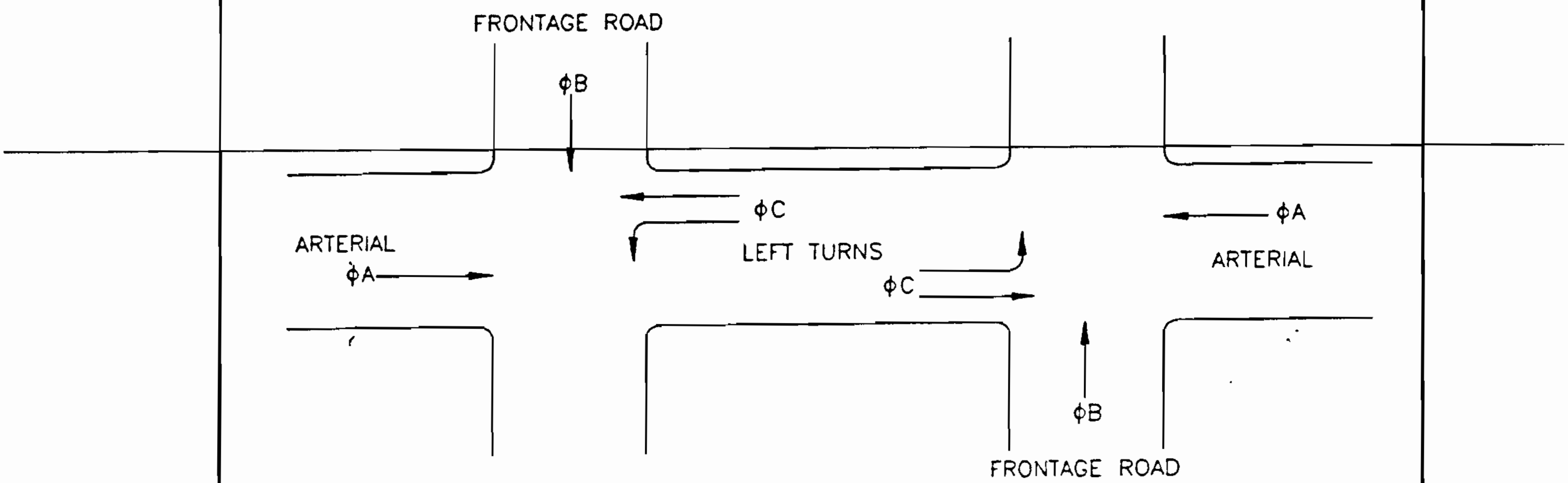
Interchange Movement Screen. To enter volumes, number of lanes, and minimum phase times, one should use the assistant function <F3>. When the data entry in the assistant window is complete, the saturation flow rates for each movement are automatically calculated when the user presses the <F3> key or the <ESC> key. Figure 4-5 shows the Interchange Movement Screen with and without the Assistant Window.

1. **Volumes.** The user enters the volume for each movement for both sides of the intersection. Arrows illustrate the movements corresponding to the cross street, frontage road and interior on the screen along with the description of the movement. If U-turn or free right turn lanes are present, one enters the volumes for left-then-left (U-turns) and right turns as 0.

2. **Number of Lanes and Lane Assignments.** The user must enter the number of lanes for each approach. PASSER III automatically shows the default lane assignments made based on movement volumes entered. The user enters the allowable movements to be made from the lanes shown in the lower portion of the assistant screen.

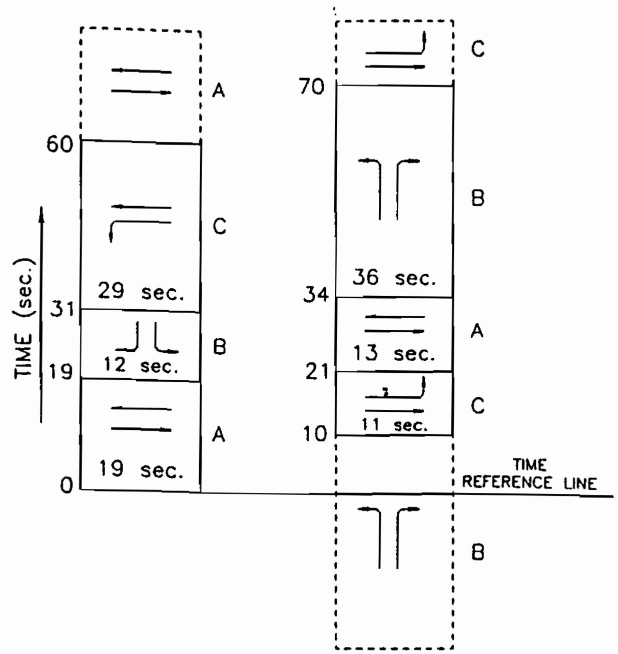
   The user must assign an R for rights, L for left turns, T for throughs and/or a U for U-turns to at least one lane if non-zero volumes were entered for the movements. No letters should be entered for zero volume movements. That is, at least a partial lane for each movement must exist, and **no** allowable lanes for movements that do not exist are possible. It should be noted that more than one movement may be entered per lane.

   If short right or left turn lanes impacting an adjacent through lane's capacity exist, the number of through lanes or calculated saturation flow rate should be reduced by an amount corresponding to the loss in capacity; e.g. if a 10 percent loss in capacity occurs, one should reduce the number of through lanes or calculated saturation flow rate input to the program by 10 percent.



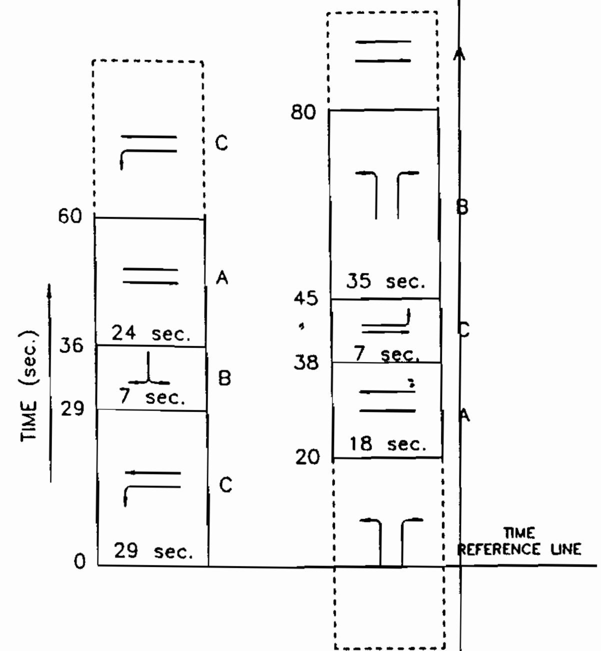
PHASING CODE	LEFT SIDE PHASE SEQUENCE			LEFT TURN SEQUENCE	RIGHT SIDE PHASE SEQUENCE		
1	A	B	C	LEAD-LEAD	A	B	C
2	A	C	B	LAG-LEAD	A	B	C
3	A	B	C	LEAD-LAG	A	C	B
4	A	C	B	LAG-LAG	A	C	B
1A	A	B	C	TTI-LEAD	A	B	C

Figure 4-2. PASSER III Phase Sequence Codes



Phasing = ABC/ABC or TTI Lead-Lead

Internal Offset = Beginning of the arterial phase on the left side of the interchange to the ending of the ramp or frontage road phase on the right side of the interchange. The arterial phase on the left side starts at 0 seconds on the time reference line and frontage road phase on the right side ends 10 seconds later on the right. Therefore, the internal offset is 10 seconds for the existing conditions.



Phasing = ACB/ACB or Lag-Lag

Internal Offset = Beginning of the arterial phase on the left side of the interchange to the ending of the ramp or frontage road phase on the right side of the interchange. The arterial phase on the left side starts 36 seconds after the time reference line and the frontage road phase on the right side ends 80 seconds after the time reference line. Therefore, the internal offset is 44 (80 - 36) seconds for the existing conditions.

Figure 4-3. Examples of PASSER III Internal Offset Calculations

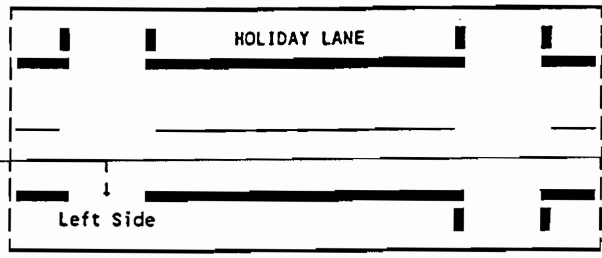
```
                    P A S S E R   III  -  90            Version 1.00
            Texas Department of Highways & Public Transportation

                          INTERCHANGE  1 DATA

             Cross Street Name        Permitted Left Turns (Y/N)
               HOLIDAY LANE           Left-side: Y  Right-side: Y

            Interior-Travel-Time       Interior-Queue-Storage
             --Rgt-->    <--Lft--       Left-Side  Right-Side
              (Sec)        (Sec)        Left Thru  Left Thru
               12           12           10   10    10   10

                         SIGNAL PHASING DATA

        Type of      Run Delay-Offset   ----- Seconds Offset -----
        Phasing      Analysis (Y/N)     Forced Int.     Forced Ext.
        -------          -----          -----------     -----------
        Lead-Lead          N
        Lag-Lead           N
        Lead-Lag           N
        Lag-Lag            N
        TTI-Lead           N *             10

        <Esc>exit screen    <PgUp><PgDn>next interchange
```

*For existing conditions, enter N for all Run Delay-Offset Analysis options, and enter the existing internal offset in the column for Forced Internal Offset and the row for the existing phasing type.

Figure 4-4. Signal Data Screen - Simulating Existing Conditions

P A S S E R III - 90 Version 1.00

Texas Department of Highways & Public Transportation

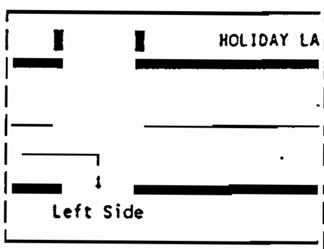


Vol/ Hour	Sat Flow	Min Phase	CROSS-STREET	Vol/ Hour	Sat Flow	Min Phase
··35	342		right-turn	228	1114	
144	1405	10	straight-through	102	499	10
125	1800		straight-then-left	602	1800	
			FRONTAGE ROAD			
8	562		right-turn	139	1081	
42	2952	10	straight-through	302	2349	10
291	1289		left-then-straight	200	1114	
95	421		left-then-left (U)	107	596	
			INTERIOR			
602	1710	10	left-turn	125	795	10
302	1800		straight-through	435	2765	

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

P A S S E R III - 90 Version 1.00

Texas Department of Highways & Public Transportation



Assistance For

LEFT SIDE, CROSS-STREET

Ideal Saturation Flow Rate	1800
Approach Grade (%)	0.0
Number of Approach Lanes	2
Minimum Phase Length (Sec)	10

Movements	Movement Vol(vph)	Heavy Veh (%)
Rights	35	0
Thrus	144	0
Lefts	125	0

	Lane Width	Allowable Movements
Right Lane	12.0	RT
Left Lane	12.0	L

<F3> to Calculate, Then Exit

Vol/ Hour	Sat Flow	Min Phase	CROSS-STREE
35	342		right-turn
144	1405	10	straight-thro
125	1800		straight-then
			FRONTAGE RO
8	562		right-turn
42	2952	10	straight-thro
291	1289		left-then-str
95	421		left-then-lef
			INTERIOR
602	1710	10	left-turn
302	1800		straight-thro

<F3>assistance <Esc>exit <PgUp

Figure 4-5 Interchange Movement Screen

3. **Saturation Flow Rates.** PASSER III has the capability of calculating the saturation flow rate for each movement based on the *Highway Capacity Manual* methodology. The saturation flow rate for each movement is automatically calculated when the user exits the assistant window (after the remaining data such as percent grade, percent heavy vehicles, etc., are entered).

4. **Minimum Green.** For simulating existing conditions, one enters the existing phase times (G + Y + RC) instead of true minimum values. The sum of the phase times for each side should equal the existing cycle length for simulation purposes.

   If an actuated controlled interchange is simulated, the user enters the average green times and clearance intervals as the minimum phase times, see Data Collection for a description of how to determine average phase times and cycle lengths. One should note, however, that actuated control will operate better than predicted by PASSER III as long as volume to capacity ratios are less than 0.95.

After entering all the required data for existing conditions, the user presses the <ESC> key and then returns to the *Main Menu*. The next step is to Run the program.

4.3 Calibration

It is appropriate at this point to stress the importance of input data quality and program calibration. Incorrect or inaccurate data will result in the program's output not representing the actual conditions in the field. Thus, the program's output may indicate a problem when, in fact, one does not exist (or vice versa). Any new timing plan developed from this data will not be the optimum for the conditions that exist at the interchange. Thus, it is extremely important that the program's output accurately reflect existing operation. Otherwise, your results are meaningless. **It is strongly recommended that no optimization be done until the analyst is satisfied that the program is properly calibrated.**

For example, if the program predicts oversaturation or long delays for movements that you know from field observations operate at an acceptable level-of-service, it is probable that the program underestimated the movement's saturation flow rate (assuming no data coding errors). Likewise, if the program predicts undersaturation or short delays for movements that you know from field observations experience cycle failures and long delays, it is probable that the program overestimated the movement's saturation flow rate. In the first example, the program will overestimate delay and attempt to allocate additional green time to accommodate vehicles that do not really exist. In the second example, the program will underestimate measures of effectiveness and fail to allocate enough green time to accommodate those vehicles that do exist.

4.4 Output Interpretation

After running PASSER III, the *Output Menu* (Figure 4-6) will appear when processing is complete. The user may view individual sections of the output or the entire output. To print the output file, the user either selects *Entire Output* or a corresponding section of the output in the *Output Menu* and then hits the <F3> key.

The output data contained for each screen is explained below:

Problem Identification Data. This screen contains general information used for identification purposes such as:

1. Freeway Name;
2. City Name;
3. District Number;
4. Date; and
5. Run Number.

Input data for the isolated intersection such as:

1. Number of interchanges = 1;
2. Lower cycle length;
3. Upper cycle length;
4. Cycle increment; and
5. Whether internal offsets were optimized or evaluated.

Movement Interchange Data. The input values for volumes, saturation flows, and minimum phase times are listed.

Interchange Phasing Data. The user specified phasing for PASSER III to analyze are listed, along with user specified internal offset, internal queue storage, permitted left-turn treatment, and interior travel times.

Delay Offset Diagrams. This screen became available when the user requested an internal offset-delay optimization run. When analyzing existing conditions or evaluating a specific internal offset, the optimization is skipped and the screen is not available in the output file.

General Signalization Information. This screen contains the phase times, v/c ratio, delay, and storage ratio for each movement for the left and right side of the interchange. The total interchange delay, phase order, and internal offset are also noted. PASSER III also assigns levels of service to various measures of effectiveness such as delay, v/c ratio, and the storage ratio. Table 4-2 shows level of service criteria used by PASSER III.

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


                            OUTPUT MENU

                Problem     - Identification Data
                Movement    - Interchange Data
                Interchange- Phasing Data
                Link        - Geometry Data
                Delay       - Offset Diagrams
                Optimal     - Progression Solution
                Frontage Rd- Progression Information
                General     - Signalization Information
                Signal      - Phasing Information
                Time        - Space Diagram
                Entire      - Output File
                Return      - To MAIN MENU

                     ENTER YOUR CHOICE --> R
          Print File = C:\TRAFFIC\PASSER3\DATA\HOLIDAY.OUT


                    Escape key to exit screen
```

Figure 4-6. PASSER III Output Menu

Table 4-2 Level of Service Criteria for Operational Measures of Effectiveness at Signalized Diamond Interchanges

Measures of Effectiveness	Level of Service					
	A	B	C	D	E	F
Volume to Capacity Ratio[a]	<0.6	<0.7	<0.8	<0.85	<1.0	>1.0
Average Vehicle Delay (sec/veh)[b]	<6.5	<19.5	<32.5	<52.0	<78.0	>78.0
Interior Storage Ratio[c]	<0.05	<0.10	<0.30	<0.50	<0.80	>0.80

[a]Source. Guide for Designing and Operating Signalized Intersections in Texas (12).
[b]Highway Capacity Manual, 1985 - stopped delay multiplied by 1.3 for total delay (5).
[c]PASSER III-84 User's Manual.

Signal Phasing Information. This screen gives the phase interval number, the left side phase sequence, right side phase sequence, and the corresponding phase interval length. The cycle length, phase order, and internal offset are also noted.

Other Output. The *Link Geometry Data*, *Optimal Progression Solution*, *Frontage Road Progression Information*, and the *Time Space Diagram* screens pertain to progression systems and are not available for isolated interchanges.

4.5 Example Problem

The next section addresses the evaluation of an existing diamond interchange. A retiming plan has been requested for an isolated diamond interchange in a Texas urban area. The interchange is isolated and considered a full diamond with frontage roads. The interchange operates with a three-phase, lead-lead, phase sequence, and two interconnected pretimed controllers. The existing cycle length equals 90 seconds. It has been determined that the a.m. peak timing plan will be evaluated.

Two-hour turning movement counts were conducted during the a.m. peak period from 7:00 to 9:00 a.m. and the peak 15 minute period was determined to be from 8:30 to 8:45. The previous chapter, Data Requirements, discusses further the process for collecting the data for the existing conditions. The 15 minute counts were multiplied by four to determine the design hourly volumes. Figures 4-7 and 4-8 illustrate the existing geometry and traffic volumes at the interchange.

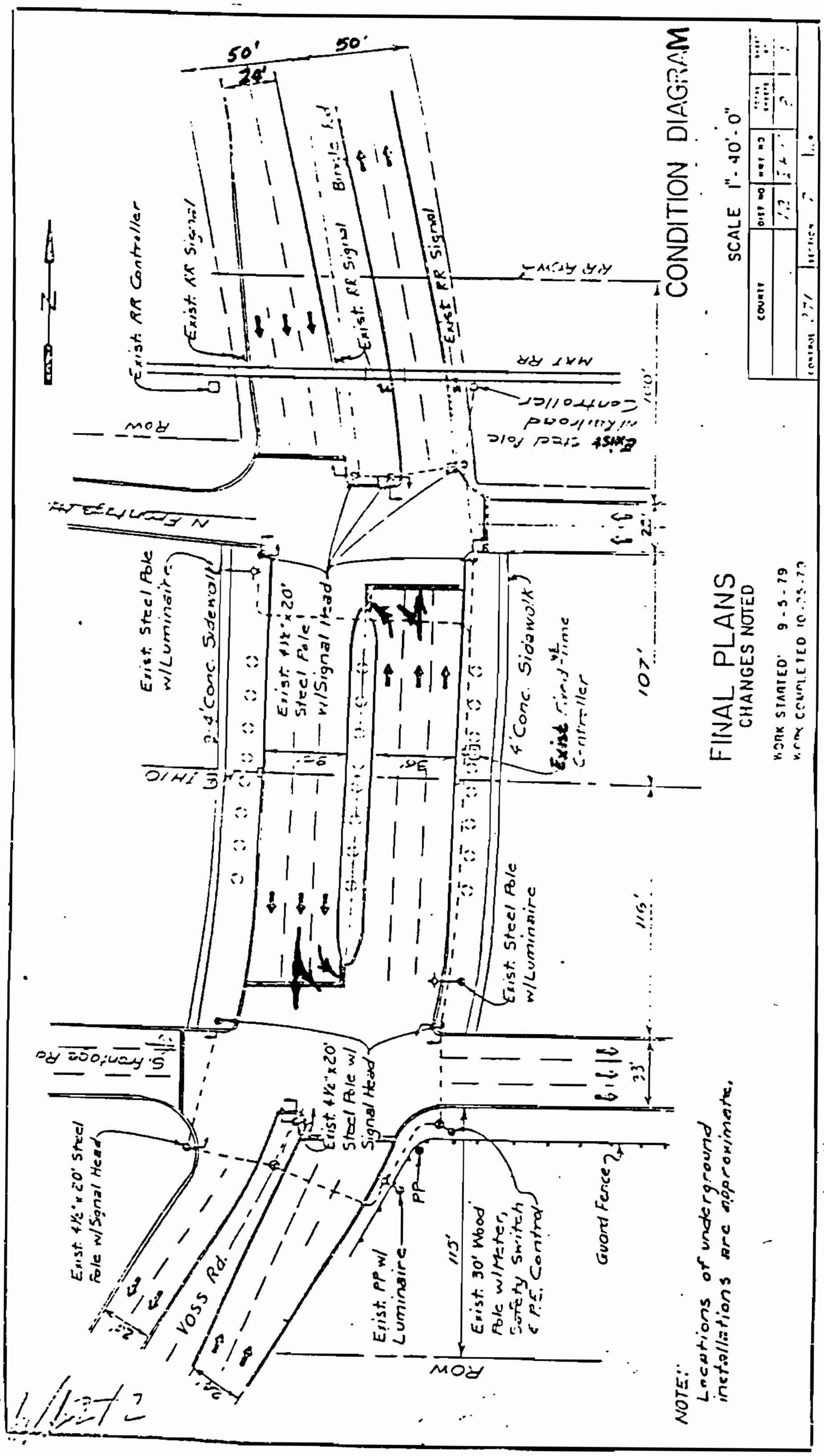


Figure 4-7. Plan Sheet of Example Diamond Interchange

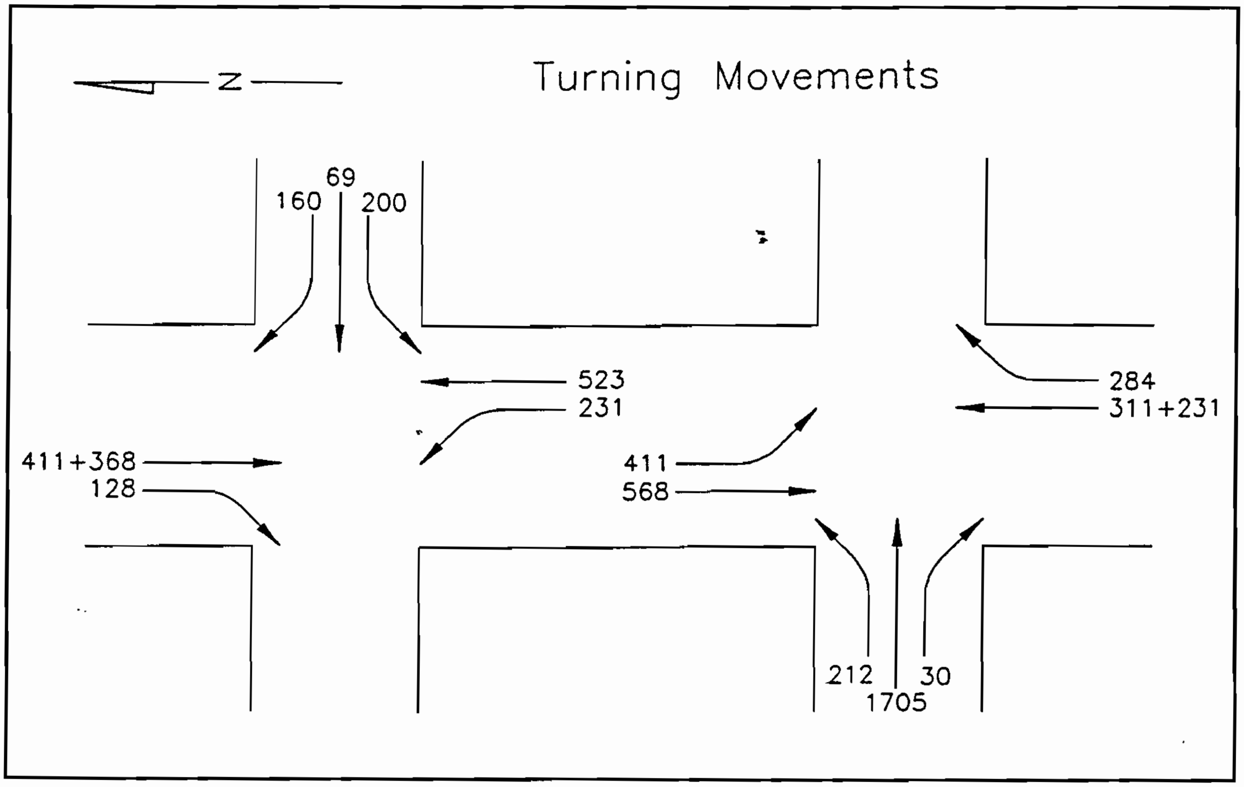


Figure 4-8. Turning Volumes for Example Diamond Interchange

Entering the Data. The next step in the evaluation process is running PASSER III to evaluate the existing conditions at the interchange and to determine needed improvements. The analyst should follow the process described below for simulating existing conditions. Site specific comments are as follows:

Freeway Identification. Analysts entered the existing cycle length of 90 seconds for the upper and lower cycle length, and the number of interchanges equals 1 for the isolated mode.

Interchange and Signal Phasing Data. The cross street is Bingle Road, and there are no permitted left turns; therefore, 'N' is coded for permitted left turns.

Interior Travel Time. Analysts did not determine the interior travel time in the field, but, rather, based it on the interchange width of 243 feet, which corresponds to an estimated travel time of 11 seconds (see Table 4-1). One should note that, although actual field data yields more accurate results, Table 4-1 can be used when such data is not available. It is important to remember, however, that the values in the table are estimates and that the resultant signal timing plan may have to be fine tuned after actual implementation in the field.

Interior Queue Storage. Three interior lanes for both directions exist, including one exclusive left lane, one exclusive through lane, and a shared left plus through lane. Based on 25 feet per vehicle, analysts determined that enough storage for 9 left turning vehicles and 18 through vehicles on the left side, and for 11 left turning vehicles and 16 through vehicles on the right side existed. An example calculation is shown below:

The storage distance for the diamond interchange is 223 feet and 3 lanes in each direction are available for storage.

(223 feet)/(25 ft/veh) = 9 veh per lane * 3 lanes = 27 vehicles

Because the lane is an optional left or through lane, none, part, or all of it can be allocated for storage of left-turning vehicles; i.e., the minimum and maximum storage for left turns is 9 and 18 vehicles, respectively. One usually determines the actual allocation as a proportion of the total traffic.

There are 523 vehicles going through the interior and 231 left-turning vehicles on the left side of the interchange. The total number of vehicles equals 754 vehicles. Thirty percent of vehicles are turning left, and 70 percent are going through. Using proportions:

(27 veh. total) * (0.30 left turns) = 8.1 or 8 vehicles;
(27 veh. total) * (0.70 throughs) = 18.9 or 19 vehicles;

Because through movements cannot be in the left lane, however, left turns occupy one lane (9 vehicles storage), and the through movements occupy two lanes or 18 vehicles storage.

Additional Lanes. In general, increasing the number of lanes and/or providing exclusive lanes for each movement implies increased capacity and thereby reduced delay. Right-of-way acquisition and the construction of additional lanes at the interchange, however, is an expensive improvement strategy, and one usually resorts to this option only after all other strategies have proved unsuccessful.

To simulate the effects of this improvement, the user accesses the *Movement Interchange Data Screen* and uses the assistant key to simulate the addition of extra lanes. The user increases the total number of lanes and then makes adjustments to the type of movements allowed from those lanes. New saturation flow rates will be calculated based on the additional lanes and allowable movements.

5.6 Example

The following example uses data from the interchange discussed in Section 4.5. The following text discusses optimization strategies and PASSER III results based on those strategies. For convenience, Figure 5-2 illustrates the measures of effectiveness for the existing conditions.

```
<GS101>
* * * INTERCHANGE  1 BINGLE                                     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)  22.0   20.0   48.0   70.0   *  23.0   39.0   28.0   51.0
                                               *
 V/C RATIO          1.14    .75    .29    .19  *   .76    .97    .69    .35
  LEVEL OF SERVICE  F      C      A      A     *   C      E      B      A
                                               *
 DELAY (SEC/VEH) 156.91  59.18    .36    .05   *  43.12 166.94  3.91    .00
  LEVEL OF SERVICE  F      E      A      A     *   D      F      A      A
                                               *
 STORAGE RATIO                    .04    .03   *                .04    .00
  LEVEL OF SERVICE                A      A     *                A      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   LEAD-LEAD      TOTAL INTERCHANGE DELAY   64.12 VEH-HRS/HR
   INTERNAL OFFSET   5 SEC      CYCLE LENGTH    90 SEC
```

Figure 5-2. Measures of Effectiveness for Existing Conditions

Optimize Offset. The first optimization strategy tried was to determine if the existing offset was optimal. Using the existing green splits, phasing and cycle length, analysts optimized the internal offset by entering a 'Y' beside the lead-lead phase sequence. PASSER III reported that a 6 second offset would be better than the existing 5 second offset.

In examining the program's output, however, the only real effect was on the interior storage ratios (which went to 0, indicating that no vehicles are required to stop in the interior of the interchange with the 6 second offset). The total interchange delay was reduced slightly, but only by approximately 1 percent. No significant decrease occurred in v/c ratios. Figure 5-3 illustrates the results of this optimization run.

```
<GSI01>
* * * INTERCHANGE  1 bingle                                     RUN 01 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)  22.0   20.0   48.0   70.0    *  23.0   39.0   28.0   51.0
                                                *
 V/C RATIO          1.14   .75    .29    .19    *   .76    .97    .69    .35
  LEVEL OF SERVICE  F      C      A      A      *   C      E      B      A
                                                *
 DELAY (SEC/VEH)  156.91  59.18   .00    .00    *  43.12 166.94   .00    .00
  LEVEL OF SERVICE  F      E      A      A      *   D      F      A      A
                                                *
 STORAGE RATIO                    .00    .00    *                 .00    .00
  LEVEL OF SERVICE                A      A      *                 A      A
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   LEAD-LEAD      TOTAL INTERCHANGE DELAY   63.65 VEH-HRS/HR
   INTERNAL OFFSET   6 SEC      CYCLE LENGTH   90 SEC
```

Figure 5-3. Measures of Effectiveness for Offset Optimization

Optimize Green Splits. The next optimization strategy tried was to determine if the green splits could be reapportioned to provide better performance. The existing cycle length of 90 seconds and lead-lead phase sequence remained constant, but analysts set the minimum green times to 10 seconds for each movement. In addition, a 'Y' was entered beside the lead-lead phase sequence for optimization of the offset.

The results of this strategy show that an internal offset of 28 seconds and new green split allocations provide noticeably lower v/c ratios and better levels of service. Level of Service A was reported for the movements at the left side intersection, and Level of Service D was reported for the movements at the right side intersection. This result is a significant improvement over existing conditions. The new timing plan decreased the total interchange delay to 40.78 vehicle-hours per hour, or a 36 percent decrease in delay from existing conditions. The storage ratio for the left and right side of the interchange, however, increased compared to existing conditions. Figure 5-4 illustrates the results from this run.

```
<GSI01>
* * * INTERCHANGE  1 bingle                                     RUN 01 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)  39.6   24.6   25.8   65.4  *  21.5   44.5   24.0   45.5
                                              *
 V/C RATIO          .58    .58    .59    .21  *   .83    .84    .83    .40
  LEVEL OF SERVICE  A      A      A      A    *   D      D      D      A
                                              *
 DELAY (SEC/VEH)   22.44  37.44   6.21   1.09 *  51.32  94.33  26.88  10.93
  LEVEL OF SERVICE  C      D      A      A    *   D      F      C      B
                                              *
 STORAGE RATIO                    .24    .15  *                 .88    .48
  LEVEL OF SERVICE                C      C    *                 F      D
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER   LEAD-LEAD      TOTAL INTERCHANGE DELAY   40.78 VEH-HRS/HR
   INTERNAL OFFSET  28 SEC      CYCLE LENGTH    90 SEC
```

Figure 5-4. Measures of Effectiveness for Green Split Optimization

Optimize Cycle Length. The next optimization strategy tried was to determine an optimal cycle length. Using Webster's Minimum Delay Cycle equation, analysts determined an 82 second cycle to be the cycle for minimum delay. Two runs were made varying the cycle length ranges from 75 to 90 seconds, and from 95 to 110 seconds. Analysts modeled the same phase sequence (lead-lead) but set the minimum green times to 10 seconds and optimized the offset for each cycle.

PASSER III reported that a cycle between 80 and 85 seconds yields minimal delay while providing a reasonable level of service for the v/c ratios. This result indicates that the 90 second cycle for existing conditions proved reasonable. A 26 second offset was determined as optimal for the 85 second cycle. Delay is reduced an additional 3.2 percent; however, the storage ratio still presents a concern on the right side of the interchange. Figure 5-5 illustrates the results of this run.

<GSI01>
* * * INTERCHANGE 1 bingle   RUN 01 PAGE 6A

*** GENERAL SIGNALIZATION INFORMATION ***

MEASURES OF EFFECTIVENESS	LEFT-SIDE INTERSECTION A	B	C	A+C	RIGHT-SIDE INTERSECTION A	B	C	A+C
PHASE TIME (SEC)	37.3	23.3	24.4	61.7	20.4	41.9	22.7	43.1
V/C RATIO	.58	.59	.59	.21	.84	.84	.84	.40
LEVEL OF SERVICE	A	A	A	A	D	D	D	A
DELAY (SEC/VEH)	21.56	36.13	4.35	.56	50.38	96.55	25.80	10.48
LEVEL OF SERVICE	C	D	A	A	D	F	C	B
STORAGE RATIO			.15	.09			.83	.47
LEVEL OF SERVICE			C	B			F	D

PHASE ORDER LEAD-LEAD   TOTAL INTERCHANGE DELAY 39.49 VEH-HRS/HR
INTERNAL OFFSET 26 SEC   CYCLE LENGTH 85 SEC

Figure 5-5. Measures of Effectiveness for Cycle Length Optimization

Phase Sequence Optimization. The next optimization strategy tried was to determine if an optimal phasing sequence exists that will provide minimal delay, acceptable v/c ratios and acceptable storage ratios. Analysts used a cycle length range of 75 to 90 seconds based on previous runs. A 'Y' was entered beside all possible phasing patterns.

Lag-lead phasing with a cycle length between 80 and 85 seconds and an offset of 6 seconds resulted in an interchange delay of 38.57 veh-hrs/hr and 39.26 veh-hrs/hr, respectively. The storage ratio is acceptable and improved from the existing conditions. Figure 5-6 illustrates the results of the 85 second phase sequence optimization run.

```
<GS101>
* * * INTERCHANGE  1 bingle                                        RUN 01 PAGE  6A

                 ***    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)  37.3   23.3   24.4   61.7   *  20.4   41.9   22.7   43.1
                                               *
 V/C RATIO          .58    .59    .59    .21   *   .84    .84    .84    .40
  LEVEL OF SERVICE  A      A      A      A     *   D      D      D      A
                                               *
 DELAY (SEC/VEH)   21.56  36.13   .00   5.42   *  50.38  96.55  25.33  6.64
  LEVEL OF SERVICE  C      D      A      A     *   D      F      C      B
                                               *
 STORAGE RATIO                    .00    .24   *                 .33    .31
  LEVEL OF SERVICE                A      C     *                 D      D
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    LAG -LEAD      TOTAL INTERCHANGE DELAY   39.26 VEH-HRS/HR
   INTERNAL OFFSET   6 SEC       CYCLE LENGTH    85 SEC
```

Figure 5-6. Measures of Effectiveness for Phasing Optimization

Left-Turn Treatment. The next strategy involved evaluating the effect of allowing protected plus permitted left-turn movements. Analysts used the optimized conditions from the previous run as a starting point. Lag-lead phasing, 85 second cycle, and a 6 second offset were evaluated with minimum green splits set equal to 10 seconds. A 'Y' was entered for permissive left turns at the left and right side of the interchange.

Results from PASSER III show that total interchange delay was reduced only slightly, due to the fact that the opposing traffic volume was heavy, and vehicles were unable to utilize the permitted portion of the left-turn phase. Figure 5-7 illustrates the results of this run, and Table 5-1 summarizes the results and comparison of existing conditions and the optimization strategies tried in this example problem.

```
<GSI01>
* * * INTERCHANGE  1 bingle                                        RUN 01 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)  37.3   23.3   24.4   61.7 *  20.4   41.9   22.7   43.1
                                               *
 V/C RATIO           .58    .59    .59    .21 *   .84    .84    .84    .40
  LEVEL OF SERVICE   A      A      A      A   *   D      D      D      A
                                               *
 DELAY (SEC/VEH)   21.56  36.13   .00   5.42 *  50.38  96.55  22.98   6.64
  LEVEL OF SERVICE   C      D      A      A   *   D      F      C      B
                                               *
 STORAGE RATIO                     .00    .24 *                 .30    .31
  LEVEL OF SERVICE                 A      C   *                 D      D
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    LAG -LEAD      TOTAL INTERCHANGE DELAY    38.99 VEH-HRS/HR
   INTERNAL OFFSET    6 SEC      CYCLE LENGTH    85 SEC
```

Figure 5-7. Measures of Effectiveness for Permissive Lefts

Table 5-1 Comparison of Existing Conditions and Optimization Strategies

Conditions	Cycle Length	Interchange Delay (veh-hrs/hr)	Maximum v/c	Maximum Storage Ratio
Existing Conditions	90	65.04	1.14 (F)	0.33 (D)
Optimize Offset	90	63.65	1.14 (F)	0.00 (A)
Optimize Offset and Splits	90	40.55	0.84 (D)	0.67 (E)
Optimize Cycle Length	80	39.49	0.84 (D)	0.65 (E)
Optimize Phasing	85	39.26	0.84 (D)	0.38 (D)
Optimized Plus Permitted Lefts	85	38.99	0.84 (D)	0.38 (D)

6.0 IMPLEMENTATION

The next step in the retiming process is to enter the new (optimized) timing plan into the controller for field implementation. To accomplish this step in the process, the analyst must translate the computer output into useable settings for the controller. This requirement is due to the fact that the PASSER III output format remains constant, while controller hardware configurations vary. Although some similarities exist, translation of PASSER III output to each controller type format will differ.

This section addresses the PASSER III output as it relates to the phase numbering scheme found in the TxDOT solid state diamond interchange controller unit, as well as the calculation of green splits and clearance intervals from the program's output. Finally, a methodology for computing actuated controller settings is discussed, along with comments about pretimed controller settings. In the following discussion, this report will use the example interchange addressed in previous chapters for illustration.

6.1 Phase Numbers

The TxDOT diamond interchange controller is an actuated controller with phase numbers corresponding to the standard NEMA 8-phase controller. The controller assigns phase numbers to movements at the diamond interchange as shown in Figure 6-1. Basically the arterial or cross-street movements are assigned Phases 2 and 6, the frontage roads are assigned Phases 4 and 8, and the interior left turns are assigned Phases 1 and 5. The interior through movements are assigned 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 operates as two independent four-phase rings and has the capability of switching between four-phase and three-phase diamond operation. Figure 6-2 shows the ring assignments.

The example discussed on the following pages illustrates the relationship between the *Signal Phasing Information* output (Figure 6-3) and diamond interchange phasing numbers as shown in Figure 6-2. The optimized timing plan from previous examples is used to illustrate the comparison. Analysts determined the optimal phasing to be lag-lead phasing with a 6 second internal offset. Figure 6-4 shows the relationship between PASSER III output phase sequencing and the diamond interchange phase numbers.

On the left side of the interchange, PASSER III's Phase A corresponds to the controller Phase 2, and has a duration, including clearance, of 50.5 (38.5 + 12.0) seconds. PASSER's Phase C corresponds to controller Phase 1, and has a duration, including clearance, of 30 (12 + 18) seconds. Finally, PASSER's Phase B corresponds to controller Phase 4, and has a duration, including clearance, of 19.5 (6.0 + 13.5) seconds.

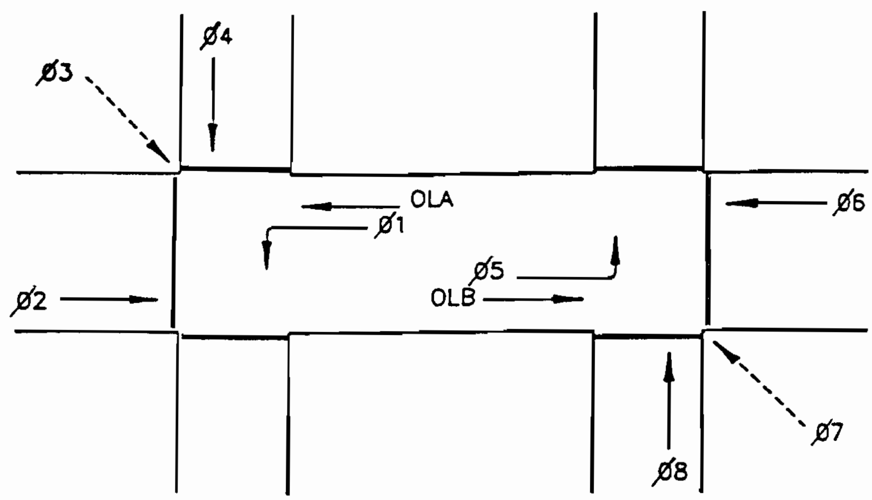


Figure 6-1. Phase Number Assignments for Diamond Interchange Movements

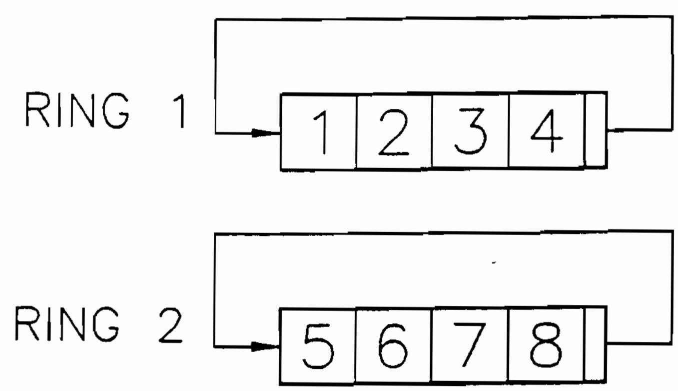


Figure 6-2. NEMA 8-Phase Controller Illustration

```
<SPI01>
* * * INTERCHANGE  1 bingle                                          RUN 01 PAGE  68

                     ***   SIGNAL PHASING INFORMATION    ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
                * LEFT-SIDE SEQUENCE   * RIGHT-SIDE SEQUENCE  *
                *  A      C       B    *  A      B        C   *
PHASE INTERVAL  * <----  <----    |    * <----   ^        ^   *  PHASE INTERVAL
    NUMBER      *        |----    |    *         |    ----|   *   LENGTH (SEC)
                * ---->  V        V    * ---->   |    ---->   *
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
      1         *         A            *         B            *        6.00
      2         *         A            *         C            *       22.70
      3         *         A            *         A            *        8.60
      4         *         C            *         A            *       11.80
      5         *         C            *         B            *       12.60
      6         *         B            *         B            *       23.30
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   INTERNAL OFFSET      6 SEC                          CYCLE LENGTH   85 SEC
                                                       PHASE ORDER LAG -LEAD
```

Figure 6-3. PASSER III Signal Phasing Information Output Screen

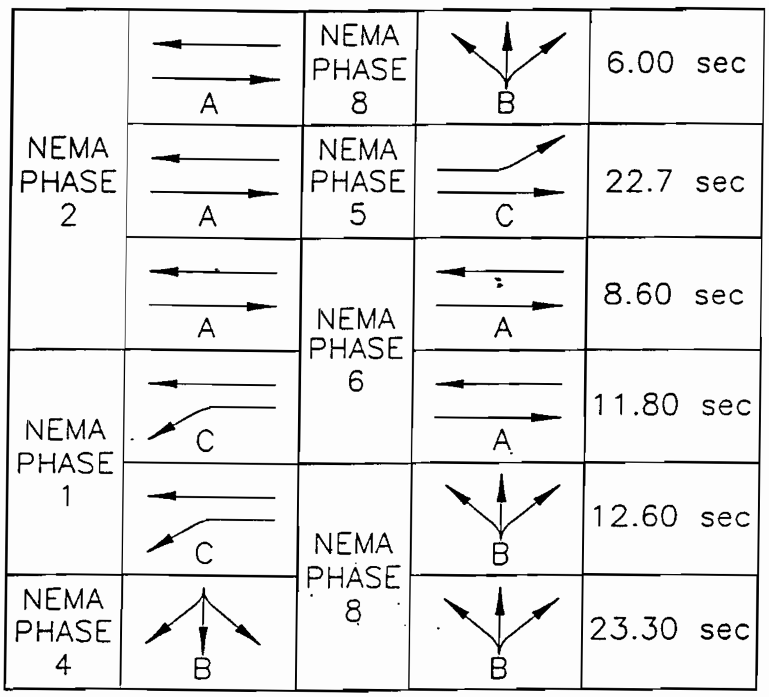


Figure 6-4. Relationship between Diamond Controller and PASSER III Phases

On the right side of the interchange, PASSER's Phase A corresponds to controller Phase 6, and has a phase duration, including clearance, of 24 (6.0 + 18.0) seconds. PASSER's Phase C corresponds to controller Phase 5, and has a phase duration, including clearance, of 52 (38.5 + 13.5) seconds. PASSER's Phase B corresponds to controller Phase 8, and has a duration, including clearance, of 24 (12.0 + 12.0) seconds. Overlap A is concurrent with PASSER III's left-side Phases A and C, and has a duration of 80.5 (18.0 + 12.0 + 12.0 + 38.5) seconds. Overlap B is concurrent with PASSER III's right-side Phases A and C, and has a duration of 76.0 (38.5 + 13.5 + 6.0 + 18.0) seconds.

6.2 Phase Lengths

As mentioned previously, the phase lengths reported by PASSER III include the green plus yellow plus any red clearance time for that phase. PASSER III reports the phase lengths in two places: *General Signalization Information*, and *Signal Phasing Information*. Figure 6-5 shows the *General Signalization Information* output with the *Signal Phasing Information* below it.

The *General Signalization Information* reports the phase lengths for Phase A,B, and C or 6, 8, and 5 on the right side of the interchange. One determines green interval durations by subtracting the yellow plus red clearance time from the reported phase length. The same information is reported for phases A, B, and C or 2, 4, and 1 on the left side of the interchange.

The Signal Phasing Information reports phase interval lengths. PASSER III Phase Interval 1 in Figure 6-5 corresponds to controller Phase 2 + 8, PASSER III Phase Interval 2 corresponds to controller Phase 2 + 5, PASSER III Phase Interval 3 corresponds to controller Phase 2 + 6, PASSER III Phase Interval 4 refers to controller Phase 1 + 6, PASSER III Phase Interval 5 refers to controller Phase 1 + 8 and finally, PASSER III Phase Interval 6 corresponds to controller Phase 4 + 8.

```
<GSI01>
* * * INTERCHANGE  1 bingle                                    RUN 01 PAGE  6A

                 ***   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)  37.3   23.3   24.4   61.7   *  20.4   41.9   22.7   43.1
                                                *
 V/C RATIO           .58    .59    .59    .21  *   .84    .84    .84    .40
  LEVEL OF SERVICE  A      A      A      A     *   D      D      D      A
                                                *
 DELAY (SEC/VEH)   21.56  36.13    .00   5.42  *  50.38  96.55  25.33   6.64
  LEVEL OF SERVICE  C      D      A      A     *   D      F      C      B
                                                *
 STORAGE RATIO                     .00    .24  *                 .33    .31
  LEVEL OF SERVICE                 A      C    *                 D      D
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    LAG -LEAD      TOTAL INTERCHANGE DELAY   39.26 VEH-HRS/HR
   INTERNAL OFFSET   6 SEC       CYCLE LENGTH   85 SEC
```

```
<SPI01>
* * * INTERCHANGE  1 bingle                                    RUN 01 PAGE  6B

                  ***   SIGNAL PHASING INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
                * LEFT-SIDE SEQUENCE   * RIGHT-SIDE SEQUENCE  *
                *   A       C      B   *   A       B      C   *
PHASE INTERVAL  * <----   <----    |   * <----     ^        ^ *  PHASE INTERVAL
   NUMBER       *         |----    |   *           |    ----| *  LENGTH (SEC)
                * ---->   V        V   * ---->     |    ----> *
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                      *                      *
     1          *         A            *           B          *      6.00
     2          *         A            *           C          *     22.70
     3          *         A            *           A          *      8.60
     4          *         C            *           A          *     11.80
     5          *         C            *           B          *     12.60
     6          *         B            *           B          *     23.30
                *                      *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
  INTERNAL OFFSET     6 SEC                         CYCLE LENGTH   85 SEC
                                                    PHASE ORDER LAG -LEAD
```

Figure 6-5. PASSER III Signal Phasing Output Screens

6.3 Actuated Controller Settings

PASSER III was designed to analyze pretimed or traffic-responsive fixed-sequence signalized diamond interchanges. The phase lengths reported by PASSER III are generally thought of as pretimed settings. No guidance is given as to how the PASSER III phase lengths are converted to actuated settings. The following guidelines for converting PASSER III output into actuated settings are based, in part, on a procedure developed by Skarbardonis for converting PASSER II settings to actuated controller settings (12).

The discussion focuses on the three phases that occur on each side of the interchange. It is assumed that the ring configuration and phase relationships are as shown in Figures 6-1 and 6-2. Other assumptions include that PASSER III's Phase A, the arterial or cross road through phase, is the sync phase, and that a constant cycle length and phase sequence will be used during the control period. The following procedural guidelines were obtained from the FHWA Report (13). One should apply the procedures to both sides of the interchange.

Minimum Green. The minimum green duration for Phases A, B, and C (Phases 2, 4, and 1 or Phases 6, 8, and 5, respectively) are based on three factors: pedestrian walk time, driver expectancy, and the location of the loop detector in relation to the stop bar. In the case of pedestrians, the phase length for the street parallel to the pedestrian's path should be long enough to allow the pedestrian to cross safely. Referring to the second factor, a phase length should be at least 6 to 10 seconds to satisfy a driver's expectancy. Regarding the location of the detector, the minimum green time should be long enough to clear all vehicles stopped between the detector and the stop bar when using advance detection; however, one should note that minimum green times can be shorter than 6 seconds when using presence detection (i.e., no stop line detection). The analyst should base the minimum green time on the largest green time requirement of the three factors.

The following relationships describe the calculations of the minimum green and phase durations. Minimum phase length durations include green plus yellow plus red clearance.

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

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

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

For phase C:
(Phase 1 and 5)

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

where:

G_{min} = minimum green interval duration for phase, in seconds;
P_{min} = minimum duration of phase, in seconds;
Y = yellow interval duration of phase, in seconds;
RC = red clearance interval duration of phase, in seconds;
D = distance from stop-line to nearest edge of detector serving phase, in feet;
L_Q = space occupied by queued vehicle, in feet; (use 25 feet/vehicle);
S = saturation flow rate of critical movement, in vphgpl;
l_1 = start-up lost time in phase, in seconds; (use 2.0 seconds);
l_2 = ending lost time in phase, in seconds; (use 2.0 seconds);
W = steady WALK interval for phase, in seconds;
FDW = flashing DON'T WALK for phase, in seconds; see Table 6-1.

Vehicle Extension for Stop line Detection. The vehicle extension interval for stop line 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 queue demand has been served, one should establish the maximum allowable gap first and this value should be 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:

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

where:

GAP_{max} = maximum allowable gap for phase, in seconds; (see Figure 6-6)
Q = total flow rate on all approaches served during phase, in vph;
D = average duration of extended green after the queue dissipates, in seconds.

Table 6-1 WALK and Flashing DON'T WALK Interval Durations. (7)

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: $f = 1 - e^{-P * C/3600}$
P = pedestrian flow rate during the control period, pph;

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

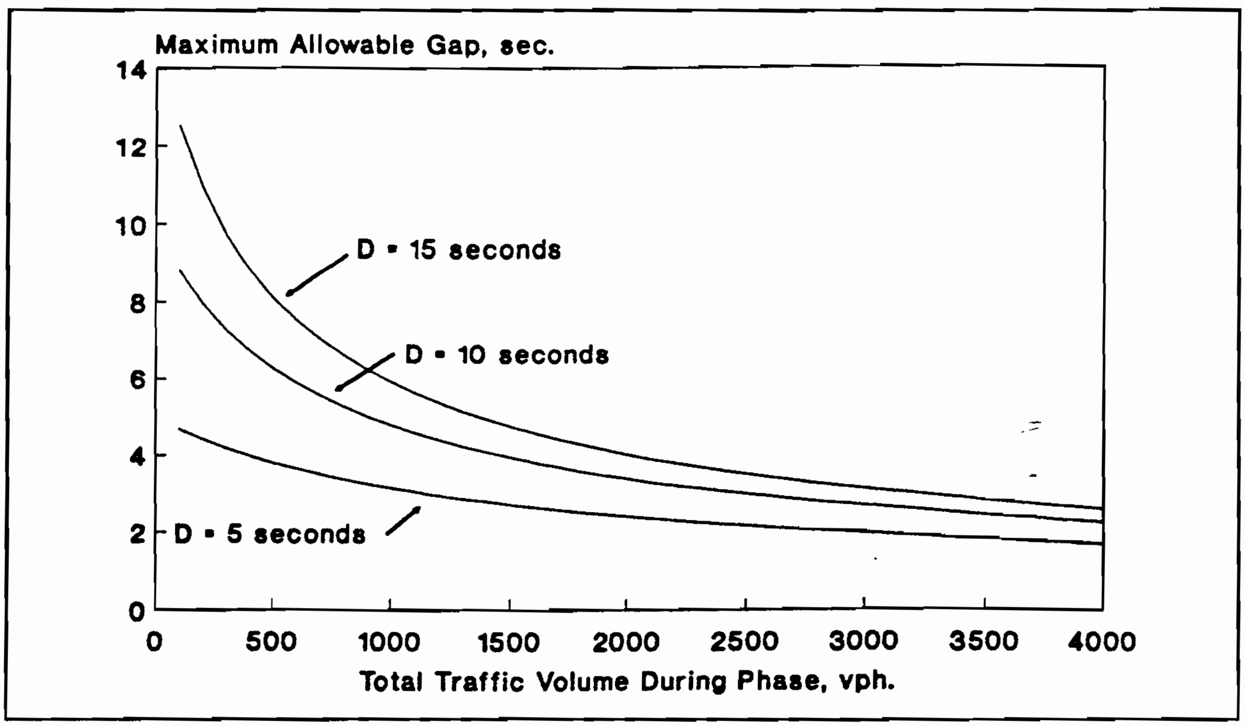


Figure 6-6. Maximum Allowable Gap versus Phase Volume

A reasonable value of D would seem to equal 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 analyst can use the following equation to calculate the duration of the vehicle extension for Phases B and C:

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

where:

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

Vehicle Extension for 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 presentation of the yellow interval. Thus, safety considerations for advance detection, as opposed to performance settings with stop line detection, dictate the vehicle extension setting.

In general, the vehicle interval equals the travel time from the detector to the stop line; however, this time can become so large for high speed approaches that it can prove 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 stop line. Studies have shown that almost all drivers who are less than 2.0 seconds from the stop line 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:

$$VE = \text{larger of} \begin{cases} [(D - 14)/V] - 1.5 \\ \text{or} \\ 2.0 \end{cases}$$

6.4 Minimum Phase Times for Pretimed Controllers

The minimum green time for pretimed controllers is based on the same factors as actuated signals except for the detector factor. Driver expectancy and pedestrian crossing times are basic factors for calculating the minimum green times for pretimed controllers.

6.5 Example of Minimum Phase Calculations and Vehicle Extension

The following example illustrates the calculations of minimum green times from PASSER III output. The example interchange is assumed to have actuated control with advanced detection (no stop line detection) and pedestrian push buttons for illustration purposes. Figure 6-7 shows a sketch of the interchange used in this example.

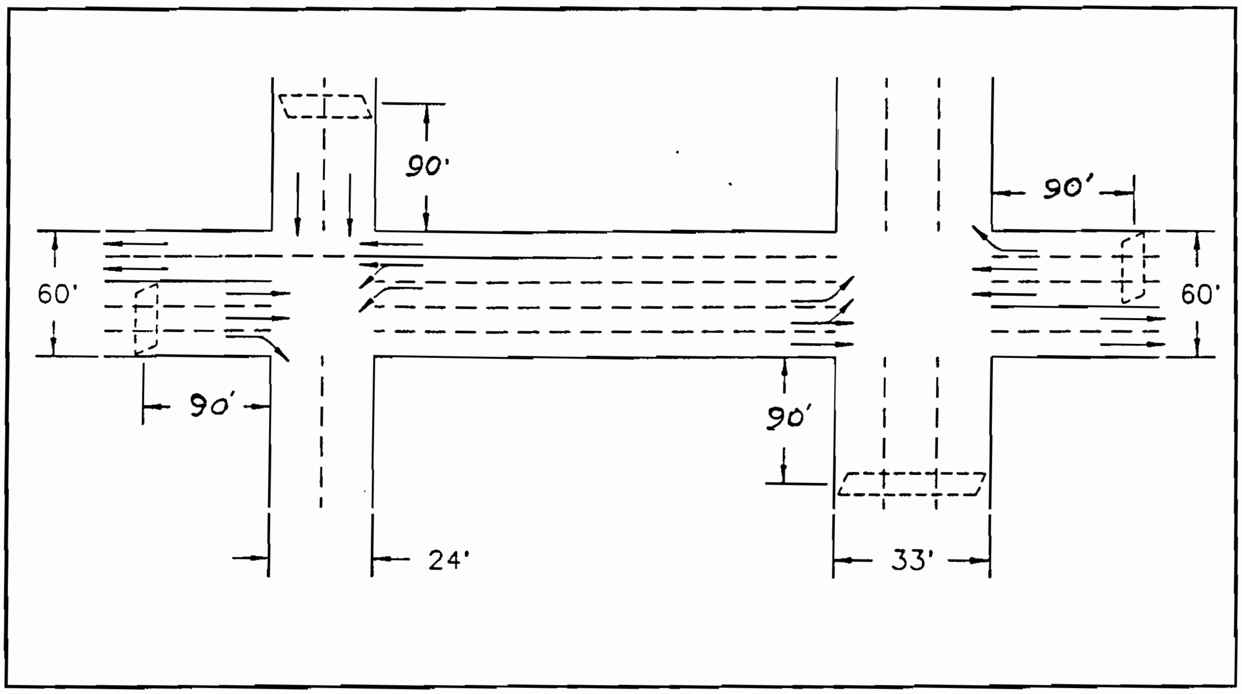


Figure 6-7. Sketch of Example Diamond Interchange

Minimum Green Time. For pedestrians crossing the arterial or cross street, the duration of Phase B will need to be long enough for pedestrians to safely cross the street. If for example, the pedestrian demand on the right-side arterial in the example interchange is greater than 10 pedestrians/cycle, then:

SOLID WALK = 7 seconds (from MUTCD);

FLASHING DON'T WALK = (60 - 6)/3.5 = 15.4 seconds; and

Minimum Phase Time, P_{min} = 7.0 + 15.4 = 22.4 or 23 seconds for pedestrians.

Detector type and location may also impose minimum phase constraints at the interchange. As mentioned, advance detection with no stop line detection is used at the example interchange. The minimum phase duration should be long enough to clear vehicles between the stop line and the detector:

$$P_{min} = (D/L_Q \times 3600/S) + (l_1 + l_2)$$

where:
D = 54 ft;
L_Q = 25 ft;
S = say 3400 vphgpl;
l_1 = 2.0 seconds; and
l_2 = 2.0 seconds;

$$P_{min} = \frac{54}{24} \times \frac{3600}{3400} + (2.0 + 2.0) = (2.16 \times 1.05) + 4.0 = 6.29 \, seconds$$

The larger of the two minimums (pedestrian or detector), 23 seconds in this case, equals the minimum phase time needed for the frontage phase on the right side of the interchange. The minimum green time would be:

23 sec. - Y - RC = 19 seconds

The phase time allocated to Phase B on the right side of the interchange is 42 seconds as seen previously in the *General Signal Information* output in Figure 6-5, which is longer than minimum value required by pedestrians. Analysts should make minimum phase calculations for Phase A and B (Phases 2, 6, 4, and 8) on each side of the interchange. One should compare the phase times recommended by PASSER III to these minimums to make sure the minimum requirements are satisfied.

Phase C minimum phase times are based on driver expectancy, 6.0 + Y + RC, and usually falls within the range of 8 to 10 seconds. Phase C on both sides of the interchange in the optimized example have phase times of approximately 20 seconds, well above the minimum values.

Vehicle Extension for Advanced Detection. As discussed earlier, the formula for estimating the vehicle extension for advanced detection is:

$$\begin{aligned} VE &= [(D - 14)/V] - 1.5 \\ &= [(90-14)/44] - 1.5 = 0.227 \end{aligned}$$

or 2.0 seconds;
0.2 is less than 0.227 seconds.

Therefore, use 2.0 seconds.

6.6 Maximum Green Calculations

The following steps form guidelines for calculating the maximum green and phase durations.

1. After PASSER III has optimized the phase lengths and all minimum phase requirements are met, calculations of the maximum green time follow.

2. Check the volume to capacity condition for each phase. The v/c ratio should equal 0.85 or less. If it is not, the phase will operate more nearly as a pretimed controller rather than actuated during the control period. One may obtain the v/c ratio directly from PASSER III's *General Signalization Information* screen.

3. If the phase's v/c ratio is less than 0.85, one can use the optimum phase duration from PASSER III output (G) to determine the maximum phase duration (G_{max}). In this case, the maximum phase duration is calculated as:

$$G_{max} = G.$$

If the phase v/c ratio falls between 0.85 and 0.95, one should calculate the maximum phase duration as (12):

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

If the v/c ratio is more than 0.95, the capacity of the interchange may be inadequate, or one should make a signal timing change to lower the v/c ratio below 0.95. If a signal timing change is made, the analyst may then calculate the maximum phase duration by the equation above.

One should make a check to ensure that the minimum phase duration is not greater than the maximum phase duration. This condition is possible if phase durations for Phases A and B were determined based on pedestrian actuations. If this occurs, the f value should be changed to 1.0. If P_{min} is still greater than G_{max}, set P_{min} equal to G_{max}.

One may calculate the maximum green interval (g_{max}) by:

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

Example of Maximum Phase Duration. The output from PASSER III's *General Signalization Information* for the example interchange in Figure 6-5 shows that all the v/c ratios are less than 0.85; therefore, the phase duration G may be assumed to be G_{max}.

$$g_{max} = G_{max} \text{ (or G)} - Y - RC$$

6.7 Yield and Force-off Points

A force-off point is defined as the point which terminates an actively timing actuated phase. A force-off is a non-latching pulse that must not be given while the controller is timing a minimum interval (minimum green, WALK, or pedestrian clearance). The yield point relates to the permissive period, but more specifically, to a point in the cycle of a coordinated system, where opposing phases are permitted to give right of way to one or more opposing phases.

Analysts make the determination of when each of these inputs should be active based upon the timing plans developed and the configuration of the local controller hardware. Note that the analyst must fully understand the characteristics of the local controller equipment before implementing appropriate timing plans. One of the most variable functions is the reference point in the controller's sequence of displays (7). One should note that yield and force-off points are often automatically calculated by the controller, and no computations are required; however, more experienced traffic signal personnel can calculate and manually enter them in the controller.

The FHWA report (13) gives the following steps for calculating yield and force-off points:

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

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

where:

g_i = effective green for phase i, in seconds;
X_i = volume-to-capacity ratio of critical phase i (from PASSER output);
G_i = optimum duration of phase i, in seconds (from PASSER output);
l_i = sum of lost time components during phase i, ($l_i = l_1 + l_2$), in seconds;
VE_i = vehicle extension for phase i; and
$(Y\text{-}l_2)_i$ = clearance minus the end lost time for phase i;

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:

g_a = effective green for phase (A);
C = optimum cycle length, in seconds (from PASSER output);
l_T = total lost time, in seconds; $l_T = l_a + l_b + l_c$.
g_b = effective green for phase (B); and
g_c = effective green for phase (C).

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

$$v/c_a = (G_a - l_a)/g_a \quad < 0.90$$

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

$$g_a = (G_a - l_a) * X_a/0.90$$

Then reduce the effective green times for Phases B and C using the following equation (7):

$$g_i = [C - g_a - l_T] * [Q_i/(Q_b + Q_c)] \qquad i = b, c$$

where:
- g_i = revised effective green for phase i, in seconds;
- C = cycle length, in seconds;
- g_a = effective green for phase (A), in seconds ;
- l_T = total lost time, in seconds; $l_T = l_a + l_b + l_c$
- Q_i = critical flow rate during phase i, in seconds;

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

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

where:
- $G_i(Avg)$ = average duration of phase i, in seconds;
- g_i = effective green for phase i, in seconds; and
- l_i = lost time of phase i, in seconds;

5. To determine the optimum offset relationship between the two intersections at 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 left-side offset to yield point (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, one can establish the offset to yield point 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 does not equal the offset reported by PASSER III. The offset reported by PASSER III is defined as the timed 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 analyst can use PASSER III's optimum offset and actual green splits to determine the yield point for the right-side intersection relative to the system reference (and thus, the left-side intersection). The calculation of the right-side offset depends 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 on right-side intersection, in seconds;
O_l	=	offset to yield point on left-side intersection, in seconds;
O_p	=	optimum offset from PASSER III output in Step 1, in seconds; and
G_i	=	optimum duration of phase i for left or right intersection (from PASSER III output), in seconds.

One should note that all offsets are in terms of one cycle. 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) equals 140 seconds and the cycle length equals 100 seconds, then the actual relative offset equals 40 seconds (i.e., O_r = 140 - 100 = 40 seconds).

8. At this point, one can determine the force-off points using the optimum phase durations, G (from PASSER III) and the yield points established in the preceding task. The analyst calculates, the force-off points 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. Figure 6-8 illustrates the results of calculations for yield points and force-off points for an example interchange.

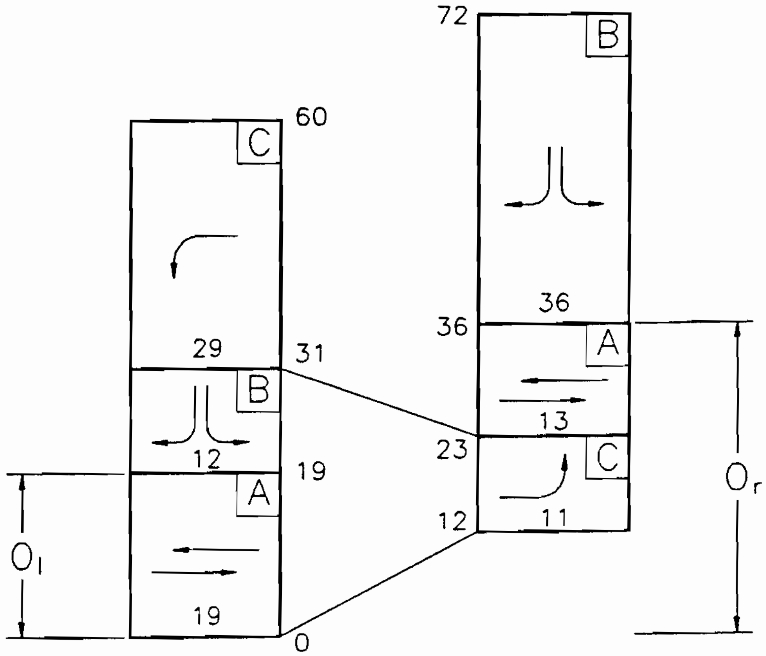
6.8 Fine Tuning

The following discussion follows suggestions and guidelines presented by Yauch and Gray (14). The final step in the implementation phase of retiming signals is fine tuning the signal timing plan. Fine tuning involves observing the signal timing plan in operation after its installation in the controller and determining if the new plan is operating effectively. Based on observations, minor adjustments may be needed to improve the performance of the timing plan in the real world setting. Most adjustments will be made to the phase lengths or offsets.

Results from signal timing optimization computer runs should not be considered absolute or completely correct. Input data may not reflect the real world situation. While signal optimization software are tools to help produce a good timing plan, engineering judgment and field observation must also be part of the implementation process.

Fine Tuning In-house. Other reasons for field observation and fine tuning are that scaled measurements may have been used for distances or data may have been entered incorrectly into the controller. One should perform some fine tuning before actual field implementation. This step involves verifying input data into computer programs and verifying that output results are reasonable. Analysts should also review the transposed data from the computer output to controller settings for accuracy. If these steps are taken before field implementation, adjustments in the field should prove minor.

Fine Tuning in the Field. Fine tuning signal plans in the field involves the verification of plan implementation cycle length, phase splits, and offsets. Field fine tuning also involves determining the effects of the new timing plan on traffic flow. Before determining the operational effects, controller settings should be verified. One should allow the traffic to "settle" before making observations and changes to the controller as part of the field fine tuning process. Drivers may react hesitantly or erratically due to the change in signal timing and/or phasing. The true effect of how the new control strategy affects traffic flow may not be apparent immediately due to driver behavior. Therefore, one should not make observations and measurements until drivers become familiar with the new changes.



Left-Side Intersection (O_l = 19 seconds, given)

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

Right-Side Intersection O_r = 12 (from second PASSER run)

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

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

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

Figure 6-8. Calculation for Yield Points and Force-offs for the Example Interchange

Cycle Length and Phase Splits. For pretimed controllers, analysts may use a stopwatch to time the individual splits and the cycle length to verify controller pin settings. To verify actuated controller settings, VEH DET or MAX RECALL functions should be locked on. This setting will force all phases to time the maximum, set on MAXI or MAXII for that phase. After verifying the settings in the field, the functions should be locked "off." Otherwise, the maximum time will be assigned to each phase whether needed or not, and most benefits from the actuated controller will be lost.

One should note that visual confirmation of the phase splits and cycle length proves extremely important, especially for actuated controllers. Check to see that phases do not max out on a regular basis, and that the resultant cycle length does not become too long. One can determine the efficient or inefficient operation of a diamond interchange signal timing plan very quickly during peak hour operation.

Offsets. As discussed previously, assuming that the arterial phase A is the sync phase, the right side offset (O_r) is defined as the start of Phase A on the left side to the end of Phase A on the right side. For isolated diamond interchanges, the left offset (O_l) equals the duration of Phase A on the left side. One may also measure these phase times with a stopwatch and verify them.

Since the analyst may have estimated the travel times from the left-side (right-side) to the right-side (left-side) of the interchange by the distance between intersections, estimated travel times may differ from those in the field. In some cases, therefore, the time at which Phase C begins on either side of the interchange may be affected. Some adjustment may prove necessary to prevent excessive stopping in the interior of the interchange so that green time is not wasted. Starting the interior through green a few seconds before the platoon's arrival can also provide a safety benefit by separating the conflicting traffic movements by a few additional seconds.

Fine tuning timing plans in the field can have a significant effect on the performance of the signal timing plan. Minor changes, such as a two second increase for a phase, will result in 60 additional vehicles per phase discharging at the approach. This process should be followed for each timing plan implemented at the diamond interchange. **Engineering judgment, in combination with signal timing tools and public feed-back, is the key to developing a good retiming plan.**

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7.0 PROJECT DOCUMENTATION

Assessing the benefits of the new signal timing plan is an important and final step in the signal retiming process. The following sections use two types of documentation. First, traffic signal analysts are interested in the benefits obtained from implementing a new timing plan. Traffic control improvement plans require justification to decision makers before expenditures are allocated. Verifying estimated improvements (i.e., benefits) attributable to signal retiming assists the analyst in future fund allocations for projects. Second, traffic signal analysts are interested in documenting any decisions pertinent to the signal timing process for future reference.

7.1 Estimation of Benefits

To document the benefits of a new timing plan, analysts often use before and after studies. Measures of effectiveness, such as delay, stops, fuel consumption, queues, and v/c ratios, are used as a basis of comparison. One should first establish the objectives or goals of the project before undertaking the project. Some objectives may include:

1. Improve safety at the interchange;
2. Reduce system delay at the interchange;
3. Improve air quality;
4. Reduce fuel consumption; and
5. Increase interchange operational efficiency.

From some combination of these or other goals, measures of effectiveness are chosen for use in the before and after analysis. PASSER III can be used to estimate chosen measures for both the before (existing) and after (optimized) conditions. The differences in the before and after conditions form the benefits of the new signal timing plan. Because PASSER III's analysis period is one hour, one should multiply the benefits by unit costs and then convert them to daily and annual totals for the life of the project.

It is important to remember that when estimating benefits, one should use actual traffic volumes rather than the adjusted traffic volumes used to determine optimum signal timings during the peak hour; i.e., one should attribute benefits to the actual number of vehicles at the interchange. It also is desirable to have field data from the before and after conditions that verify the magnitude of the estimated benefits.

It is important to note that analysts can use PASSER III to estimate benefits for both pretimed and actuated control. In both cases, the benefits attributable to signal retiming are the difference between the before and after conditions. Actuated control, however, will result in better operation than that predicted by PASSER III as long as the volume to capacity ratios at the interchange are less than 0.95. The improvement due to actuated control is an approximate 15 percent reduction in individual MOEs when v/c ratios are less than 0.85 and a correspondingly lesser reduction as v/c ratios approach capacity.

Example Calculations. In the example problems discussed in the previous sections, analysts used PASSER III to evaluate existing conditions at the diamond interchange and then to produce an optimized timing plan. PASSER III reports the following measurements of effectiveness: v/c ratio, delay, storage ratio, and total interchange delay.

Assuming that the objectives of the retiming project were to reduce delay and increase interchange capacity, the following example calculations show the before and after conditions at the diamond interchange as seen in Table 7-1. The difference in the delay for the before (existing) interchange conditions and the after (optimized) interchange conditions is:

Reduction in delay = 64.12 - 38.99 = 25.13 veh-hrs/hr

Typically, benefits for retiming signals range from five to forty percent reductions in delay, stops, and fuel consumption, depending on the type of retiming strategy used (16). Generally, optimization of green splits or cycle length optimization will produce improvements of around five percent, while geometric and signal hardware improvements may show as much as a forty percent overall improvement. The percentage of improvement also depends on how bad the signal timing plan was before it was optimized.

To estimate the total benefits of an optimized signal timing plan, multiply the delay reduction (or other improvements reported by an analysis tool such as PASSER III) by the number of hours a timing plan is in operation. If three timing plans are used in a day, typically the a.m. and p.m. peak timing plans will be used for one to two hours each, and the off-peak timing plan will be used for ten to twelve hours for benefit analysis, i.e. twelve to fourteen hours of the day are used for benefit analysis. The following steps show how benefits may be calculated per day, per year, and for the life of the project:

1. **Compute Hourly Benefits.** For each timing plan, one calculates the improvement in measures of effectiveness, such as stops, delay, and fuel consumption. For example, the delay due to signalization for the optimized (after) timing plan is subtracted from the delay due to signalization for the existing (before) timing plan.

2. **Compute Benefits for Each Timing Plan.** For each timing plan, multiply the savings (stops, delay, or fuel consumption) by the number of hours that the timing plan is in operation. As discussed previously, one may multiply the a.m. peak reduction by 2 hours, the p.m. reduction by 2 hours, and the off peak reduction by 10 hours.

Table 7-1 Comparison of Before and After Measures of Effectiveness

		LEFT SIDE		RIGHT SIDE	
Measures of Effectiveness	**Phase**	**Before**	**After**	**Before**	**After**
V/C ratio	Arterial	1.14	.58	.76	.84
	Frontage Road	.75	.59	.97	.84
	Left Turn	.29	.59	.69	.84
	Interior Through	.19	.21	.35	.40
Delay (sec/veh)	Arterial	156.91	21.56	43.12	50.38
	Frontage Road	59.18	36.13	166.94	96.55
	Left Turn	.36	.00	3.91	22.98
	Interior Through	.05	5.42	.00	6.64
Total Interchange Delay (veh-hrs/hr)	Entire Interchange	64.12	38.99		

3. **Compute Daily Benefits.** Next, sum the reductions (stops, delay, and fuel consumption) for each timing plan; (a.m. delay reduction * 2) + (p.m. delay reduction * 2) + (off peak reduction * 10). This sum equals the total reduction for each measure of effectiveness in stops per day for stops, vehicle-hours per day for delay, and gallons per day for fuel consumption.

4. **Compute Annual Benefits.** To estimate the annual benefit, the reductions per day are multiplied by 300 days per year (not counting weekends). Express the yearly reductions in stops per year for stops, vehicle-hours per year for delay, and gallons per year for fuel consumption.

5. **Compute Benefits for Life of Project.** Typically the life of a signal timing plan is three to five years. To estimate the benefit of reductions over the life of a project, multiply the yearly reductions (stops, delay, and fuel consumption) by the life of the project. To allocate a dollar amount to the savings due to delay reductions, select a cost from a reference such as the AASHTO Manual on User Benefit Analysis of Highway and Bus-Transit Improvements (17) per stop, per vehicle-hour of delay, and per gallon of fuel.

Although fuel consumption reduction is normally associated with a series of signals, reduction of delay at an isolated interchange or intersection would also reduce fuel consumption, however, PASSER III does not report fuel consumption for an isolated interchange.

Some cities have published information regarding the benefits of signal retiming to motorists. This information allows local citizens and public officials to recognize the benefits gained through traffic signal retiming projects. A previous study conducted on signal retiming in 44 Texas cities (2,243 signals retimed) resulted in annual reductions in fuel consumption, delay, and stops of 9.1 percent (30 million gallons), 24.6 percent (43 million hours), and 14.2 percent (1.7 billion stops) (18). It is important to reiterate that signal retiming benefit citizens directly by reducing fuel consumption, delay time, and the number of stops at a signalized intersection.

7.2. Benefit-Cost Analysis

Other considerations in determining benefits from a new timing plan involve the cost of preparing and implementing the new timing plan. Costs may be estimated by person-hours used to collect data and prepare data for analysis, computer costs, and person-hours needed to implement the timing plan in the field. An example of an analyst's cost estimate may look like the example in Table 7-2.

For example, say a district has 450 signals and a total budget for the signal section of $1,387,000, such as that shown in Figure 7-1. If $160,000 of the total budget is used primarily for signal timing, this expenditure would equal $356/signal per year, or each signal could be retimed for $1067/signal every three years. One can see that any significant reduction in delay and fuel consumption would easily pay for the cost of retiming. The costs of data collection and field implementation must also be considered.

Some estimates of retiming costs given by various agencies range from $500 to $1800 per intersection (15). Another estimate figured one person - week for retiming a signal, which corresponds to one person timing 50 signals in a year; of course, several persons work on one project at a time. These estimates include data collection and development of timing plans. Costs will be higher for geometric improvements or major signal hardware replacement.

After benefits and costs of the signal retiming project have been computed, it is a simple matter to calculate a benefit-cost ratio for the project. Typical ranges from past projects from $20 to $100 dollars in motorist benefits for every dollar spent on the signal retiming projects.

7.3 Documentation of Decisions

As in all other aspects of engineering and TxDOT projects, liability is an important concern. Analysts should document the final signal timing plan agreed upon for implementation. This documentation includes steps taken toward developing the timing plan. Documentation of tasks performed and decisions made concerning signal retiming should be made. Documentation should include pedestrian considerations, clearance time calculations, left turn phasing, etc. One should record any unusual design procedures or engineering judgement decisions.

Documentation is recommended when implementing and fine tuning timing plans, including traffic control and safety procedures taken to protect the traveling public. It is recommended that one copy of the signal timing plans currently in operation be kept in the controller and at least one copy of the plans be kept in the office or project files. It also is recommended that two copies of the signal's maintenance records be kept, as these records are becoming increasingly important in tort liability cases. As with signal timing plans, one copy of the maintenance records should be kept in the controller and the second copy should be kept in the office.

Table 7-2 Analyst's Cost Estimate for a Typical Retiming Project

Cost Item	Level/Type	Time	Cost	Comments
Personal	Engineer	20 hrs	$715.60	$37.78 per hour
	Oprtns. Supt	32 hrs	$743.68	$23.24 per hour
		40 hrs	$990.00	$24.75 per hour
	Traffic Tech.	32 hrs	$427.52	$13.36 per hour
		115 hrs	$1,656.00	$14.40 per hour
	Total		$4532.80	Hourly rates include salary plus 30% overhead and fringe benefit allowance
Expenses	Equipment		$33,000.00	6 Eagle EPAC 300 Controllers
	Vehicle	90 hrs	$585.00	Bucket Truck
	Training		$444.00	PASSER III Training
Total Local Costs			$34,029.00	
Consulting	Timing Plans		$7,250.00	
	Install Controllers		$15,000.00	
	Total		$22,250.00	
Total Project Cost			$56,279.00	

Salaries and Fringe Benefits	
Signal Engineering - $266,667 x 60% =	$160,000
Signal Shop - $900,000 x 60% =	$540,000
Overtime and Standby Pay for Signal Maintenance	$ 32,000
Motor Pool Charges for Signal Surveillance and Maintenance Vehicles	$120,000
Supplies	$ 25,000
Repairs of Equipment by Vendors (including Maintenance of Central Computer Equipment)	$ 15,000
Signal Parts and Components for Maintenance	
Funded from Operating Budget	$170,000
Capital Improvements Funds (knockdowns, replacement of controllers and detectors) estimated	$325,000
TOTAL	$1,387,000

Figure 7-1. Example District's Budget for a Signal Section

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