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16. Abstract

This document provides guidelines and procedures for the retiming of isolated intersections, 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 retiming project.

Section One, Introduction, provides background information and discusses how to determine if a signal should be retimed. Section Two, Isolated Intersections, discusses the characteristics of an isolated intersection, 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 an isolated intersection.

Section Four, Evaluation, provides information needed for the simulation of existing intersection operations, including the use of currently available software programs. Section Five, Optimization, provides the guidelines to optimize timings at a typical isolated intersection. Section Six, Implementation, explains the interpretation of optimized 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

ISOLATED INTERSECTIONS

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Texas Transportation Institute
The Texas A&M University System
College Station, Texas 77843-3135

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.

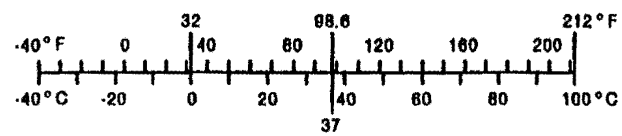
Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

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 retiming projects addressed in 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.

This document provides implementation guidelines and procedures for retiming signalized isolated intersections.

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.

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1.0 INTRODUCTION

1.1 Background

With urban congestion increasing and available funding decreasing in Texas cities, Texas Department of Transportation (TxDOT) personnel face a growing problem of developing low-cost solutions to increase 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 a cost-effective and transportation systems management (TSM) measure. Results from several studies demonstrate that substantial energy savings can be achieved 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 to minimize delays and stops and improve traffic progression.

Because of the diversity of retiming project types and the number of techniques and tools available, however, there exists no single procedure or set of guidelines that is applicable 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, a set of guidelines and procedures for several types of typical traffic signal retiming projects would prove beneficial 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 include the types and amounts of data to be collected, the procedures for doing so, the analytic procedures and software packages that are available, the types of projects for which they are suited, 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 in this study 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 individual guidelines and procedures for retiming isolated signalized intersections. It includes the procedures for data collection, the types and amounts of data to be collected, and the analytical procedures and software packages that are available for traffic signal retiming projects.

1.3 Organization

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

1.0 Introduction
- 1.1 Background
- 1.2 Objectives
- 1.3 Organization
- 1.4 When to Retime Signals at Isolated Intersections

2.0 Isolated Intersections
- 2.1 Definition
- 2.2 Intersection Phasing
- 2.3 Types of Control
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7.0 Project Documentation
- 7.1 Estimation of Benefits
- 7.2 Benefit-Cost Analysis
- 7.3 Documentation of Decisions

8.0 References

1.4 When to Retime Signals at Isolated Intersections

Public complaints are usually the first signs of signal operational problems. While traffic signal analysts cannot address all complaints, the number of complaints indicates a need for at least a field observation and/or possibly an engineering study. Some common complaints include: excessive approach delay, left turn delay, and excessive queues. Field observations can be made 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 than 150 seconds). In some cases, equipment such as detectors may need repair. If one rules out these problems, retiming may be a solution to improving the signal's operational efficiency. As a rule of thumb, field observations or studies should be made 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, changes in major traffic generators and changes in the geometrics of the roadway or intersection also may create the need for retiming signals. Some jurisdictions recommend yearly inspection and documentation, by field data and/or video, of their traffic signal operations. This type of documentation will help identify operational problems before they become severe.

2.0 ISOLATED INTERSECTIONS

2.1 Definition

A traffic signal is installed at an intersection to provide for the safe, orderly, and efficient movement of traffic (1). An intersection is considered an isolated intersection when it operates independently of adjacent signals; i.e., its timing remains independent of other intersections. Because few benefits arise from coordination when signal spacings are long, a signal located a mile or more from another signal generally should be operated as an isolated intersection.

Operational problems that may exist at an isolated intersection include: excessive approach delay, excessive left-turn delay, and excessive queues. Excessive delays could result from the backup of long queues of left-turn vehicles into the through lanes or the blocking of the left-turn lane by long queues of through traffic. Such problems usually arise due to increased or changing travel demand or poor signal timing. The following sections discuss characteristics and operational considerations of isolated intersections.

2.2 Intersection Phasing

The phasing at an intersection can be divided into three parts: movement numbers, type of left-turn treatment, and phasing sequence. The following paragraphs describe each of these components.

Movement Numbers. Most methods of signal timing analysis use the NEMA configuration for numbering movements as shown in Figure 2-1. Each NEMA movement corresponds to a phase in an 8-phase NEMA controller. Combinations of these movements also are considered phases. To number the movements, start with Movement 1, (always a left-turn movement) and move clockwise numbering each left-turn 3, 5, and 7. Movement 2 (always a through movement) is always opposite from Movement 1. Proceed clockwise to number the remaining through Movements 4, 6, and 8.

Note that Movement 1 conflicts with Movement 2, Movement 3 conflicts with Movement 4, Movement 5 conflicts with Movement 6, and Movement 7 conflicts with Movement 8. These movement pairs, known as conflicting movements, cannot have a green indication at the same time. This general rule serves as the basis for the signal timing methodologies and hardware described in this report.

Left-turn Treatment. Phasing schemes can be classified by the type of left-turn treatment that exists at the intersection. Left-turn movements can be protected only, permitted only, or protected-permitted (combined), as illustrated in Figure 2-2 and described on the following page:

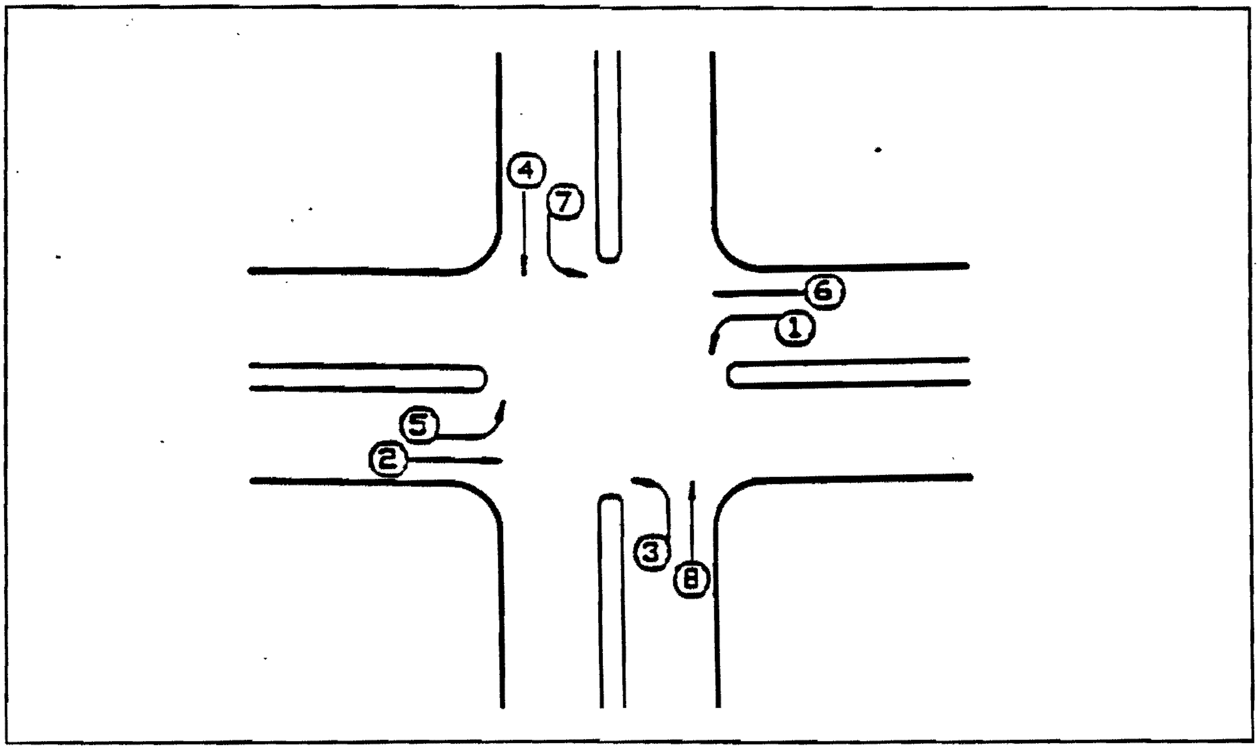


Figure 2-1. NEMA Configuration for Numbering Phase Movements

Geometry	Permitted Phasing	Protected Phasing	Combined Phasing
No Bay	1	2	3
Bay	4	5	6

Phasing

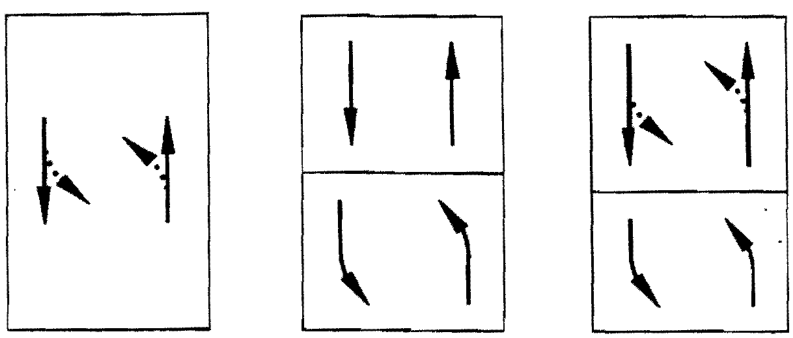


Figure 2-2. Left-Turn Treatment

Permitted Only - Left turns may proceed on a green ball after yielding to oncoming traffic (phases 1,3,5,and 7 do not exist).

Protected Only - Left turns may proceed only during a green left-turn arrow.

Combined - Left turns may proceed during the protected phase and on a green ball after yielding to oncoming traffic.

The type of left-turn treatment provided at an intersection depends upon a number of factors, like left-turn volume, opposing through volume, available sight distance, and the number of opposing through lanes. Figure 2-3 illustrates an example of one set of guidelines for the selection of the appropriate left-turn treatment to implement at a signalized intersection (2).

Phase Sequence. The phase sequence represents the order in which the phases are displayed at a signalized intersection. Timing plans or field observation can provide the phasing information for existing conditions. Phase sequences are often described by the order in which left turns occur. Left-turn phases may be leading, lagging, lag-lead or lead-lag for the main street and/or cross street. It is not unusual, however, for the phasing sequence on the main street to be different from the phasing sequence on the cross street.

Leading Lefts - Both left-turn movements proceed before the through movements.

Lagging Lefts - Both left-turn movements proceed after the through movements.

Lag-lead/lead-lag - One left-turn movement and its adjacent through movement proceed before or after the opposing left-turn and its adjacent through movement.

If the duration of the phases serving the two concurrent movements equal one another (i.e., the concurrent phases start and end at the same time), the phasing is described as "without overlap" phasing. If the duration of the phases serving the two concurrent movements do not equal one another (i.e.,the concurrent phases start at the same time, but end at a different time), the phasing is described as "with overlap" phasing. These two phasing sequence descriptors are analogous to single and dual ring control, respectively. It should be noted that the terminology for describing phase sequences may vary between signal timing software and traffic controller hardware. Figure 2-4 illustrates one set of guidelines for selecting the type of left-turn phasing sequence to implement at a signalized intersection.

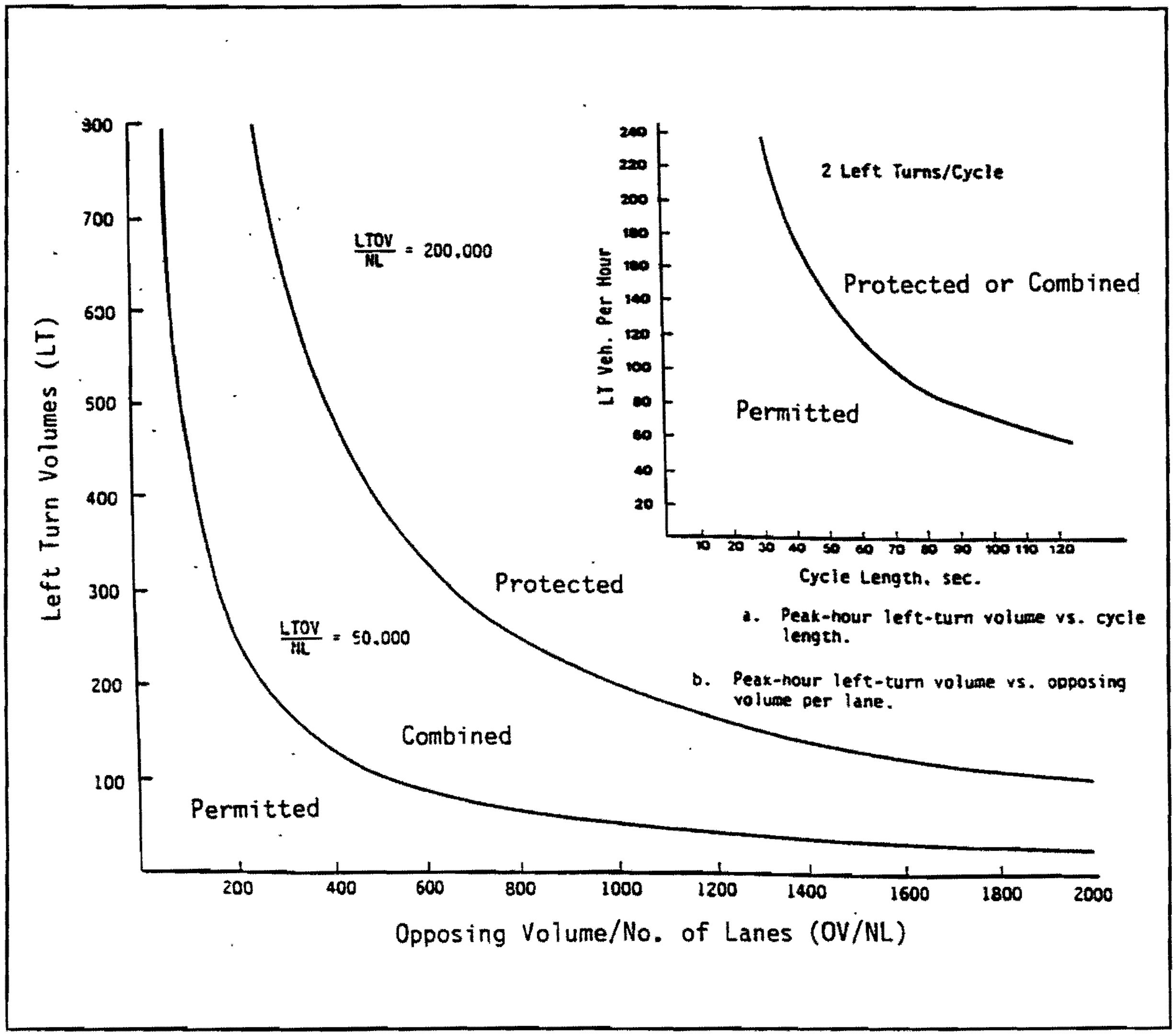
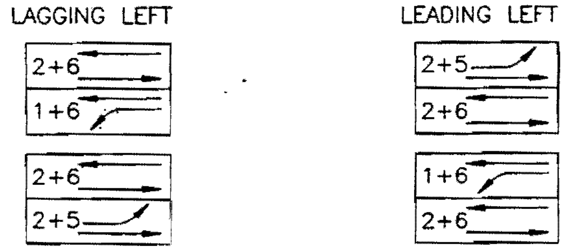


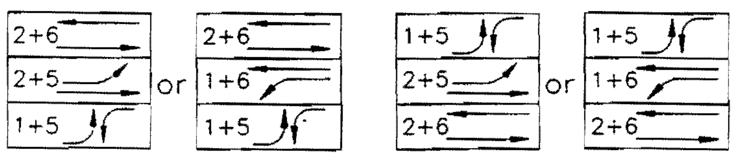
Figure 2-3. Left-Turn Phasing Guidelines Based on Traffic Volumes

LEFT-TURN PHASING	GUIDELINES

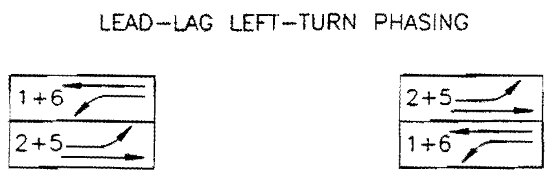


SPECIAL PHASING
* Applicable for T-intersections or where opposing left-turns are prohibited.
* Can be used with either pretimed or actuated control.
* Use should be kept to a minimum where no left turn lane exists due to possibility of one movement blocking the other.

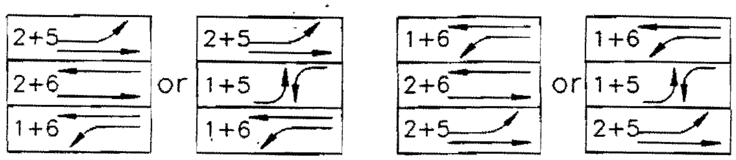
NO OVERLAP PHASING
* Simple and inexpensive.
* Can be provided with pretimed or single ring actuated controllers.
* Should be kept to minimum since both left-turns receive exactly the same green time.
* Lagging left should only be used when system operation needs dominate safety consideration.



OVERLAP PHASING
* Applicable for dual ring actuated controllers.
* An efficient method of providing protected only left-turn phase.
* Through movement is allowed to move when there is no opposing left-turn traffic.



SPLIT PHASING (NO OVERLAP)
* Applicable when left-turn volume is very heavy in both directions and is equal or greater than the companion through movement.
* Effective when one of the through lanes must be used as an optional left-turn lane or where a separate left-turn lane can not be provided.



OVERLAP PHASING
* Either pretimed or dual ring actuated controller can be used.
* As efficient as dual left with overlap phasing.
* Should be used only for interconnected signals where its use will increase the width of the progressive band.

Figure 2-4. Guidelines for Selecting Left-Turn Phasing Sequences

2.3 Types of Control

Isolated intersections can be controlled by one of three methods: pretimed, semi-actuated, or full actuated control. The type of controller at an intersection affects the number of timing plans that can be implemented, as well as the number of phases and sequences that are possible. This document addresses pretimed and fully actuated control strategies because they represent the majority of field installations.

Pretimed Control. Pretimed control strategies are typically used when a limited number of traffic patterns exist at an intersection and there is no significant deviation from those patterns. Pretimed control assigns right of way in a predetermined manner. The major elements of pretimed control are fixed cycle length, fixed phase sequence, and fixed phase length. Depending on the equipment, several timing plans may be used with the appropriate timing plan being implemented automatically at fixed times of the day. Usually, there is one timing plan for the a.m. peak period, one timing plan for the p.m. period, and one timing plan for the off-peak period. Pretimed control is usually not recommended for isolated intersections except in the following cases:

1. Where traffic at the intersection remains constant and predictable;

2. When no more than three phases are required; and

3. Where expertise is unavailable for properly operating, diagnosing, or maintaining actuated equipment.

Basically two types of pretimed controllers exist: electromechanical and solid state. One or more dials (usually no more than three dials) driven by a synchronous motor comprise the electromechanical controller. Each dial corresponds to a different timing plan; for example, there is one dial for the a.m. period, one dial for the p.m. period, and one dial for the off-peak period. A solid state controller is similar to an electromechanical controller except the mechanical parts (the dial units, camshafts, and keys) are replaced by solid state components and operations are controlled by a microprocessor. For more details about the hardware, refer to the *Traffic Control Systems Handbook* (3). It should be noted, however, that the Texas Department of Transportation (TxDOT) no longer purchases pretimed controllers. Rather, they purchase full actuated controllers and then use them as pretimed controllers when the conditions warrant.

Actuated Control. Actuated control is suited to isolated intersections when traffic patterns are unpredictable and demand varies. Traffic actuated control attempts to adjust green times, and in some cases, skip phases to provide the green time where vehicular demand warrants. Detectors placed in the approach lanes provide demand information to the controller. The basic timing parameters are yellow plus red clearance times, minimum green times, green extension interval, and maximum green interval. Definitions for these basic timing parameters follow:

Yellow plus red clearance time - The portion of time that occurs at the end of the phase and provides adequate time for all vehicles to safely clear the intersection. This parameter is based upon vehicle speed, the width of the intersection, and driver expectancy.

Minimum green time - The length of time considered to be the shortest amount of time that a phase is allowed to be green. This parameter is usually based upon pedestrian walk time or the location of the detector. The actual green time cannot be less than the "minimum green" time.

Extension interval - The portion of time that the green interval can be extended is based on detector location. A minimum extension time should allow a vehicle sufficient time to travel from the point of detection to the stop line.

Maximum green interval - The maximum green interval is the longest time a green indication will be displayed in the presence of a call on a conflicting phase.

Two basic hardware designs exist for actuated controllers; the Type 170 and the NEMA standard. The states of California and New York jointly developed Type 170 controllers. These controllers require software to operate; and, changes in traffic conditions or control strategies can be accommodated by revisions to the software. NEMA (National Electrical Manufacturers Association) controller's specifications meet standards reflecting input from traffic engineers, installers of traffic signal equipment, and professionals in the field of traffic control. The NEMA specifications describe physical and functional requirements for fully actuated signal controllers. For more information on these controllers, refer to the *Traffic Control Systems Handbook* (3). It should be noted that TxDOT only purchases actuated controllers conforming to NEMA standards, and that TxDOT has developed a set of standard specifications for its controllers.

Actuated controllers are generally classified as single-ring or dual-ring controllers. Each controller provides for different combinations of phase sequences. The single-ring controller is a four-phase controller which handles phases sequentially and returns to the first phase at the end of the series. A specified sequence is used; and, phases cannot occur simultaneously (overlap cannot be used). Figure 2-5 illustrates the single-ring controller. The dual-ring controller is an eight-phase controller with phases corresponding to individual movements at the intersection. Concurrent timing can be used with a dual-ring controller and allows a variety of non-conflicting phase selections. Phases can be skipped and overlap phasing can be used. Figure 2-6 illustrates the dual-ring controller.

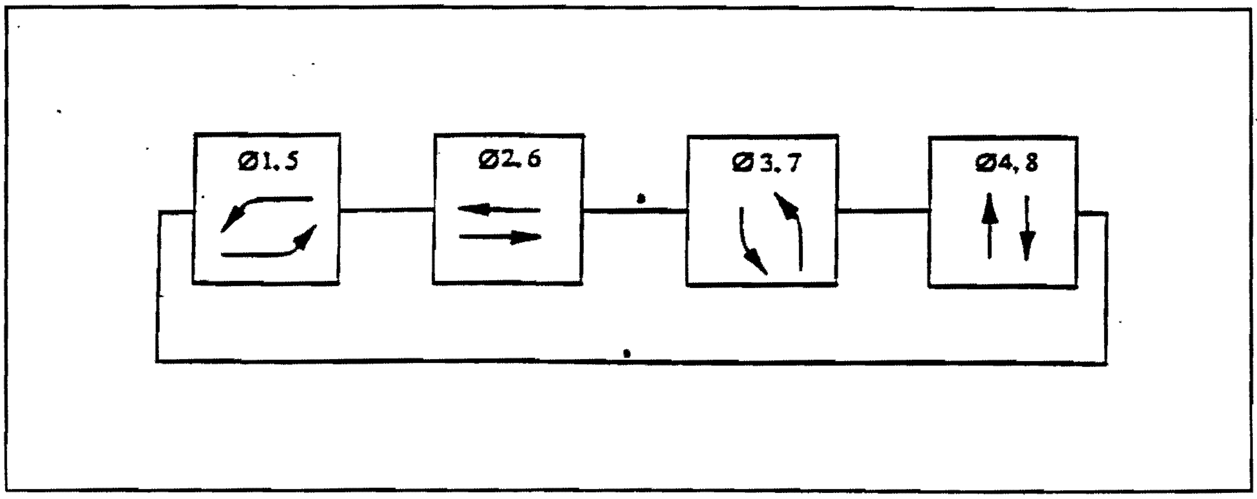


Figure 2-5. Single-Ring Controller (Sequential)

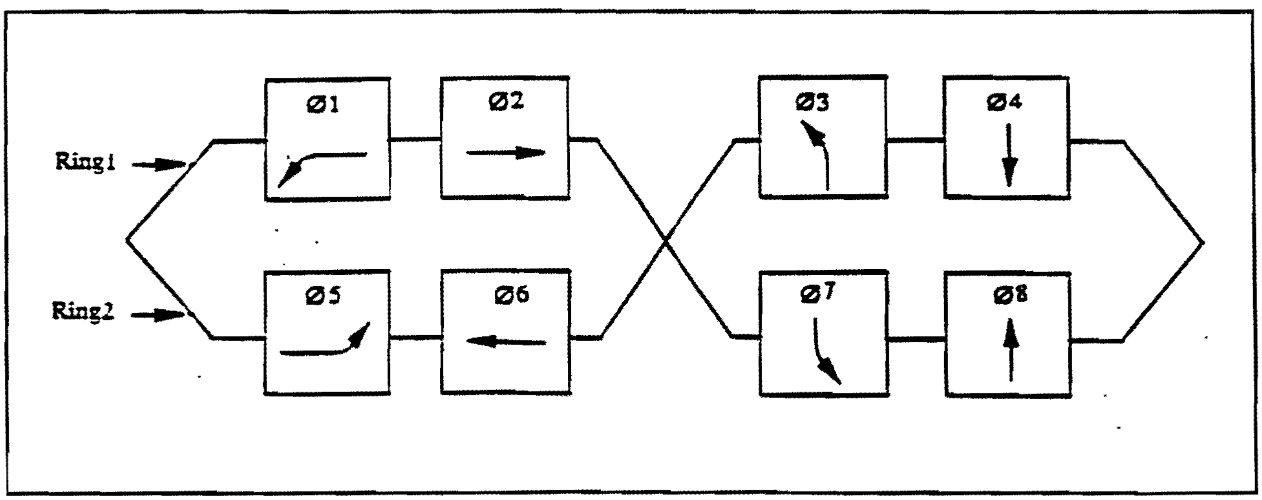


Figure 2-6. Dual-Ring Controller (Concurrent)

The type of control selected may affect intersection operation. At low to moderate volume to capacity ratios, actuated controllers are almost always more efficient, because of the capability of adjusting green times and/or skipping phases in response to variations in vehicular demands. In fact, even at conditions approaching capacity, there is still enough variation in vehicular demands, particularly relative to left-turn phases, that actuated controllers can be more efficient than pretimed controllers. In addition, actuated controllers are more responsive to high volume conditions because their maximum cycle lengths are generally longer than pretimed controller cycle lengths.

2.4 Signal Timing Methods

Signal timing attempts to find a "best" solution characterized by an appropriate cycle length and green splits. The best solution may be one that minimizes delay, fuel consumption, excessive queues, or increases the capacity of an intersection. Typically, the ideal saturation flow rate per lane is 1800 vehicles per hour of green time (4); however, this rate can vary between 1600 and 2000 vehicles per hour of green depending on local conditions. Thus, one should realize that flow rates measured in the field should always be considered more accurate than estimated values.

When two approach lanes cross each other, the combined flow rate of the two lanes (3600 vehicles per hour of green) falls to 1800 vehicles per hour or less at the intersection area. As a result, green time has to be allocated separately to each movement in an optimum manner. This solution depends on the characteristics of the intersection and the objective of the traffic signal analyst.

Webster's equations for calculating delay, minimum delay cycle length, and green splits (5) provide the basis for one often-used signal timing method. Webster's signal timing method starts with the determination of the intersection's critical movements and flow ratios and continues with a determination of the minimum delay cycle length and green splits for each movement at the intersection.

Determination of the Critical Movements. The critical movements at an intersection should be identified in order to determine the green splits for the various phases. One should remember, however, that the critical movements for an intersection can vary for different phasing sequences. Table 2-1 illustrates a process for identifying the critical movements on both the arterial and cross streets.

Table 2-1. Critical Phase Movements for Different Signal Phasings[a]

Signal Phasing	Arterial Movements[b]	Cross Street Movements[b]
Two phase with no left turn bay (Throughs first without overlap)	Max (1 + 6) or (2 + 5)	Max (3 + 8) or (4 + 7)
Two phase with left turn bay (Throughs first without overlap)	Max (1, 2, 5, or 6)	Max (3, 4, 7, or 8)
Four phases (without overlap)		
Dual left leading	Max (1 or 5) + Max (2 or 6)	Max (3 or 7) + Max (4 or 8)
Dual left lagging	Max (2 or 6) + Max (1 or 5)	Max (4 or 8) + Max (3 or 7)
Leading left	Max (1 or 6) + Max (2 or 5)	Max (3 or 8) + Max (4 or 7)
Lagging left	Max (2 or 5) + Max (1 or 6)	Max (4 or 7) + Max (3 or 8)
Four Phases (with overlap)		
Dual left leading	Max (1 + 2) and (5 + 6)	Max (3 + 4) and (7 + 8)
Dual left lagging	Max (1 + 2) and (5 + 6)	Max (3 + 4) and (7 + 8)
Leading left	Max (1 + 2) and (5 + 6)	Max (3 + 4) and (7 + 8)
Lagging left	Max (1 + 2) and (5 + 6)	Max (3 + 4) and (7 + 8)

[a] Critical movement volumes are expressed on a per lane basis.
[b] Movement numbers refer to physical traffic movements, not to NEMA phases.

Minimum Delay Cycle. Delay is often used as a measure of effectiveness to determine the efficiency of a signalized intersection's operation. Figure 2-7 illustrates the variation in delay with cycle length, and the equation and location for Webster's minimum and minimum delay cycle lengths. Figure 2-8 illustrates the variation in minimum delay cycle lengths at various volume levels. Note that if traffic volumes fluctuate during the day, an intersection may have a different minimum delay cycle and phasing sequence for different periods during the day. Webster's equation for calculating the minimum delay cycle is as follows:

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

where:
- C_o = minimum delay cycle length (seconds);
- L = total lost time (seconds); and
- ΣY = sum of the critical flow ratios, $y_1 + y_2 + \ldots + y_i$ (y_i = volume for critical movement i divided by the saturation flow rate for critical movement i).

The range of cycle lengths that generally result in acceptable operation (near minimum delay) at a typical intersection is $0.8\,C_o < C < 1.3\,C_o$.

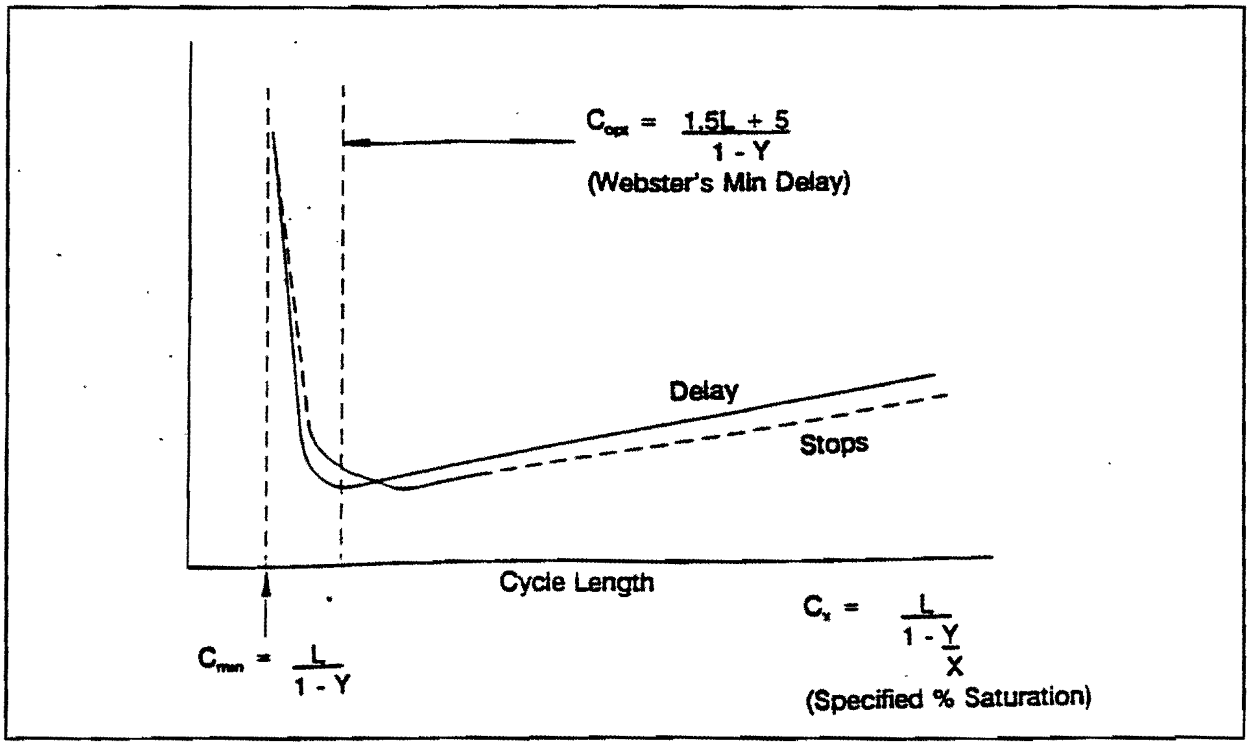


Figure 2-7. Variation in Delay with Cycle Length

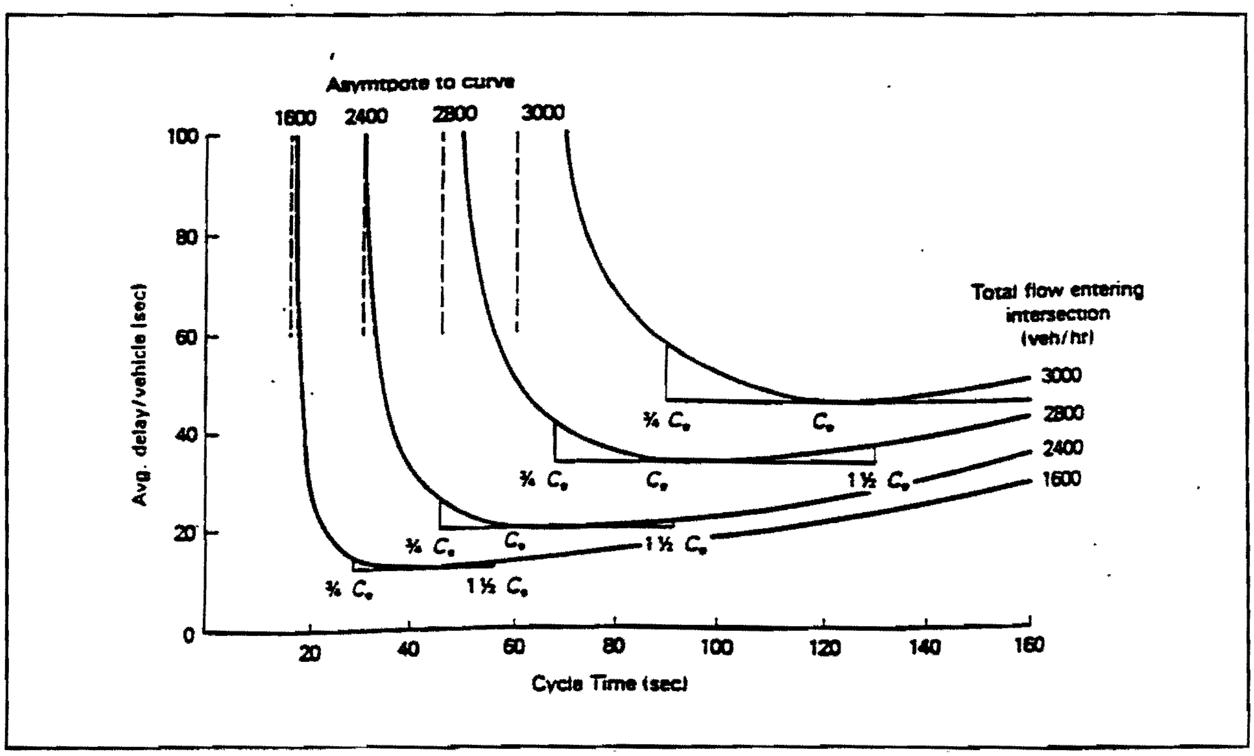


Figure 2-8. Variation in Delay with Cycle Length for Different Volume Levels

One should note that the entire cycle is not available for the actual movement of vehicles. Some of the time is lost as reaction and start-up lost time at the beginning of the green interval, while some of the time is lost during the unused yellow and red clearance time when no vehicles move through the intersection.

Green Splits. After determining the minimum delay cycle, green time must be allocated to each phase. The green time allocated to each phase is the green split for that phase. Webster's method of allocating green time to the various intersection movements is discussed below.

Identifying critical movements and calculating flow ratios help determine the optimum green splits for the intersection. The sum of the critical flow ratios is denoted by the variable Y. The first step in calculating the green splits is to calculate the overall volume to capacity ratio, X_I. Webster's method assumes X_I to be the volume to capacity ratio for each critical movement. The equation for calculating the intersection volume to capacity ratio is as follows:

$$X_I = (Y * C) / (C - L)$$

where:
- X_I = intersection volume to capacity ratio;
- Y = sum of the critical flow ratios;
- C = cycle length (sec); and
- L = total lost time at the intersection (seconds). The lost time per phase generally ranges from 3 to 5 seconds.

The equation for determining the green time for each critical movement is as follows:

$$g_i = (y_i * C) / X_I$$

where:
- g_i = phase time = G + Y + RC - L (sec);
- y_i = flow ratio for movement i;
- C = cycle length (sec); and
- X_I = intersection volume to capacity ratio.

2.5 Measures of Effectiveness

The measures of effectiveness (MOEs) used in the evaluation of signal timing alternatives include volume to capacity ratio, average delay, queue length, total stops, and fuel consumption. Each of these MOEs is discussed below.

Volume to Capacity Ratio (v/c). According to the *Highway Capacity Manual* (4), the volume to capacity ratio represents 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 lane group during the same time interval.

Capacity at an intersection, the denominator, 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. Saturation flow rates and available green time provide the basis for determining capacity at signalized intersections.

The saturation flow rate 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.

The volume to capacity ratio (v/c ratio (X_i)) can therefore be computed as follows:

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

where:
- X_i = volume to capacity ratio of lane group or approach i;
- v_i = volume of lane group or approach i (veh/hr);
- c_i = capacity of lane group or approach i (veh/hr);
- g_i = effective green time for lane group or approach i (seconds);
- C = cycle length (sec); and
- S_i = saturation flow rate for lane group or approach i (vplphg).

Volume to capacity ratios greater than 1.0 indicate over capacity conditions (i.e., more vehicles than capacity); volume to capacity ratios less than 1.0 indicate under capacity conditions. These conditions should be noticeable from field observations; i.e., if there is not enough green time to clear the queue of stopped vehicles, the approach or movement is over capacity.

Delay. Delay is a measure of effectiveness commonly used to estimate the level of service at signalized intersections. Delay at a signalized intersection can either be observed directly in the field or calculated based on traffic, geometric, and signal timing parameters. Equations based on Webster's delay formulations, estimate delay caused by signalized intersections (5).

Two types of delay exist: stopped delay and total delay. Stopped delay is the amount of time a vehicle actually stops and waits for a green indication and/or the queue of vehicles to clear. It is more easily measured in the field than total delay. Total delay represents an estimate of the number of stops and interferences encountered by a vehicle due to the signal. Total delay includes delay due to acceleration and deceleration, reduced speed due to interference from other vehicles, and delay due to stops.

The *Highway Capacity Manual* (4) contains the most widely used model 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. Hence, no vehicles wait for more than one cycle to pass through the intersection. The first part of the equation for stopped delay with uniform arrivals is as follows:

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

Vehicle arrival patterns, however, are not uniform. They are more likely to be random in nature. 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_1 = uniform delay (sec/veh);
- d_2 = incremental delay (sec/veh);
- DF = delay adjustment factor for either quality of progression or control type.
- C = cycle length (seconds);
- g = green time per phase (seconds);
- X = volume to capacity ratio for that phase;
- Min (X,1) = the lesser value of either X (v/c ratio for lane group) or 1.0;
- m = a calibration term representing the effect of arrival type and degree of platooning; and
- c = capacity of lane group, (vehicles/hour).

The intersection stopped delay is as follows:

$$d = d_1 * DF + d_2$$

Total delay is calculated by multiplying stopped delay by a factor of 1.3. The 0.38 constant in the uniform delay equation would be 0.5, and the 173 constant in the incremental delay equation would be 225 for the calculation of total delay. The 1.3 factor is based on field studies of observed delay.

$$D = 1.3 * d$$

where: D = total delay, (sec/veh); and
d = stopped delay, (sec/veh).

Despite the wide acceptance of the equations in the HCM, other methods for calculating delay exist. PASSER II-90, SOAP-84, and TRANSYT-7F are software packages commonly used for the analysis of signalized intersections. These programs are discussed in detail elsewhere in this report; however, it is appropriate to mention that the equations used by these programs estimate delay differently than does the HCM. These differences are primarily the result of progression and oversaturation effects, and the programs generally prove more accurate for those conditions than the HCM equations.

Level-of-Service. Level of service (LOS) indicates a range of operating conditions on a particular type of facility. The 1985 *Highway Capacity Manual* (4) defines the LOS as a qualitative measure describing operational conditions within a traffic stream and their perception by motorists and/or passengers. LOS for signalized intersections is defined in terms of stopped delay per vehicle. Delay represents a measure of driver discomfort, frustration, fuel consumption, and lost travel time. It is a complex measure that depends on a number of variables. Table 2-2 indicates LOS criteria for signalized intersections.

When analyzing an intersection's operation, delay and volume to capacity ratios should be studied simultaneously. Acceptable measures of delay do not necessarily imply acceptable volume to capacity ratios and vice versa. Although volume to capacity ratios affect delay, one measure does not necessarily predict the other. "Acceptable" delay or volume to capacity ratios depend on the lane group or approach being analyzed. A high average delay value may be more acceptable for a minor lane group or approach than for a major or more important movement. For example, it may be acceptable for three or four vehicles on a minor approach to "wait" for 40 seconds, if several hundred vehicles on a major approach only "wait" for 20 seconds.

Table 2-2. Level of Service Criteria

Level of service	Volume to capacity ratios	Average stopped delay (sec/veh)	Average total delay (sec/veh)
A	≤ 0.60	less than 5.0	less than 6.5
B	≤ 0.70	5.1 to 15.0	6.6 to 19.5
C	≤ 0.80	15.1 to 25.0	19.6 to 32.5
D	≤ 0.85	25.1 to 40.0	32.6 to 52.0
E	≤ 1.00	40.1 to 60.0	52.1 to 78.0
F	> 1.00	greater than 60	greater than 78

Queue Length. Queue length is another basic measure of performance. It is of particular importance when limited queue storage space exists. A heavy left-turn demand or a short left-turn bay can cause queues of left-turn vehicles to backup into the through lanes and block them. Similarly, long queues in the through lanes can block the entrance to the left-turn bays. Such problems give rise to excessive delays to traffic and cause driver frustration, which in turn can lead to safety problems.

According to Akcelik (6), the average number of vehicles in the queue at the start of the green period is given by:

$$N = qr + N_o$$

where:
- N = average number of vehicles in queue (veh);
- q = arrival flow rate in vehicles per second (veh/sec);
- r = effective red time in seconds (sec); and
- N_o = average overflow queue in vehicles and given by:

$$N_o = \frac{QT_f}{4}\left(z + \sqrt{z^2 + \frac{12(x-x_o)}{QT_f}}\right)$$

where:
- Q = capacity in vehicles per hour;
- T_f = flow period (usually 15 minutes);
- z = x - 1;
- x = degree of saturation (q/Q);

x_o = 0.67 + sg/600;
s = saturation flow rate (veh/sec); and
g = effective green time (sec).

A theoretical model, which assumes that vehicles join the queue when they reach the stop line, provides the basis for the above equation for estimating queue length; however, since vehicles join the queue prior to reaching the stop line, the equation underestimates the maximum queue length. As a result, the maximum queue length is given by:

$$N_m = \frac{qr}{1 - y} + N_o$$

where: N_m = maximum length of the queue; and
y = flow ratio (volume/saturation flow rate).

Stops. Number of stops is another basic measure of performance from which other (secondary) measures of performance (like fuel consumption) can be obtained. Every vehicle which comes to a complete stop at an intersection experiences a small delay. According to Akcelik (6), the average number of stops per vehicle is called the stop rate and is given by:

$$h = 0.9\left(\frac{1 - u}{1 - y} + \frac{N_o}{qC}\right)$$

where: h = average number of complete stops per vehicle (stop rate);
u = green time ratio (effective green time / cycle length);
y = flow ratio (demand volume /saturation flow rate);
q = arrival flow rate in vehicles per second;
C = cycle time in seconds; and
N_o = average overflow queue (veh) as defined in the equation for calculating queue lengths.

The number of stops per movement results from multiplying the stop rate (h) by the demand volume in vehicles per hour.

Fuel Consumption. Faced with fuel shortage and increased fuel prices, traffic signal analysts have become more and more interested in fuel consumption estimates. Fuel is consumed travelling between intersections; decelerating to a stop and accelerating to the desired speed; and idling while stopped at traffic signals on red. The following model is used to calculate fuel consumption in both TRANSYT-7F and PASSER II-90.

$$\begin{aligned} F \quad &= (A_{11} + A_{12}*V + A_{13}*V^2) * TT \\ &+ (A_{21} + A_{22}*V + A_{23}*V^2) * D \\ &+ (A_{31} + A_{32}*V + A_{33}*V^2) * S \end{aligned}$$

where:
- F = estimated total system fuel consumption, gal/hr;
- TT = total travel, veh-mile/hr;
- D = total delay, veh-hr/hr;
- S = total stops, stops/hr;
- V = cruise speed, mph; and
- A_{ij} = regression model beta coefficients, and is given by,

$$A_{ij} = \begin{vmatrix} 0.75283 & -1.5892\text{ E-3} & 1.50655\text{ E-5} \\ 0.73239 & 0.0 & 0.0 \\ 0.0 & 0.0 & 6.14112\text{ E-6} \end{vmatrix}$$

2-6. Example Problem

Example 2-1 illustrates the methodology for calculating the minimum delay cycle and the green splits for a signalized intersection. Figure 2-9 shows the traffic volume information for the intersection.

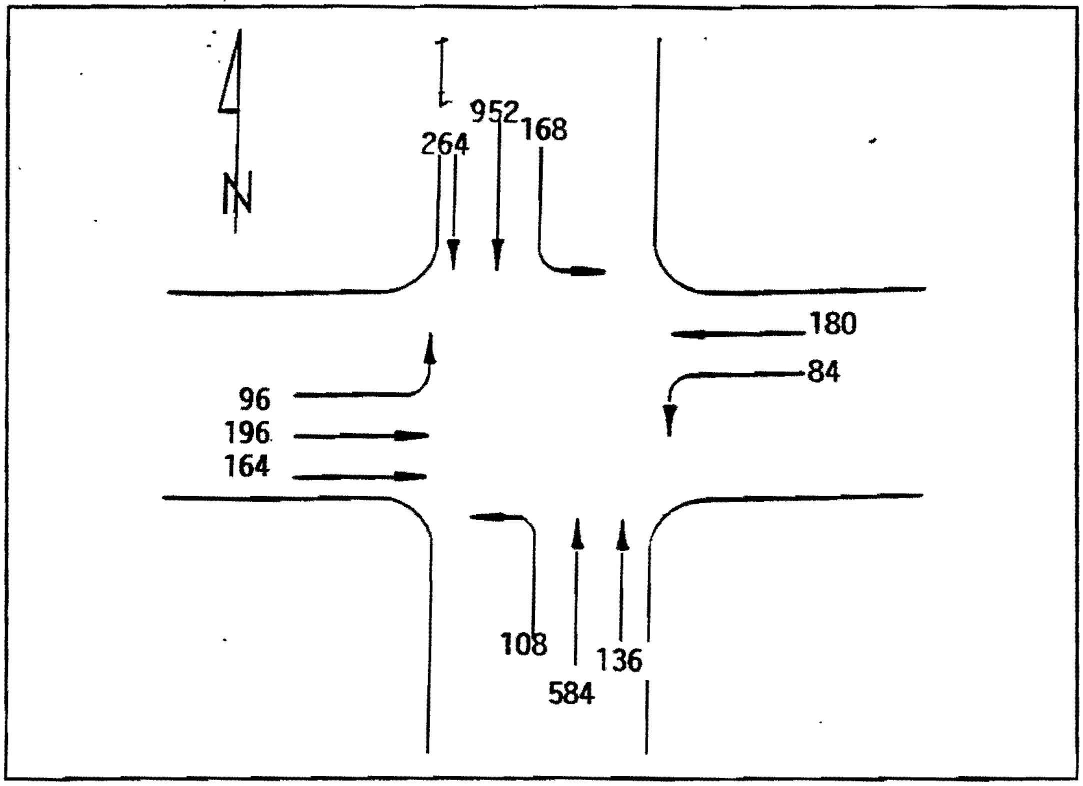


Figure 2-9. Intersection Diagram for Example 2-1

Given: Information (including volumes) shown in Figure 2-9;
All left turns are protected;
Saturation flow rate for through movements = 1800 vplphg;
Saturation flow rate for left-turn movements = 1700 vplphg; and
Lost time per phase = 4 seconds.

Find: Minimum delay cycle length, and green splits.

Solution: Two basic types of phasing sequences can be used at any intersection, as mentioned earlier (i.e., lead-lag or lag-lead with overlap phasing and leading left or lagging left phasing). This example describes the procedure to determine the minimum delay cycle length (C_o) and green splits for both types of phasing.

Lead-lag or Lag-lead Phasing with Overlap

Figure 2-4 illustrates lead-lag with overlap phasing. The critical movements can be obtained by determining the largest sum of the conflicting movement volumes per lane. Through and their opposing left-turn movements represent the conflicting movements for both the main street and cross streets. From Figure 2-9, the conflicting movement volumes at the intersection are:

NBL (1)	= **108** vehicles	SBT (2)	= (952 + 264) / 2 = **608** vehicles
SBL (5)	= 168 vehicles	NBT (6)	= (584 + 136) / 2 = 360 vehicles
EBL (3)	= **96** vehicles	WBT (4)	= **180** vehicles
WBL (7)	= 84 vehicles	EBT (8)	= (196 + 164) / 2 = 180 vehicles

The movements with their volumes in bold are the critical volumes. The numbers in parenthesis indicate the NEMA phase numbering for the various movements. Critical movement volumes were calculated as follows:

Movement	NBL	+	SBT	>	SBL	+	NBT
Volumes	**108**	+	**608**	>	168	+	360
Phase #	(1)	+	(2)	>	(5)	+	(6)
Movement	EBL	+	WBT	>	WBL	+	EBT
Volumes	**96**	+	**180**	>	84	+	180
Phase #	(3)	+	(4)	>	(7)	+	(8)

The sum of the critical movements equals:

(NBL + SBT) + (EBL + WBT) = (108 + 608) + (96 + 180) = **992**
(1) + (2) + (3) + (4)

Using a saturation flow for through movements of 1800 vplphg and a saturation flow for left turns of 1700 vplphg, the flow ratios for the critical movements are calculated as follows:

$$y_i = V_i / S_i$$

$y_1 = 108 \ / \ 1700 \quad = 0.064$

$y_2 = 608 \ / \ 1800 \quad = 0.338$

$y_3 = 96 \ / \ 1700 \quad = 0.056$

$y_4 = 180 \ / \ 1800 \quad = \underline{0.100}$

$\Sigma Y \quad = 0.558$

Similarly, the flow ratios for the non-critical phases are calculated.

$y_5 = 168 \ / \ 1700 \quad = 0.099$

$y_6 = 360 \ / \ 1800 \quad = 0.200$

$y_7 = 84 \ / \ 1700 \quad = 0.049$

$y_8 = 180 \ / \ 1800 \quad = 0.100$

The minimum delay cycle length is calculated based on the sum of the critical flow ratios and the total lost time per cycle:

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

where: $L = (4 \text{ sec/phase}) * 4 \text{ phases} = 16$ seconds;
$C_o = (1.5 * 16 + 5) \ / \ (1 - 0.558) = 65.6$ seconds;

Rounded to $C_o = 70$ seconds.

The 70 second cycle must be allocated to the four critical and four non-critical phase movements. The green splits are determined as follows (using a 70 second cycle length):

$$g_i = (y_i * C)/X_I$$

where: $X_I = Y * (C / (C - L))$
$= 0.558 * (70 / (70 - 16)) = \mathbf{0.723}$

Calculating the portion of the cycle length for the arterial (G_A) and the cross street (G_C)

$$G_A = (C - L) (Y_{Ac}/Y) + L_A$$

where:
- C = Cycle length;
- L = Total lost time;
- Y_{Ac} = Sum of the flow ratios for the critical phases on the arterial;
- Y = Sum of the flow ratios for the critical phases on the arterial and cross streets; and
- L_A = Lost time for the arterial phases.

$G_A = (70 - 16) (0.402/0.558) + 8$
$G_A = 46.9 \text{ seconds} = \mathbf{47\ seconds}$
$G_C = 70 - 47 = \mathbf{23\ seconds}$

Calculating the green splits for the critical phases on the arterial:

$G_1 = (G_A - L_A) (y_1/Y_{Ac}) + l_1$
$G_1 = (47 - 8) (0.064/0.402) + 4$
$G_1 = 10.2 \text{ seconds} = \mathbf{10\ seconds}$
$G_2 = 47 - 10 = \mathbf{37\ seconds}$

Calculating the green splits for the critical phases on the cross street:

$G_4 = (G_C - L_C) (y_4/Y_{Cc}) + l_4$
$G_4 = (23 - 8) (0.100/0.156) + 4$
$G_4 = 13.6 \text{ seconds} = \mathbf{14\ seconds}$
$G_3 = 23 - 14 = \mathbf{9\ seconds}$

Calculating the green splits for the non-critical phases on the arterial:

$$G_5 = (G_A - L_A)\,(y_5/Y_{An}) + l_5$$
$$G_5 = (47 - 8)\,(0.099/0.299) + 4$$
$$G_5 = 16.9 \text{ seconds} = \mathbf{17\ seconds}$$
$$G_6 = 47 - 17 = \mathbf{30\ seconds}$$

Calculating the green splits for the non-critical phases on the cross street:

$$G_7 = (G_C - L_C)\,(y_7/Y_{Cn}) + l_7$$
$$G_7 = (23 - 8)\,(0.049/0.149) + 4$$
$$G_7 = 8.9 \text{ seconds} = \mathbf{9\ seconds}$$
$$G_8 = 23 - 9 = \mathbf{14\ seconds}$$

where:
Y_{Ac} = Sum of the flow ratios for the critical phases on the arterial;
Y_{Cc} = Sum of the flow ratios for the critical phases on the cross street;
Y_{An} = Sum of the flow ratios for the non-critical phases on the arterial; and
Y_{Cn} = Sum of the flow ratios for the non-critical phases on the cross street.

The calculated phase times for the through phases must satisfy the minimum pedestrian requirements. These requirements are 3 to 7 seconds for pedestrians to begin walking plus adequate time for them to reach the middle of the furthermost lane at a comfortable walking speed. Assuming 11-foot wide lanes and a walking speed of 4 feet per second, the minimum pedestrian requirements are calculated below. Phase times for the through movements must be increased if the pedestrian requirement is higher than the calculated green split. One can accommodate this need either by increasing the cycle length or by taking some time away from other movements. In the example, 1 second of green is taken from Phase 2 and Phase 6 and added to Phase 4 and Phase 8.

Sum of the phases:

Critical Phases		Non-critical Phases		Pedestrian Requirement
G_1	= 10 seconds	G_5	= 17 seconds	
G_2	= 36 seconds	G_6	= 29 seconds	$3 + 49.5/4 \approx 15$ seconds
G_3	= 9 seconds	G_7	= 9 seconds	
G_4	= 15 seconds	G_8	= 15 seconds	$3 + 49.5/4 \approx 15$ seconds
Sum	= 70 seconds	Sum	= 70 seconds	

Dual left leading or dual left lagging phasing without overlap

Figure 2-3 illustrates lead-lead or lag-lag without overlap phasing. The critical movements can be obtained by determining the sum of the largest of the left turn and the through volumes per lane. From Figure 2-9 and Table 2-1, critical movement volumes are calculated as follows:

NBL (1)	= 108 vehicles	SBT (2)	= (952 + 264) / 2 = **608** vehicles
SBL (5)	= **168** vehicles	NBT (6)	= (584 + 136) / 2 = 360 vehicles
EBL (3)	= **96** vehicles	WBT (4)	= **180** vehicles
WBL (7)	= 84 vehicles	EBT (8)	= (196 + 164)/ 2 = 180 vehicles

Maximum of NBL (1) or SBL (5) + maximum of SBT (2) or NBT (6)
Maximum of 108 or **168** + maximum of **608** or 360

Maximum of EBL (3) or WBL (7) + maximum of WBT (4) or EBT (8)
Maximum of **96** or 84 + maximum of **180** or 180

The sum of the critical movements equals:

(SBT + SBL) + (EBL + WBT) = (608 + 168) + (96 + 180) = **1052**
(2) + (5) + (3) + (4)

Using a saturation flow rate for through movements of 1800 vplphg and a saturation rate flow for left turns of 1700 vplphg, the flow ratios for the critical movements are calculated as follows:

$$y_i = V_i / S_i$$

$y_2 = 608 / 1800 = 0.338$

$y_5 = 168 / 1700 = 0.099$

$y_3 = 96 / 1700 = 0.056$

$y_4 = 180 / 1800 = \underline{0.100}$

$\Sigma Y = 0.592$

Similarly, the flow ratios for the non-critical phases are calculated.

$y_1 = 108 \ / \ 1700 \quad = 0.064$

$y_6 = 360 \ / \ 1800 \quad = 0.200$

$y_7 = 84 \ / \ 1700 \quad = 0.049$

$y_8 = 180 \ / \ 1800 \quad = 0.100$

The minimum delay cycle length is calculated based on the sum of the critical flow ratios and the total lost time per cycle:

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

where: $L = (4 \text{ sec/phase}) \times 4 \text{ phases} = 16 \text{ seconds};$
$C_o = (1.5 * 16 + 5) \ / \ (1 - 0.592) = 71 \text{ seconds};$

Rounded to the nearest 5 seconds, $C_o = 70$ seconds.

The 70 second cycle must be allocated to the four critical and four non-critical phase movements. The green splits are determined as follows (using a 70 second cycle length):

$$g_i = y_i C \ / \ X_I$$

where: $X_I = Y \ * (C \ / \ (C - L))$
$= 0.592 * (70 \ / \ (70 - 16)) = \mathbf{0.767}$

Note that the intersection volume to capacity ratio (X_I) when using a phasing pattern without overlap is slightly higher than the intersection volume to capacity ratio (X_I) when using a phasing pattern with overlap.

Calculating the portion of the cycle length for the arterial (G_A) and the cross street (G_C)

$$G_A = (C - L) (Y_{Ac}/Y) + L_A$$

where: C = Cycle length;
L = Total lost time;
Y_{Ac} = Sum of the flow ratios for the critical phases on the arterial;
Y = Sum of the flow ratios for the critical phases on the arterial and cross streets; and
L_A = Lost time for the arterial phases.

G_A = (70 - 16) (0.437/0.592) + 8
G_A = 47.9 seconds = **48 seconds**
G_C = 70 - 48 = **22 seconds**

Calculating the green splits for the critical phases on the arterial:

$G_2 = (G_A - L_A)\ (y_2/Y_{Ac}) + l_2$
G_2 = (48 - 8) (0.338/0.437) + 4
G_2 = 34.9 seconds = **35 seconds**
G_5 = 48 - 35 = **13 seconds**

Calculating the green splits for the critical phases on the cross street:

$G_4 = (G_C - L_C)\ (y_4/Y_{Cc}) + l_4$
G_4 = (22 - 8) (0.100/0.156) + 4
G_4 = 13.6 seconds = **14 seconds**
G_3 = 22 - 14 = **8 seconds**

Assigning the green splits for the non-critical phases on the arterial and cross streets:

$G_1 = G_5 =$ 13 seconds
$G_6 = G_2 =$ 35 seconds
$G_7 = G_3 =$ 8 seconds
$G_8 = G_4 =$ 14 seconds

where: Y_{Ac} = Sum of the flow ratios for the critical phases on the arterial;
Y_{Cc} = Sum of the flow ratios for the critical phases on the cross street;
Y_{An} = Sum of the flow ratios for the non-critical phases on the arterial; and
Y_{Cn} = Sum of the flow ratios for the non-critical phases on the cross street.

The calculated phase times for the through phases must satisfy the minimum pedestrian requirements. These requirements are 3 to 7 seconds for pedestrians to begin walking plus adequate time for them to reach the middle of the furthermost lane at a comfortable walking speed. Assuming 11-foot wide lanes and a walking speed of 4 feet per second, the minimum pedestrian requirements are calculated below. Phase times for the through movements must be increased if the pedestrian requirement is higher than the calculated green split. One can accommodate this need either by increasing the cycle length or by taking some time away from other movements. In the example, 1 second of green is taken from Phase 2 and Phase 6 and added to Phase 4 and Phase 8.

Sum of the phases:

Critical Phases		Non-critical Phases		Pedestrian Requirement
G_2	= 34 seconds	G_6	= 34 seconds	$3 + 49.5/4 \approx 15$ seconds
G_5	= 13 seconds	G_1	= 13 seconds	
G_3	= 8 seconds	G_7	= 8 seconds	
G_4	= 15 seconds	G_8	= 15 seconds	$3 + 49.5/4 \approx 15$ seconds
Sum	= 70 seconds	Sum	= 70 seconds	

Tables 2-3 and 2-4 summarize the measures of effectiveness for the intersection in Example 2-1 using the formulae mentioned earlier for different measures of effectiveness. One can estimate the level-of-service (LOS) for each movement on the basis of the average delay per vehicle. The LOS thus obtained can be compared with the volume to capacity ratio for the respective movements. Generally, the delay LOS is consistent with the volume to capacity ratio for the movement. There may be some movements, however, where this consistency may not exist. For example, some low volume movements may experience low volume to capacity ratios but relatively high delay. This result occurs whenever a small amount of delay is divided by a small number of vehicles; i.e., low volume turning movements.

In order to have a rough estimate of the overall intersection performance, one obtains a weighted average of the delay values by multiplying each delay value with its corresponding hourly volume and dividing the sum of the products by the total volume entering the intersection during the hour. Thus, one obtains a value of average intersection delay in seconds per hour of operation. This value may be used as measure of the effectiveness of the intersection's signal timing. In Example 2-1, the weighted average delay for with overlap phasing is 17.92 seconds compared to 18.05 seconds for without overlap phasing. This difference is not significant because of balanced left-turn and through volumes; however, for signal timing plans having high and low volume movements in the same phase, the difference in the two alternatives could be significant.

Table 2-3. Measures of Effectiveness Using Lead-Lag Phasing with Overlap

Mvmt (#)	Vol (vph)	green (sec)	v/c Ratio	Max. Que. Length* (veh/cyc)	Avg Delay (sec/veh)	Total Delay (hr/hr)	Stops (veh)	Fuel Consu. (gal/hr)
1	108	10	0.44	2	28.6	0.86	94	1.230
2	1216	36	0.66	18	13.3	4.48	805	8.684
3	96	9	0.44	2	29.4	0.78	85	1.117
4	180	15	0.47	4	24.9	1.24	144	1.851
5	168	17	0.41	3	22.8	1.06	129	1.620
6	720	29	0.48	11	15.3	3.05	475	5.426
7	84	9	0.38	2	28.7	0.67	72	0.956
8	360	15	0.47	7	24.4	2.44	284	3.642
	2932					14.60	2087	24.53

Table 2-4. Measures of Effectiveness Using Lead-Lead Phasing without Overlap

Mvmt (#)	Vol (vph)	green (sec)	v/c Ratio	Max. Que. Length* (veh/cyc)	Avg Delay (sec/veh)	Total Delay (hr/hr)	Stops (veh)	Fuel Consu. (gal/hr)
1	108	13	0.34	2	25.1	0.75	86	1.110
2	1216	34	0.70	19	15.1	5.10	853	9.406
3	96	8	0.49	2	31.2	0.83	89	1.178
4	180	15	0.47	4	24.9	1.24	144	1.851
5	168	13	0.53	3	27.5	1.28	143	1.864
6	720	34	0.41	9	11.7	2.34	417	4.581
7	84	8	0.43	2	30.2	0.70	76	1.000
8	360	15	0.47	7	24.4	2.44	284	3.642
	2932					14.70	2092	24.63

*If a movement has more than one lane, the queue length is divided by the number of lanes.

3.0 DATA REQUIREMENTS

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

The following sections discuss guidelines and suggestions for complete and accurate data collection needed for retiming isolated intersections. These data are used in the development of timing plans for both pretimed and traffic actuated environments. Section 4.0, "Evaluation," Section 5.0, "Optimization," and Section 6.0, "Implementation," describe the recommended use of the data.

Three types of data need to be collected:

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

The type of data needed for analyzing an intersection varies somewhat for each intersection. A worksheet for recording data, such as the one specified in the *Highway Capacity Manual* (4) and illustrated in Figure 3-1, is helpful.

The first question to be asked is how many timing plans are needed? The number of timing plans necessary depends on the fluctuation of traffic demand throughout the day and the type of control equipment available. Data should be collected during the periods of interest. For example, data for developing an a.m. peak timing plan should be collected during the a.m. peak time period, data for developing an off-peak timing plan should be collected during the off-peak time period, etc.

3.1 Traffic Data

Traffic data identifies both the demand and the capacity of the intersection. The quantification of demand requires observation of the daily traffic volume at the intersection, the traffic volume during the peak period, and the traffic volume for specific turning movements. One calculates the capacity of an intersection based on the saturation flow rate and available green time for each movement. The number of heavy vehicles using the intersection, bus stops and parking near the intersection, and the number of pedestrians that cross at the intersection affect the saturation flow rate. The following text elaborates on the collection of traffic data.

INPUT WORKSHEET

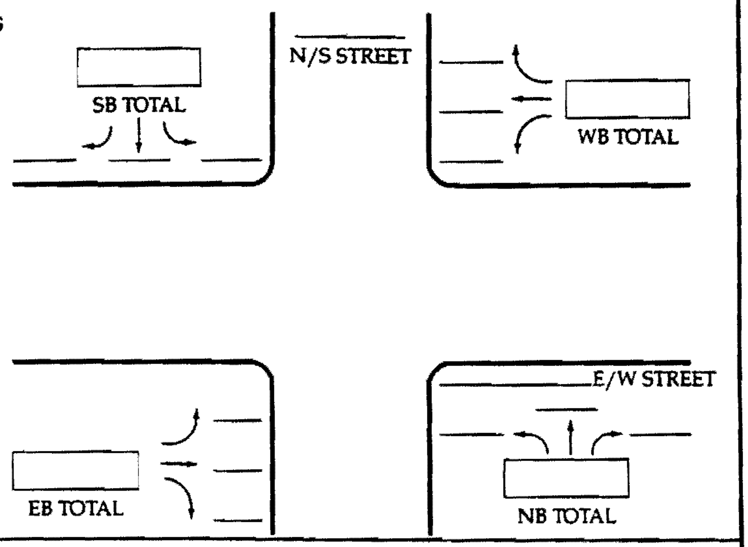
Intersection:______________________________ Date:______________

Analyst:______________ Time Period Analyzed:________ Area Type: ☐ CBD ☐ Other

Project No.:__________________ City/State:__________________

VOLUME AND GEOMETRICS

NORTH



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane		Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button		Arr. Type
			Y or N	N_m				Y or N	Min. Timing	
EB										
WB										
NB										
SB										

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated								

Protected turns | Permitted turns | ------- Pedestrian | Cycle Length_____Sec

Figure 3-1. HCM Worksheet to Summarize Intersection Data

Traffic Volumes. Traffic volumes should be obtained for the a.m., p.m., and off-peak periods as a first step in this process. A 24-hour count should be made to determine the peak periods (or peak 15 minutes) and the fluctuation in traffic demand. A 24-hour count can be taken by placing tube counters on all approaches at the intersection or by dumping detector counts from the controller. Figure 3-2 shows an example of a 24-hour count printout and the determination of the peak hours.

Turning Movements. Once the peak period is determined, manual counts are necessary to record the volumes for individual movements or lane groups during the peak period or period of interest. There are twelve possible movements that need to be counted at each signalized intersection, as shown in Figure 3-3. Generally, turning movement counts should be made in 15-minute intervals during the two hour a.m. or p.m. peaks, and for one hour during the off-peak period.

One adds the highest four consecutive 15-minute volumes together to determine the highest peak or off-peak hour flow rate **or** adjustments to the hourly counts may be made using a peak hour factor (PHF). It may prove helpful to record intersection data on a worksheet, such as the one in the *Manual of Traffic Engineering Studies* (7), shown in Figure 3-4. A sketch illustrating the orientation of the intersection and its basic features will also be helpful for recording intersection data.

During congested periods, it is important that the volume counted be the demand rather than the 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 include those vehicles that arrive at the back of the queue rather than those vehicles that depart when the signal is green. It should noted, however, that this procedure is for counting and not 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.

Right Turn on Red (RTOR). To more accurately describe the existing conditions, the number of vehicles making right turns on the red interval should be recorded. This number will be subtracted from the total right-turn volume when modeling (analyzing) the existing conditions at the intersection.

Peak Hour Factor (PHF). After making these counts, adjustments may be necessary to account for the peak flow period. One relates peak rates of flow to hourly volumes through the use of the peak hour factor. The ratio of total hourly volume to the maximum 15-minute rate of flow within the hour defines the PHF.

TRAFFIC COUNT SHEET

LOCATION: WALKER DR. East of DANTROSE DATE: SEPTEMBER 26-27, 1984

DIRECTION OF TRAVEL: WEST-BOUND DAY: WEDNESDAY-THURSDAY

TIME	COUNT	HOUR COUNT	TIME	COUNT	HOUR COUNT	TIME	COUNT	HOUR COUNT
24:00	15	41	8:00	215	604	16:00	145	643
:15	14		:15	129		:15	142	
:30	7		:30	126		:30	165	
:45	5		:45	134		:45	191	
1:00	6	25	9:00	105	483	17:00	200	715
:15	7		:15	104		:15	186	
:30	8		:30	141		:30	178	
:45	4		:45	133		:45	151	
2:00	7	15	10:00	117	482	18:00	115	441
:15	3		:15	124		:15	116	
:30	4		:30	108		:30	110	
:45	1		:45	133		:45	100	
3:00	4	10	11:00	122	559	19:00	106	380
:15	2		:15	136		:15	107	
:30	3		:30	132		:30	95	
:45	1		:45	169		:45	72	
4:00	0	17	12:00	154	639	20:00	72	233
:15	7		:15	128		:15	57	
:30	5		:30	139		:30	57	
:45	5		:45	218		:45	47	
5:00	2	26	13:00	150	599	21:00	54	157
:15	7		:15	177		:15	26	
:30	7		:30	132		:30	43	
:45	10		:45	140		:45	34	
6:00	17	120	14:00	128	536	22:00	34	96
:15	27		:15	136		:15	21	
:30	30		:30	138		:30	22	
:45	46		:45	134		:45	19	
7:00	83	670	15:00	155	630	23:00	17	60
:15	128		:15	117		:15	14	
:30	206		:30	184		:30	13	
:45	253		:45	174		:45	16	

A.M. PEAK:
(07:30- 08:30)

206	803
253	
215	
129	

P.M. PEAK:
(16:45 -17:45)

191	755
200	
186	
178	

24-HOUR TOTAL:
(12:00 WED-12:00 THUR)

8181

Figure 3-2. Example of 24-hour Count Data and Peak Periods

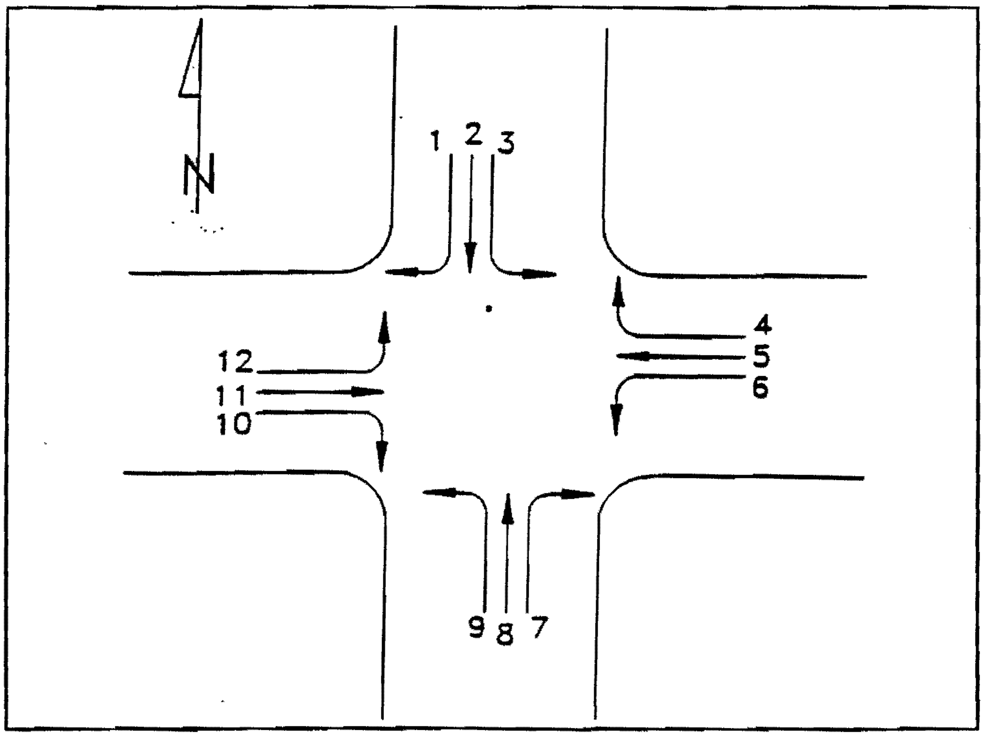


Figure 3-3. Turning Movements to be Counted at a Typical Intersection

TABULAR SUMMARY OF VEHICLE COUNTS

Observer ______ Date ______ Day ______ City ______

INTERSECTION OF ______ AND ______

R = Right turn
S = Straight
L = Left turn

TIME BEGINS	from NORTH				from SOUTH				TOTAL North South	from EAST				from WEST				TOTAL East West	TOTAL ALL
	R	S	L	Total	R	S	L	Total		R	S	L	Total	R	S	L	Total		

Figure 3-4. Worksheet to Record Intersection Data

If a traffic signal analyst uses 15-minute counts, then:

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

where: PHF = peak hour factor;
V = the highest hourly volume; and
V_{15} = the highest 15-minute count within that hour.

Generally, one states demand volumes in terms of vehicles per hour for the peak hour. For analysis, peak hours are normally adjusted to flow rates in vehicles per hour for a 15-minute period. For example, analysts made a 24-hour count and determined the a.m. peak hour to be between 7:00 a.m. and 8:00 a.m. The hourly volume was determined to be 900 vehicles per hour. Analysts then determined the peak 15-minute flow rate within that hour to be 300 vehicles in 15 minutes or 1200 vehicles per hour.

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

Thus, the peak hour factor for the a.m. peak equals 0.75, in this example. The peak hour flow is 900 vehicles per hour and the peak 15-minute flow rate equals 1200 vehicles per hour. For timing purposes, the peak hour flow rate can be calculated either by dividing the hourly volume by the peak hour factor or by multiplying the peak 15-minute flow rate by four. In either case, the calculated peak hour flow rate equals 1200 vehicles per hour.

Saturation Flow Rate. The saturation flow rate is the maximum flow rate of vehicles entering the intersection from either exclusive through or through-right lanes. This rate is expressed in terms of vehicles per hour of green per lane, under prevailing roadway conditions during the peak hour demand. Adjustment factors for roadway and traffic conditions, such as lane width and truck percentages, reduce the ideal saturation flow rate to an adjusted rate that is appropriate for the location. The following traffic data items adjust the ideal saturation flow rate:

Percent heavy vehicles - The number of heavy vehicles operating within the intersection should be counted. A heavy vehicle has at least 6 wheels in contact with the roadway. Heavy vehicles may be classified into three types: trucks, recreational vehicles, and buses. Heavy vehicles take up more lane space and operate differently than passenger vehicles, which contributes to a decrease in the saturation flow rate and capacity. For example, heavy vehicles accelerate from a stop at a slower rate.

Parking - Parking in the vicinity of the intersection (within 200 feet of the stop line) also will affect the flow in adjacent lanes, either by frictional effect or occasional blockage of a lane due to a parking maneuver.

Bus stops - If buses make scheduled stops at an established bus stop near the intersection (within 200 feet), restriction of flow and capacity may result in lanes adjacent to the bus stop. The time of day and frequency of bus stops should be recorded. Most bus companies post their schedules at the bus stop, and further information on bus frequency may be obtained from the bus company.

Pedestrians - The number and types of pedestrians crossing the intersection should be observed. Elderly pedestrians and children require more time to cross the street. One needs this information for calculating minimum green times, whether or not the intersections have pedestrians signals. Right turn conflicts with pedestrians also should be noted; if the right turn conflict is heavy, the capacity available for right turns may need reduction. 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 the data be based on field observations.

One should realize that saturation flow rates are extremely important when determining the capacity and required splits for specific movements. For example, if a particular movement's saturation flow rate is overestimated, less green time than is needed will be allocated to that movement. On the other hand, if a particular movement's saturation flow rate is underestimated, more green time than is needed will be allocated to that movement. Neither condition is desirable.

Because saturation flow rate is such a critical factor, it should be measured in the field if possible. Figure 3-5 shows a worksheet for calculating the saturation flow rate for an approach using field observations. If field measurements are not available, the saturation flow rate can be estimated using the procedure outlined in the *1985 Highway Capacity Manual* (4).

3.2 Signal Data

Most information classified as signal data may be taken directly from the controller for pretimed control. The fixed yellow and red intervals remain constant for actuated control; maximum green intervals also may be obtained from the actuated controller. Average green intervals and cycle lengths for actuated control may be obtained from field measurements. The following text elaborates on the necessary signal data.

Cycle Length. The cycle length for the period of interest, a.m., p.m., or off peak, should be recorded. For pretimed control, the cycle length remains constant. The cycle length may be obtained from existing timing plans, the controller, or by field measurement with a stopwatch. Signals controlled by an actuated controller will have variable cycle lengths; in these cases, an average cycle length during the study period should be determined from field measurements. It is recommended that 10 to 30 field measurements be taken and averaged to find the average cycle length.

Green Splits. The green splits for each phase should be recorded. The green split, i.e., green plus yellow plus red clearance for each phase, remains constant for pretimed control and can be obtained from existing timing plans, the controller, or by field measurement with a stop watch. The green interval, will be a variable length interval for actuated control and for analysis purposes, an average value for the green interval should be determined by recording 10 to 30 measurements of the green interval in the field and calculating an average green interval length. The yellow and red intervals remain constant for both pretimed and actuated control. This information may be obtained from timing plans or field measurements.

FIELD SHEET – SATURATION FLOW STUDY

Location:____________________

Date:____/____/____ Time:________ City:________

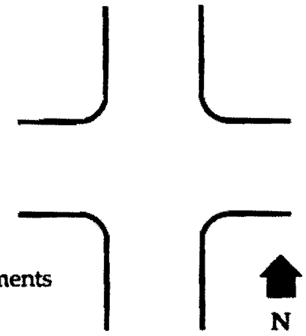
____Bound Traffic; Approaching From the________

Observers:________ Weather:________

Movements Allowed

- ☐ Thru
- ☐ Right Turn
- ☐ Left Turn

Identify all Lane Movements & The Lane Studied



Veh. in Queue	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of Saturation																		
End of Green																		
No. Veh. > 20																		
No. Veh. on Yellow																		

HV = Heavy Vehicles (Vehicles with more than 4 tires)
T = Turning Vehicles (L = Left, R = Right)
Pedestrians and buses which block vehicles should be noted with the time that they block traffic, i.e.,
P12 = pedestrians blocked traffic for 12 sec
B15 = bus blocked traffic for 15 sec

Grade________ Area Type________

Figure 3-5. Worksheet for Calculating the Saturation Flow Rate

Phasing. One should record the existing phasing, including the type and sequence of phasing, for the intersection. As discussed in Section 2.2, the left-turn treatment determines the type of phasing, and the sequence is described by the order that the left-turn phases occur. Types of phasing include protected, permitted, and protected/permitted.

Protected -	A green arrow indicates a protected left-turn movement otherwise, left-turning vehicles may not proceed.
Permitted -	Left-turn movements proceed during green ball indication after yielding to opposing traffic.
Combined -	Left-turn movements are protected with a green arrow and also may proceed during the green ball indication after yielding to opposing traffic.

Possible phase sequences at an intersection include leading lefts, lagging lefts, and lag/lead or lead/lag.

Leading Lefts	-	Both left-turns proceed before the through movements.
Lagging Lefts	-	Both left-turns proceed after the through movements.
Lag-lead/lead-lag	-	One left-turn movement and its adjacent through movement proceed before or after the opposing left-turn and its adjacent through movement.

Type of Controller. As mentioned previously, actuated or pretimed controllers may control isolated intersections. Section 2.3 addressed the characteristics and capabilities of these controllers. In general, the following attributes should be noted for each type of controller.

Actuated -	Determine if the controller is single ring or dual ring; how many timing plans, cycle lengths, and split patterns can be accommodated?
Pretimed -	Determine if the controller is electromechanical or digital; how many dials are available to provide different timing plans?

Signal Hardware. The number of signal heads and the size of the mast arm should be noted. Existing plans or field observation can provide this information. The signal hardware may be a constraint for some traffic signal timing plans. For example, the MUTCD requires two signal heads for through movements. If a separate left-turn signal is desired, a short mast arm may not accommodate the additional signal head required for the left-turn movement.

3.3 Geometric Data

Geometric data may be determined from site plans or through field inspections. The geometrics of an intersection will affect operational aspects, such as the saturation flow rate. Additionally, the geometric characteristics of an intersection will influence signal data, including the phasing and left-turn treatments. The following data is classified as geometric data.

Number of Lanes per Approach. One should record the number of lanes for each approach to the intersection. Note that the number of lanes is counted at the stop bar, not upstream or downstream of the intersection. The type of movements allowed from each lane should be recorded, including exclusive turning lanes and shared lanes. The following information also should be noted for each type of lane and/or movement:

Left-turn lanes -	the number of lanes, whether left turns have an exclusive lane, the storage length, and whether storage is adequate for the expected queue.
Through lanes -	the number of lanes, and whether the through lanes accommodate left or right turns.
Right-turn lanes -	the number of lanes, and whether right turns have an exclusive lane.

Lane Widths. The lane widths for each lane on the approach should either be measured or obtained from existing plan sheets. Lane width will affect the saturation flow rate. Lanes less than 12 feet in width reduce the available capacity.

Percent Grade. The percent grade at each approach should be recorded. This information can be obtained from existing plan sheets or by field measurement. Percent grade will affect the saturation flow, and possibly the lost time, due to longer vehicle start-up times on an uphill grade.

Location. The location of the intersection with respect to the surrounding area should be noted. For example, does the intersection lie in a central business district (CBD)? An example of a CBD would be a downtown area where arterial streets cross each other and all streets are of the same importance, creating an urban street network. The high density development of a CBD usually results in heavy pedestrian traffic, and additional parking maneuvers and turning movements.

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4.0 EVALUATION

After data collection, the next step necessary for signal retiming is the analysis of existing conditions. Evaluation of an existing control strategy requires field observation as well as analysis of existing conditions with computerized models. The sections below summarize a recommended methodology for assessing the operational efficiency of a traffic signal control strategy at an isolated intersection:

Field Evaluation

1. Check that the green intervals are long enough to clear the stopped queues during most time periods. Although this objective may not be a desirable strategy with actuated control and oversaturated conditions, cycle failure over an extended period of time indicates signal timing or geometric problems. Such problems result in long delays and queue lengths, and excess fuel consumption.

2. Check that the green intervals are short enough that no period of time exists when vehicles are not moving through the intersection. Longer than necessary green intervals create wasted time and result in unnecessary delay and longer queue lengths for the other movements.

3. Check that the left-turn queue does not exceed the left-turn storage. If so, left-turning vehicles may block the through lane and reduce its saturation flow-rate; i.e., the available through capacity cannot be fully utilized. The opposite condition, long through queues blocking access to a left-turn lane, has a similar effect on left-turn capacity. Neither condition is desirable.

Computer Analysis

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

2. Check that volume to capacity ratios for individual movements do not exceed 1.2. If so, the input data (usually capacity estimates) is probably in error, and should be corrected. If not, the green splits and/or cycle length may be too short and require lengthening.

3. Check that levels of service for volume to capacity ratios are not more than one letter grade below the levels of service for delay. If so, the green splits and/or cycle length are probably too long (wasted green time), and may require shortening.

4. Check that the estimated queue lengths do not exceed the available storage. If so, the intersection cannot operate at its full potential for moving traffic. Signal timing or geometric modifications may increase the intersection's operational efficiency.

4.1 Software Packages

One may perform evaluation or simulation of an isolated intersection's operation using computer programs, such as the Highway Capacity Software (8), PASSER II-90 (9), SOAP-84 (10, 11), TEXAS Model (12), TRAF-NETSIM (13), and EVIPAS (14). Measures of effectiveness calculated by these programs can be used to locate operational problems within the intersection and pinpoint areas needing improvement. The following section briefly describes each of the programs.

Highway Capacity Manual Software (HCS). The University of Florida developed the HCS software for the Federal Highway Administration. The program calculates saturation flow rates, delay, level of service, and other measures of effectiveness based on the *Highway Capacity Manual* (HCM) methodologies, the widely accepted standard for the analysis of signalized intersections. The HCM program is straightforward and easy to use; however, the program can only be used for evaluation, and can only evaluate one intersection at a time. For further information, consult the *Highway Capacity Software User'sManual* (HCM) (8).

PASSER II-90 (Progression Analysis and Signal System Evaluation Routine). The Texas Transportation Institute developed PASSER II-90 for the Texas Department of Transportation. The program analyzes both isolated intersections and arterial streets. In the optimization mode for isolated intersections, PASSER II-90 varies signal phasing and green splits using user-specified cycle lengths and minimum phase lengths to arrive at the optimal timing plan. The user may also perform evaluation of existing conditions. Features include provisions for actuated and pretimed control, an engineer's assistant key for calculating saturation flow rates using HCM methods, and the capability of modeling permitted left turns. Measures of effectiveness include volume-to-capacity ratios, delay, queues, stops, and fuel consumption. For further information, refer to *Arterial Signal Timing Optimization Using PASSER II-87* (9).

SOAP-84 (Signal Operations Analysis Package). The University of Florida developed SOAP-84 for the Federal Highway Administration (FHWA). The program has the capability for the analysis and optimization of isolated intersections. Like PASSER II-90, the program contains provisions for analyzing pretimed or actuated control and permitted left turns. The program determines optimum cycle lengths, but multiple runs remain necessary for phase sequence optimization. Saturation flow rates for capacity analysis may be predetermined, or calculated by coding the number of lanes for each movement. Only isolated intersections may be analyzed. Measures of effectiveness include delay, stops, fuel consumption, volume to capacity ratio, and left-turn conflicts. For further information, see the *SOAP-84 User's Manual* (10) and the *SOAP-84 Data Input Manager* (11).

TEXAS Model. The Center for Transportation Research at the University of Texas developed the TEXAS Model. The TEXAS Model evaluates and simulates existing or proposed conditions. 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 program is primarily used for evaluation and not optimization. For further information, refer to *TEXAS Model for Intersection Traffic* (12).

TRAF NETSIM Model. The Federal Highway Administration developed TRAF NETSIM, a microscopic simulation model. This model can simulate traffic control systems in great detail and can handle both isolated intersections and coordinated networks; however, the model cannot be used to optimize signal timing. 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 variables. For further information, refer to *TRAF User Reference Guide* (13).

EVIPAS. The University of Pittsburgh developed EVIPAS, an optimization simulation model for isolated intersections under actuated control, for the Pennsylvania Department of Transportation. EVIPAS can analyze and develop almost any phasing pattern available in a standard NEMA or Type 170 controller. Users select from a variety of measures of effectiveness to determine the optimal signal timing settings for pretimed, semi-actuated, fully actuated, or volume-density control with or without pedestrian actuations. For further information, refer to *EVIPAS: A Computer Model for the Optimal Design of a Vehicle Actuated Traffic Signal* (14).

These guidelines address the use of PASSER II-90 and SOAP-84 for the analysis of isolated intersections because both of these programs can simulate and optimize timing plans. In addition, an evaluation of the existing conditions using HCS is compared to the PASSER II-90 and SOAP-84 results. The other programs are limited to evaluating a given set of conditions, i.e.,they do not optimize signal timings; however, it should be noted that the two simulation programs are useful when evaluating complex geometrics or oversaturated conditions. The programs' user manuals contains further guidance on these applications.

4.2 Input Requirements

The following sections outline the basic requirements for the simulation and evaluation of existing conditions using PASSER II-90 and SOAP-84.

PASSER II-90. To evaluate existing conditions using PASSER II-90, the *Input New Data* option should be chosen from the *Main Menu* screen, shown in Figure 4-1. The *Input Menu* will then come up with the options *Input New Traffic Data*, *Input Embedded Data*, or *Input Phaser Data*.

The selection *Input New Traffic Data* prompts the "Arterial Data" input screen. The "Arterial Data" input screen, shown in Figure 4-2, allows the user to input the names and orientation of the main and cross streets. The cycle length also must be entered on this screen. For evaluating existing conditions, the user should specify the existing cycle length as both the lower and upper cycle lengths. The cycle length increment should be set to zero.

The "Vehicle Movement" screen, shown in Figure 4-3, allows the user to enter the volume, saturation flow rate, and minimum phase time for each movement. Volumes for the peak 15-minute flow, expressed as an hourly rate, are entered for left, through, and right turn movements. The right turn volume should be entered as zero if an exclusive right turn lane exists. When evaluating existing conditions, the sum of the existing phase times (coded as minimum phase times) should equal the existing cycle length. The user should specify the existing phase sequence on the "Phasing Patterns Entry" screen (Figure 4-4). Note that for existing conditions, only the existing sequence should be selected as an option.

The selection *Input Embedded Data* allows the user to specify whether the signal is pretimed or actuated, and to override the default parameters for the number of sneakers per phase (a "sneaker" is a vehicle that makes a left-turn during the yellow phase, after the opposing through traffic has stopped for the red light), the phase lost time, the saturation flow rate, the analysis period, the LOS criteria, the model used for describing permitted left-turn movements, and the permitted left-turn headway and critical gap. For a more complete description of these parameters, see the *PASSER II-90 User's Manual* (15).

Once the user has entered the intersection data, the *Run PASSER* option on the *Main Menu* should be selected. After completing the analysis, the *Output Menu* will be displayed. The *Output Menu* selection *View Best Solution* will provide information for each movement, including the volume to capacity ratio, and the corresponding LOS; vehicle delay and the corresponding LOS; the estimated queue length per lane; and the number of stops per hour. Information will also be provided for the entire intersection, including total intersection delay and fuel consumption. For existing conditions, the "minimum delay cycle" identified on the *Best Solution* output should be the existing cycle length and the phase times should correspond to the existing green splits. Figure 4-5 shows an example of the *Best Solution* output.

```
Texas Department of Highways and Public Transportation
----- P A S S E R  I I - 9 0 -----
Version 1.0

-- Main Menu --

1. Input new data.

2. Read old data from disk.

   - D:\P2\DATA loaded. -

3. Edit data.

4. Store data on disk.

5. Print current input data.

6. Run PASSER II-90.

7. Go to Output Menu.

8. Quit.

Which item do you choose? 1
```

Figure 4-1. PASSER II Main Menu Screen

```
[F2]                                                    <ESC>
                    PASSER II-90 Arterial Data

Run Number               :  1     City Name      : Dallas
Number of Intersections  :  1     Arterial Name  : Skillman
District Number          : 17     Date           : 09/15/88

Lower Cycle Length :    60    Movement #2 "A" Direction  : 0
Upper Cycle Length :    60
                              1 = North   3 = East   0 = None
Cycle Increment    :     0    2 = South   4 = West

        Output Level :  0         Isolated Operation

0 = Output All Pages
1 = Error Exit - Cover & Error Pages
2 = Less Input Data Echo
3 = Less Input Echo and Best Soln
4 = Simple - Cover, Pin.Set, T/S
5 = Debug - All Pages, Variables

Best Solution Format | 0 = PASSER II
     (0 or 1) : 0    | 1 = AAP P2
```

Figure 4-2. PASSER II Input New Traffic Data Screen

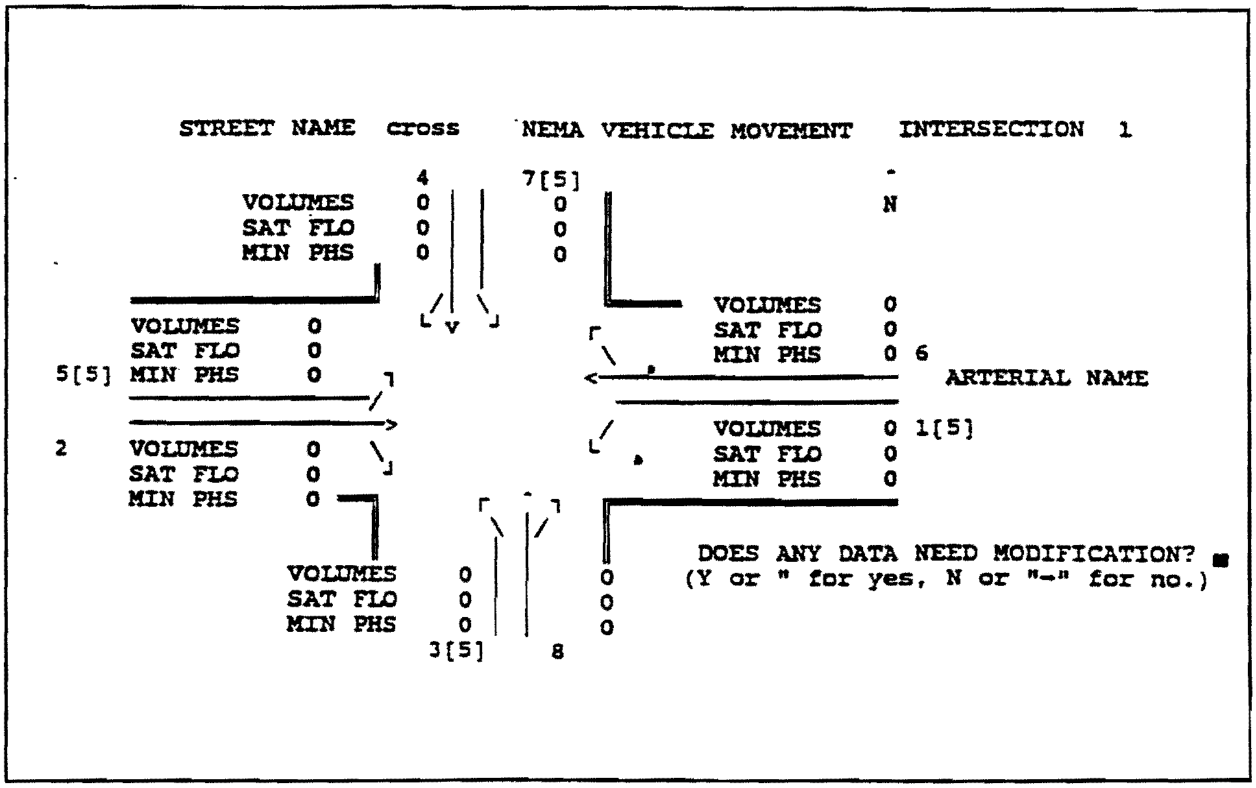


Figure 4-3. PASSER II Screen to Input Approach Volume

```
                         Phasing Patterns Entry

Arterial Name  :                          Intersection Number : 1
Cross Street   : Cross
                                        Arterial        Cross Street

Dual Lefts Leading with overlap            Y                 Y
Dual Lefts Leading without overlap         -                 -
Throughs First with overlap                -                 -
Throughs First without overlap             -                 -
Left Turn # 1 Leading with overlap         -                 -
Left Turn # 1 Leading without overlap      -                 -
Left Turn # 5 Leading with overlap         -                 -
Left Turn # 5 Leading without overlap      -                 -
Special Phasing Selection                  -                 -

Select which Phasing Patterns are needed.  <CR> to select, and <ESC> to exit
     "Y" = phasing selected, "-" = not selected, "Þ" = not possible.
   Note that "with overlap" and "without overlap" are mutually exclusive.
```

Figure 4-4. Phasing Pattern Entry Screen

```
(BEST.SOLN)
                TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
PASSER II-90     MULTIPHASE ARTERIAL PROGRESSION - 145101     VER 1.0  DEC 90

            **** BEST SOLUTION....  NEMA PHASE DESIGNATION ****
*** INT.    1     .0 SEC OFFSET     ART   ST PHASE SEQ IS LT 5 LEADS    (2+5)
Mockingbird       .0 %   OFFSET     CROSS ST PHASE SEQ IS LT 3 LEADS    (3+8)

                        ARTERIAL STREET                  CROSS STREET
CONCURRENT PHASES    2+5   2+6   1+6    TOTAL      3+8   4+8   4+7    TOTAL
PHASE TIME (SECS)    10.1  27.5  10.8    48.4      25.7  10.9  10.0    46.6
PHASE TIME (%)       10.6  28.9  11.4    50.9      27.1  11.5  10.5    49.1
                     ------------- MEASURES OF EFFECTIVENESS --------------
PHASE (NEMA)          5[5]   6      1[5]   2        3[5]   4      7[5]   8
PHASE DIRECTION      EBLTPR WBTHRU WBLTPR EBTHRU   NBLTPR SBTHRU SBLTPR NBTHRU
PHASE TIME (SEC)     10.1   38.3   10.8   37.6     25.7   20.9   10.0   36.6
V/C-RATIO             .81    .88    .42    .45      .62    .61    .40    .87
LEVEL OF SERVICE      D      E      A      A        B      B      A      E
DELAY (SECS/VEH)     73.5   33.4   44.0   23.9     35.6   37.1   44.5   33.6
LEVEL OF SERVICE      E      D      D      C        D      D      D      D
QUEUE (VEH/LANE)      1.8   10.3     .6    2.9      2.4    5.8     .5   14.5
STOPS (STOPS/HR)     109.  1028.    46.   302.     205.   481.    40.  1387.
TOTAL INTERSECTION DELAY       FUEL CONSUMPTION         MINIMUM DELAY CYCLE
    34.22 SECS/VEH               82.37 GAL/HR                97 SECS
```

Figure 4-5. PASSER II Best Solution Output

SOAP-84. SOAP-84 is coded using a series of cards (card images), rather than a series of menu options used in PASSER II-90. A number of cards are necessary for evaluation; this series of cards is termed the "input deck." The following text identifies and briefly describes the cards required for the evaluation of an isolated intersection. Note that some cards may have multiple fields; however, only the fields relevant to the evaluation of existing isolated intersection conditions are addressed.

SETUP - The SETUP card must be first in the sequence and is always required for simulation and optimization. On field two of the SETUP card, the user must specify "single-intersection, multi-period" for the analysis of an isolated intersection. The user must specify the number of time periods (48 maximum), and the period length in minutes in Fields 3 and 4, respectively.

INTERSEC - The INTERSEC card allows the user to specify the lost time per phase as well as the time period being studied; for example, the period might range from 8.00 to 9.00 a.m.

VOLUME - The VOLUME card specifies the volumes for the through plus right and left-turn movements.

CAPACITY - The saturation flow rate for each movement, or the number of lanes in each direction, is specified on the CAPACITY card. (If the number of lanes is entered, the program assumes a default capacity per lane).

DIAL - The use of the DIAL card indicates that a dial is being assigned and therefore requires pretimed operation. One DIAL card must be used for each dial. The minimum and the maximum cycle lengths are specified in Fields 4 and 5 of the DIAL card. When evaluating existing conditions, the minimum cycle length and the maximum cycle length are both specified as equal to the existing cycle length.

LEFT - The LEFT card specifies the left-turn treatment. SOAP-84 options for left-turn treatment include restrictive, no protection, and permissive; they correspond to the terms protected, permitted and protected plus permitted, as discussed in Section 1.2.

RUN - The RUN card should follow all of the data input cards listed above. The RUN card initiates the analysis process.

END - The END card contains no parameters. It is a flag to signal the end of the card sequence.

Figure 4-6 illustrates the organization of a typical INTERSEC card required for SOAP-84. Figure 4-7 illustrates the deck (datafile) listing for a single-intersection, multi-period (SIMP) analysis.

The desired SOAP-84 output is specified using the DESIGN, REPORT AND TABLE card and the PLOT card. The report outputs available for single intersection analysis include the system MOEs, intersection-specific parameters, and a SOAP-84 left-turn check report, which specifies the left-turn volume and capacity for each approach. There are 27 SOAP-84 tables available, including tables of specified volumes, calculated volumes, minimum green times, average delay per vehicle, total delay per approach, excess fuel consumption, and SOAP MOE tables, as shown in Figure 4-8. The PLOT card can be used to specify SOAP MOE plots and/or a phasing diagram.

The requirements for data input for simulating existing conditions using PASSER II and SOAP-84 are summarized in Table 4-1. Note that with the exception of specifying the type of permitted left-turn model to be used in the analysis, both programs require essentially the same input data.

4.3 Calibration

After the necessary data has been coded, the analyst should run the program (PASSER II or SOAP) and check the output for error messages. In addition, one should check the program's input echo, including the cycle length, existing phase times, and phase sequence, to confirm that the information corresponds to existing conditions.

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 failing to represent 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 intersection. As a result, it is extremely important that the program's output accurately reflect existing operation. Otherwise, your results are meaningless, and any new signal timing plan that is developed will be less than optimal. **It is strongly recommends that no optimization be done until the analyst is satisfied that the program is properly calibrated.**

CARD NAME: INTERSEC (REQUIRED for SOAP) SINGLE PERIOD USAGE: 1 PER INTERSECTION MULTI-PERIOD USAGE: 1 PER RUN		PURPOSE: provides intersection specific parameters	
FIELD	**DESCRIPTION**	**RANGE**	**DEFAULT**
1	Blank.	-	-
2	Reference number for arterial data files only.	1-9999	REQ (MISP)
3	Beginning time, in military format, of the study period for intersection analyses only.	0-2400	REQ (SIMP)
4	Ending time, in military format, of the study period for intersection analyses only.	≥ begin + period length	REQ (SIMP)
5	Lost time per phase in seconds.	1-10	3.5
6	Cycle optimization step size, in seconds.	0.5-20	5
7	Stop penalty for the performance index.	5-50	30
8	Desired saturation level in percent.	1-99	95
9	Minimum improvement level in percent.	0.1-10	0.5
10-19	Blank.		

Figure 4-6. Organization of INTERSEC Card Required for SOAP

```
SETUP       0    2   44   15    0     0     0     0     0     0        FLORIDA & GATOR
INTERSEC         1  700 1800  3.5     0     0     0     0     0
---- NOTE ... DEFAULT VALUE (5) USED FOR CYCLE OPTMIZATION STEP SIZE.
---- NOTE ... DEFAULT VALUE (30) USED FOR STOP PENALTY.
---- NOTE ... DEFAULT VALUE (95) USED FOR DESIRED SATURATION LEVEL.
---- NOTE ... DEFAULT VALUE (0.5) USED FOR DESIRED IMPROVEMENT LEVEL.

CAPACITY11   700    2    1    2     1     2     1     2     1
DIAL      45  800    1   30  120     0     0     0     0     0
DIAL      60 1000    2   30  120     0     0     0     0     0
DIAL      30 1645    1   30  120     0     0     0     0     0
LEFT       Q    0    0    1    0     1     0     0     0   ONS    EW

VOLUME    15  700  200   30  225    40   120    25   190    45
VOLUME    15  715  225   35  210    35   100    20   175    30
VOLUME    15  730  230   45  230    40   120    25   195    50
VOLUME    15  745  225   40  225    40   125    25   200    50
VOLUME    15  800  250   35  250    35   100    30   175    55
VOLUME    15  815  240   35  240    35   110    30   185    30
VOLUME    15  830  225   35  225    40   120    25   190    45
VOLUME    15  845  200   30  210    35   100    20   175    50
VOLUME    15 1600  180   30  205    40   140    25   210    45
VOLUME    15 1615  205   35  190    35   120    20   185    30
VOLUME    15 1630  210   45  210    40   140    25   205    50
VOLUME    15 1645  205   40  205    40   145    25   220    50
VOLUME    15 1700  230   35  230    35   120    30   195    55
VOLUME    15 1715  220   35  220    35   130    30   205    30
VOLUME    15 1730  205   35  205    40   140    25   210    45
VOLUME    15 1745  180   30  190    35   120    20   195    50
VOLUME    60  900  600  100  600   100   400    50   700   100
VOLUME    60 1000  525  115  675    95   300    50   650    90
VOLUME    60 1400  585   80  555    90   375    75   525   105
VOLUME    60 1500  560  115  600   120   400   100   600   110

RUN        1                                                       EXAMPLE SIMP 1
```

Figure 4-7. Deck Listing for Single Intersection Analysis

M E A S U R E S   O F   E F F E C T I V E N E S S

MOVEMENTS:	DELAY (VEH-HRS)	STOPS (%)	EXC FUEL (GAL)	EXC LEFT (VEH)	MAXIMUM QUEUE	V/C RATIO
NB THRU :	41.10	89.9	75.93		23.7	0.97
LEFT :	7.48	83.6	12.68	0.0	4.2	0.80
SB THRU :	42.91	90.6	79.21		23.7	0.97
LEFT :	7.49	82.2	12.76	0.0	3.7	0.80
EB THRU :	22.30	84.3	42.26		11.2	0.63
LEFT :	6.22	92.2	9.96	0.0	2.8	0.64
WB THRU :	44.36	93.3	78.73		19.8	0.95
LEFT :	12.75	96.6	18.42	0.0	5.5	0.99
SUMMARY :	184.62	89.9	329.95	0.0	23.7	0.99

M E A S U R E S   O F   E F F E C T I V E N E S S

ANALYSIS: PERIOD :	DELAY (VEH-HRS)	STOPS (%)	EXC FUEL (GAL)	EXC LEFT (VEH)	MAXIMUM QUEUE	V/C RATIO
700- 715:	7.70	90.0	12.50	0.0	20.4	0.85
715- 730:	7.11	89.4	11.69	0.0	20.4	0.81
730- 745:	8.69	92.0	13.82	0.0	21.1	0.89
745- 800:	8.43	95.3	13.92	0.0	18.4	0.95
800- 815:	8.98	93.4	14.07	0.0	23.7	0.98
815- 830:	8.26	92.1	13.29	0.0	22.4	0.86
830- 845:	8.16	91.1	13.14	0.0	20.4	0.85
845- 900:	7.05	88.6	11.50	0.0	18.6	0.89
900- 915:	3.96	87.7	8.19	0.0	11.0	0.82
915- 930:	3.96	87.7	8.19	0.0	11.0	0.82
930- 945:	3.96	87.7	8.19	0.0	11.0	0.82
945-1000:	3.96	87.7	8.19	0.0	11.0	0.82
1000-1015:	3.66	87.1	7.64	0.0	10.1	0.76
1015-1030:	3.66	87.1	7.64	0.0	10.1	0.76
1030-1045:	3.66	87.1	7.64	0.0	10.1	0.76
1045-1100:	3.66	87.1	7.64	0.0	10.1	0.76
1400-1415:	3.41	85.6	7.16	0.0	8.4	0.67
1415-1430:	3.41	85.6	7.16	0.0	8.4	0.67
1430-1445:	3.41	85.6	7.16	C.0	8.4	0.67
1445-1500:	3.41	85.6	7.16	0.0	8.4	0.67
1500-1515:	3.82	86.4	7.92	0.0	9.0	0.71
1515-1530:	3.82	86.4	7.92	0.0	9.0	0.71
1530-1545:	3.82	86.4	7.92	0.0	9.0	0.71
1545-1600:	3.82	86.4	7.92	0.0	9.0	0.71
1600-1615:	7.15	92.6	12.39	0.0	16.9	0.91
1615-1630:	6.87	88.1	11.35	0.0	18.0	0.82
1630-1645:	7.99	94.2	13.51	0.0	16.7	0.90
1645-1700:	8.05	94.3	13.60	0.0	18.0	0.95
1700-1715:	8.59	95.6	14.05	0.0	18.9	0.99
1715-1730:	8.13	91.1	13.12	0.0	19.8	0.91
1730-1745:	7.62	93.7	13.05	0.0	16.9	0.91
1745-1800:	6.48	91.1	11.36	0.0	15.3	0.90
SUMMARY :	184.62	89.9	329.95	0.0	23.7	0.99

Figure 4-8. SOAP Output: MOE Table

Table 4-1. Data Input Requirements for SOAP-84 and PASSER II-90

INPUT	SOAP-84 (Card)	PASSER II-90 (Screen)
Intersection Description		
Location/Name	BEGIN	Arterial Data Input
Traffic Data		
Type of Control	CONTROL	Embedded Data
Cycle Length	CONTROL	Arterial Data
Volumes	VOLUME	Movement Input
Left Turn Protection	LEFT	Movement Input
Saturation Flow Rate	CAPACITY	Movement Input
Phase Sequence	SEQUENCE	Phase Sequence
System wide Parameters		
Lost Time	HEADWAY	Embedded Data
Sneakers	LEFT	Embedded Data
Left Turn Model	------	Embedded Data
Length of Analysis	BEGIN	Embedded Data

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

4.4 Output Interpretation

Runs were made using PASSER II-90 and SOAP-84 to analyze an isolated intersection at Texas Avenue and Southwest Parkway in College Station, Texas. The Highway Capacity Software was used as a comparison for this analysis. Analysts recorded the field data collected for the site on a worksheet from the *Highway Capacity Manual* (Figure 4-9) (4). Table 4-2 shows the results obtained from the three software packages. Note that the three programs calculate essentially the same values for volume to capacity ratio and average delay. Those differences that do exist result from differences in the second term of the delay equations that are being used. It also should be emphasized that just because the programs give slightly different answers does not mean that one program is more accurate than the others. That is, they are each providing estimates of expected conditions in the field and differences between programs are within acceptable measurement error limits for this type of data.

Both PASSER II-90 and SOAP-84 provide additional measures of effectiveness; these measures of effectiveness should be used to analyze the overall level of service at the intersection. The overall level of service for the intersection as well as the level of service for each movement should be considered. All are important, and the overall and individual movement's measures of effectiveness should both fall within acceptable limits. Note that the average of four good movements and four bad movements may result in an overall average that would be considered acceptable. Such operation, however, is not desirable.

The following terms are estimated by the two programs and used to describe the efficiency and quality of service experienced at signalized intersections.

Volume to Capacity Ratio. The volume to capacity ratio (v/c ratio) describes the degree of saturation (X). Section 2.5 discusses the calculations for volume to capacity ratio. A volume to capacity ratio greater than 1.0 indicates actual or potential breakdowns. If the overall volume to capacity ratio is less than 1.0, but the volume to capacity ratio for some movements is greater than 1.0, then it is likely that green times have not been proportioned appropriately. If the overall volume to capacity ratio is greater than 1.0 (realistically impossible), or equal to 1.0, some improvements to consider include:

1. Changing intersection geometry;
2. Lengthening the cycle length; and
3. Changing the signal phasing plan.

At the other extreme, if the overall volume to capacity ratio falls much below 0.75, the cycle length is probably too long and needs shortening.

INPUT WORKSHEET

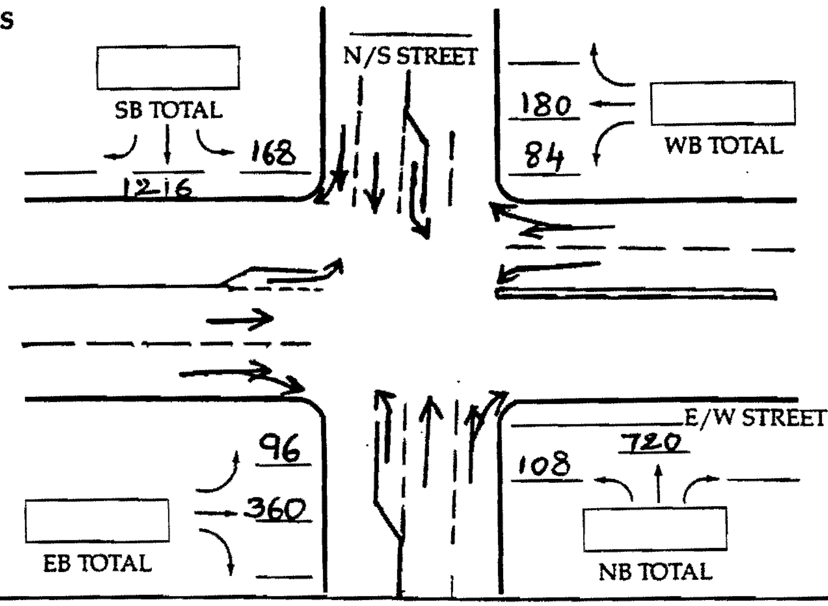
Intersection: Texas Ave. @ S.W. PKWY Date: 11-15-92

Analyst: S. SUNKARI Time Period Analyzed: 1600 Area Type: ☐ CBD ☑ Other

Project No.: ______ City/State: COLLEGE STATION

VOLUME AND GEOMETRICS

NORTH



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	0	N							3
WB	0	0	N							3
NB	0	5	N							3
SB	0	5	N							3

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = 18 Y + R = 4	G = 15 Y + R = 4	G = 22 Y + R = 4	G = 19 Y + R = 4	G = 21 Y + R = 4	G = Y + R =	G = Y + R =	G = Y + R =
Pretimed or Actuated								

Protected turns | Permitted turns | -------- Pedestrian | Cycle Length ____ Sec

Figure 4-9. HCM Worksheet for Example Intersection

Table 4-2. Comparison of Data Using Various Software Programs

	HCS			PASSER II			SOAP-84		
	v/c	delay (sec/veh)	LOS	v/c	delay (sec/veh)	LOS	v/c	delay (sec/veh)	LOS
NBT	.47	11.0	B	.47	11.2	B	.45	13.6	B
NBL	.50	23.0	C	.50	23.3	C	.50	31.3	C
SBT	.79	15.1	C	.79	15.3	C	.77	17.7	C
SBL	.77	32.6	D	.77	33.0	D	.79	43.9	D
EBT	.78	28.1	D	.78	28.4	D	.79	47.3	D
EBL	.66	30.6	D	.66	31.0	D	.69	43.5	D
WBT	.79	32.8	D	.78	33.2	D	.79	42.4	D
WBL	.58	27.4	D	.58	27.8	D	.60	39.4	D

Delay. PASSER II-90 and SOAP-84 both estimate total delay; however, the programs use slightly different equations for estimating delay. PASSER II uses the HCM delay equation for calculating total or stopped delay in units of seconds of delay per vehicle. The delay equation used in SOAP utilizes Webster's first term, but a modified version of Webster's second and third terms. SOAP reports delay in vehicle hours of delay per hour of operation.

Level-of-Service. For isolated intersections, LOS is based on the average amount of vehicle delay due to the traffic signal. These measures of delay correspond to levels of service as outlined in the Highway Capacity Manual and illustrated in Table 2-2 of Section 2.5 of these guidelines. To compare delay estimated by PASSER II-90 to delay estimated by SOAP-84, the units of delay used by SOAP (veh-hrs/hr) must be converted to seconds per vehicle as follows:

(veh-hrs/hr) * (3600 sec/hr) / (number of vehicles in an hour) = sec/veh

Queue Lengths. Queue lengths refer to the number of vehicles accumulated at the intersection during the red intervals. PASSER II-90 reports queue lengths in vehicles per lane while SOAP-84 predicts the total number of vehicles accumulated. To compare PASSER with SOAP, one should divide the value reported by SOAP by the number of lanes for that movement. Long queues may be a problem for intersections with short left-turn lanes.

Stops. Stops refers to the number of vehicles stopped per hour. PASSER II-90 uses a modified version of the Akcelik and Miller formula to estimate the total number of stops. SOAP-84 defines percent vehicles stopped as the number of vehicles in the queue at the beginning of the green, plus the number of vehicles which join the queue while the queue is still discharging, divided by the average number of arrivals per cycle.

Fuel Consumption. Both PASSER II-90 and SOAP-84 estimate the fuel consumption for each hour the signal timing plan operates. SOAP estimates fuel consumption from the percent stops calculated above. The calculation of fuel consumption is based on the volume of cars in an hour and the percent of those cars that stop.

Total Intersection Delay. The total intersection delay equals the sum of the total delay at each approach divided by the total volume of all approaches. PASSER II-90 reports average total intersection delay in seconds per vehicle and SOAP-84 estimates total intersection delay in vehicle hours per hour.

Minimum Delay Cycle. The minimum delay cycle is the cycle length that produces minimum intersection delay, as calculated using Webster's minimum delay equation. The minimum delay cycle given by PASSER II-90 is calculated from minimum green times based on pedestrian requirements and other constraints input by the user. If absolute minimum green times based solely on critical movement analysis were used as input, the reported minimum delay cycle should equal Webster's minimum delay cycle. When evaluating existing conditions, however, the existing cycle length specified by the sum of the phase splits that were input will be reported by PASSER as the minimum delay cycle length. This value is not the actual minimum delay cycle but rather a constrained cycle length based on the data input to the program.

Excess Left Turns. Excess left-turns is a measure of effectiveness reported by SOAP-84; it expresses the number of left-turning vehicles that cannot be accommodated by the protected turning interval or through natural gaps in the oncoming traffic during a permitted phase. This measure of effectiveness determines if the existing or proposed operations provide adequate opportunity for the left-turn movements.

5.0 OPTIMIZATION

After running a simulation of the existing conditions and checking input and output for accuracy, further computer runs can be used to optimize the timing plan. These runs can find cycle lengths, green splits, and phase sequences which will minimize delay, stops, and fuel consumption, or increase capacity. The "best solution," however, depends on what the traffic signal analyst is attempting to accomplish. Comparisons should be made between the best solution for the optimized runs and the evaluation of existing conditions to quantify improvement.

PASSER II-90 or SOAP-84 can both be used to make optimization runs. Most data needed for the optimization runs has already been coded for the simulation of existing conditions run. The parameters that will differ are changed by editing the data. The data base may be edited in PASSER by accessing the EDIT screen, a menu which allows the user to choose which data needs editing. To edit data in SOAP, the user "finds" the card to be edited with the FIND COMMAND and then edits that card using the EDIT COMMAND.

The following sections discuss procedures and guidelines for optimizing signal timing plans using PASSER II and SOAP. As already mentioned, the user will have previously entered most of the data required for analysis. These data should have been checked for accuracy and calibrated for local conditions. The other data will be edited depending on the type of optimization to be performed.

5.1 Editing the Data

Table 5-1 summarizes the data likely to be edited and the screen or card corresponding to this data. Further discussion of these parameters is provided in the following text.

Cycle Length. The existing cycle length may be too short or too long. The cycle length should be long enough to provide the required capacity, but short enough to minimize delay and wasted green time. A compromise may be needed to provide both an acceptable level of service and an acceptable volume to capacity ratio.

Table 5-1. Data to be Edited for Optimization

DATA	PASSER II (screen)	SOAP (card)
Cycle Length	Edit Arterial Data	CONTROL
Left Turn Protection	Edit Movement Data	LEFT
Minimum Phase Time	Edit Movement Data	MINGREEN*
Phase Sequence	Edit Phase Sequence	SEQUENCE

* MINGREEN card not used in the evaluation of existing conditions

Generally the minimum delay cycle length is the optimum cycle length. If fuel consumption or some other measure is important, however, some other cycle length may be optimum. PASSER II provides an estimate of the minimum delay cycle length, if the minimum greens add up to less than the coded cycle length. Because PASSER II and SOAP can only evaluate one cycle length at a time, multiple runs may have to be made to determine the minimum delay cycle length.

To edit the cycle length information when using PASSER II-90, the user must edit the *Arterial Data* screen; when using SOAP-84, the user must modify the CONTROL card. Figure 5-1 and Figure 5-2 illustrate the variation in delay and fuel consumption with changes in cycle length using PASSER II-90 and SOAP-84 respectively. These two figures illustrate that the trends for the variation in delay and fuel consumption with changing cycle length are similar for both programs. Cycle lengths that are too short or too long result in unnecessary delay and fuel consumption. Both programs indicate a cycle length for minimum delay of about 70 seconds.

Figure 5-1 and Figure 5-2 also indicate a cycle length of 70 to 75 seconds for minimum fuel consumption. The cycle length for minimum fuel consumption is generally slightly longer than the cycle length for minimum delay, since, fuel consumption is a function of delay as well as stops at an intersection. Increasing the cycle length causes the number of stops to decrease; i.e., fewer stops of longer duration occur. The decrease in stops tends to shift the cycle length for minimum fuel consumption away from the cycle length for minimum delay; however, since delay influences fuel consumption much more than stops, the increase in delay at higher cycle length causes an increase in fuel consumption that counteracts the decrease in fuel consumption because of fewer stops.

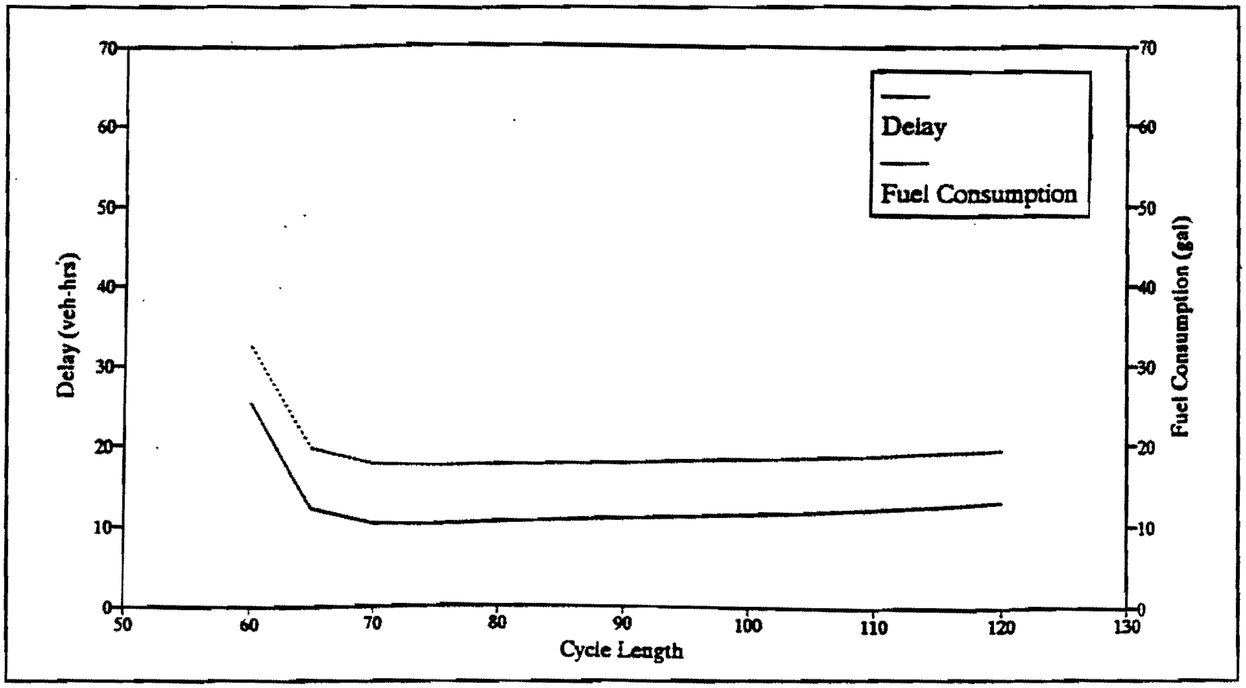


Figure 5-1. Variation in Delay and Fuel Consumption with Change in Cycle Length Using PASSER II-90

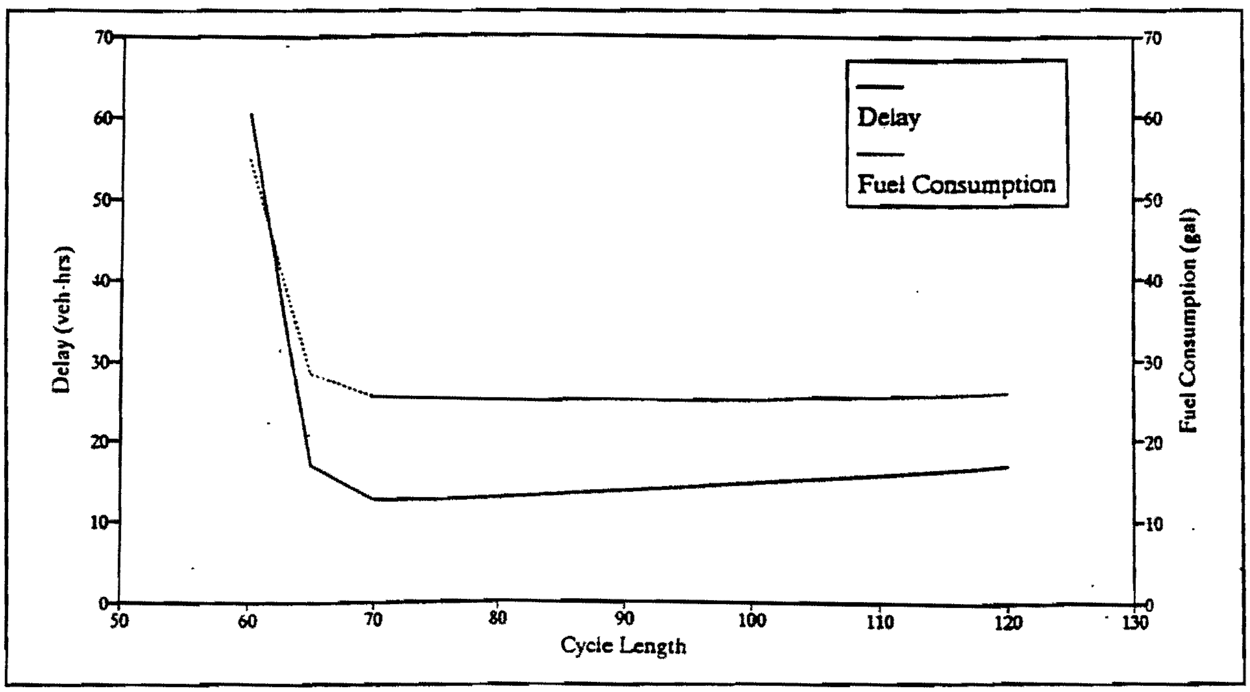


Figure 5-2. Variation in Delay and Fuel Consumption with Change in Cycle Length Using SOAP-84

Left-Turn Protection. One strategy for improving intersection capacity and operating efficiency is to allow permissive left-turns; however, it should be noted that the use of permissive left-turns is not recommended for all circumstances. For example, permissive left-turns may be inappropriate where a potential for accidents exists, where an opposing through movement has two or more lanes, and where no left-turn bay exists. If no left-turn bay exists, through movements may be blocked by vehicles waiting to turn left. Additionally, some jurisdictions have policies that do not allow permissive turns under any circumstances. Engineering judgement and site studies should be used as a basis for implementing permissive left turns.

The user makes changes in the type of left-turn protection by editing the movement data for left turns and the LEFT CARD in PASSER II-90 and SOAP-84, respectively. Multiple runs will have to be made for the different alternatives as well as a comparison of the resultant output. For a discussion of guidelines for selecting the appropriate type of left-turn treatment, see Section 2.2.

Minimum Green (Phase) Times. For pretimed and most actuated timing plans, the minimum green time for the through movements should consider pedestrians by allowing enough time for pedestrians to cross the approach. The time for pedestrians to cross an intersection is based on the width of the intersection, as well as the pedestrian walking speed, which will be slower for small children or senior citizens.

Actuated controllers, in addition to considering pedestrian constraints, must consider minimum green times to clear vehicles stored between the stop line and the detector; this is termed the detector minimum green time. The minimum green time used by the program is either the pedestrian or the detector minimum green time, whichever is larger. Both PASSER II-90 and SOAP-84 use a minimum phase time default value of 15 seconds for through movements, and 10 seconds for protected left-turn movements. These defaults include clearance times. Thus, the default green times are three to four seconds less than these values. If needed, however, these defaults can be changed to lower values.

Phase Sequence. A phasing sequence which allows permissive left-turns or overlap phasing may improve capacity. The order of the phases themselves will not change the capacity; however, the order of the protected left-turns (lagging or leading) may be critical depending upon the geometry of the intersection, specifically the presence or absence of a left-turn lane. The phase sequence selection also must consider the constraints of the existing signal hardware and intersection geometry.

The user accomplishes phase sequence selection in PASSER II-90 by editing the *Phase Sequence* screen. Only one selection should be selected for both main and cross street. If the user inputs multiple choices for the main street, the program will only consider the first choice entered. A single dash (-) indicates a phase sequence that is possible but has not been selected by the user. Three dashes (---) indicate that the phase sequence is not possible. The program determines whether or not a phase is possible by traffic and geometric data entered previously. For example, permissive left-turn phases will cause phases with protected turns to be deleted from the list of allowable options. The *Special Phasing* option reinstates a portion of the deleted phase options.

In SOAP-84, the user edits the SEQUENCE CARD to make changes to the phase sequence. Like PASSER II-90, only one sequence may be analyzed per run. The proposed phase sequence for analysis must correlate with information provided by the user on the LEFT CARD, or the program generates an error message.

5.2 Optimization

After the necessary input data has been edited, the program is ready to run. The user should view the output for the optimized run and compare the measures of effectiveness for the optimized run with the measures of effectiveness from the existing conditions run. It may prove necessary to run the program several times to find the cycle length which produces the desired results: low delay, acceptable volume to capacity ratios, or a combination of the two. Additional runs may be necessary for both PASSER II-90 and SOAP-84 to compare the effects of different phase sequences and left-turn treatments.

As an example, Table 5-2 and Table 5-3 summarize and compare the results of several optimization runs using both PASSER II-90 and SOAP-84. As shown, a cycle range of 70 to 85 seconds was found to be optimal for the various phasing sequences and types of control under consideration.

For pretimed control strategies, permitted plus protected lefts and dual lefts leading for both streets produced minimum intersection delay and acceptable volume to capacity ratios for the individual movements. Left-turn Number 5 leading with overlap for Texas Avenue and dual lefts leading for Southwest Parkway produced the "best" results when left-turn movements were protected only.

Analysis of the intersection assuming actuated control with protected lefts only produced a lower estimate of total intersection delay than pretimed control with protected lefts. Pretimed control with protected plus permissive dual lefts leading, however, produced a lower estimate of the total intersection delay when compared to actuated control.

As discussed previously, PASSER II-90 and SOAP-84 produced similar results for total intersection delay and most individual movement measures of effectiveness. Some discrepancies arose with the SBL and SBT movements, such as maximum queue and degree of saturation (volume to capacity). These discrepancies are due to the differences in the delay, queue, and/or stop equations used by the two programs.

Table 5-2. Comparison of PASSER II-90 and SOAP-84: Pretimed Control

	Protected Lefts Only	
	PASSER II-90	SOAP-84
Phase sequence on Texas	#5 left leads with overlap	STN with overlap
Phase sequence on SW Parkway	dual lefts lead	dual lefts lead
Cycle length	80 seconds	80 seconds
Total Intersection Delay	27.5 sec/veh	26.2 sec/veh
Largest queue	7.3 veh/lane	11.1 veh/lane
Largest v/c ratio	SBT - 0.82	SBT - 0.76
Largest Delay	EBL - 54.5 sec/veh	NBL - 55.0 sec/veh

	Protected plus Permitted Lefts	
	PASSER II-90	SOAP-84
Phase sequence on Texas	dual lefts lead	dual lefts lead
Phase sequence on SW Parkway	dual lefts lead	dual lefts lead
Cycle length	80 seconds	85 seconds
Total Intersection Delay	22.2 sec/veh	22.5 sec/veh
Largest queue	6.9 veh/lane	11.1 veh/lane
Largest v/c ratio	SBT - 0.79	SBT - 0.72
Largest Delay	WBT - 36.6 sec/veh	WBT - 38.6 sec/veh

Table 5-3. Comparison of PASSER II-90 and SOAP-84: Actuated Control

	Protected Lefts Only	
	PASSER II-90	SOAP-84
Phase sequence on Texas	#5 left leads with overlap	STN with overlap
Phase sequence on SW Parkway	dual lefts lead	dual lefts lead
Cycle length	80 seconds	70 seconds
Total Intersection Delay	23.4 sec/veh	23.8 sec/veh
Largest queue	6.0 veh/lane	11.0 veh/lane
Largest v/c ratio	SBT - 0.82	SBT - 0.95
Largest Delay	EBL - 46.4 sec/veh	SBL - 37.7 sec/veh

5.3 Example Problem

The intersection of University Drive and South College in College Station was selected as an example intersection to identify its existing problems and illustrate the expected improvement in traffic performance when implementing various signal retiming and geometric modifications. Figure 5-3 presents the HCM worksheet showing the existing conditions. Five cases (or scenarios) will be presented. The first case simulates the existing conditions and identifies the problems associated with the intersection. Then a series of modifications involving signal retiming and intersection geometry changes will be evaluated. The benefits associated with each case will be identified.

CASE 1: Simulate Existing Conditions.

PASSER II-90 and SOAP-84 runs were made to simulate the existing conditions at the intersection of University Drive and South College. Figure 5-4 shows the PASSER II-90 *Movement Data* screen, illustrating the intersection geometry, traffic volumes, saturation flow rates, and phase timings representative of existing conditions at the intersection.

Figure 5-5 shows the resultant measures of effectiveness for the existing conditions. From a visual examination of the measures of effectiveness, the volume to capacity ratio for the east bound through and north bound left-turn movements is very high (1.26). These movements operate at a level of service F. Thus, these two movements can be identified as the critical movements needing improvement. Note that some movements have high volume to capacity ratios, whereas other movements have lower volume to capacity ratios. This imbalance indicates that the green splits may not be optimum, and their reallocation might improve the intersection's operation.

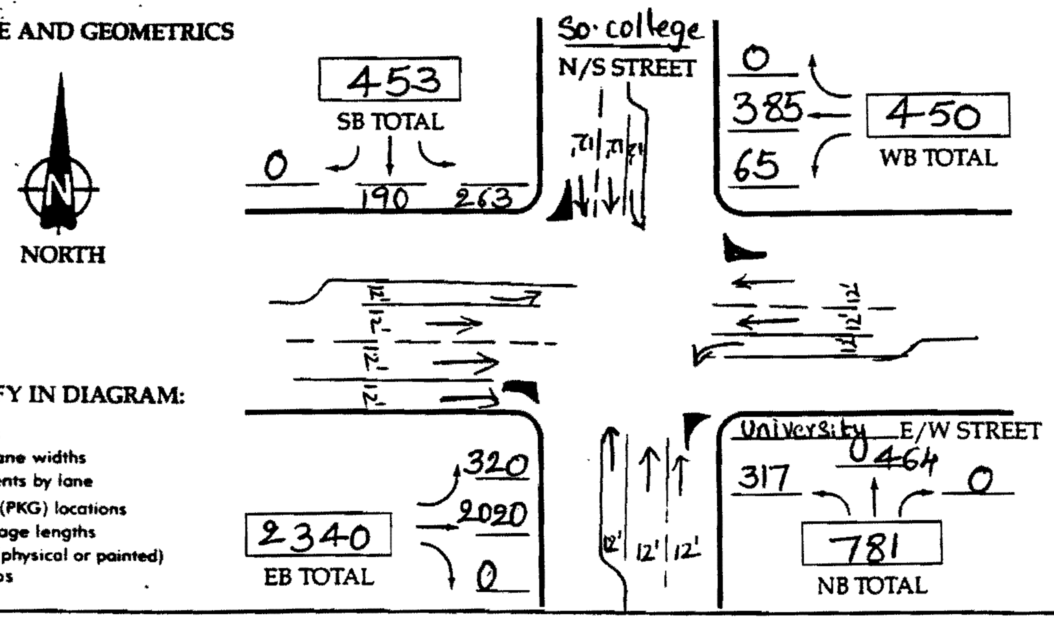
INPUT WORKSHEET

Intersection: University Dr @ So. College/Bizzell Date: 11-15-92

Analyst: S. Sunkari Time Period Analyzed: P. M Area Type: ☐ CBD ☑ Other

Project No.: City/State: college station, Tx

VOLUME AND GEOMETRICS



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

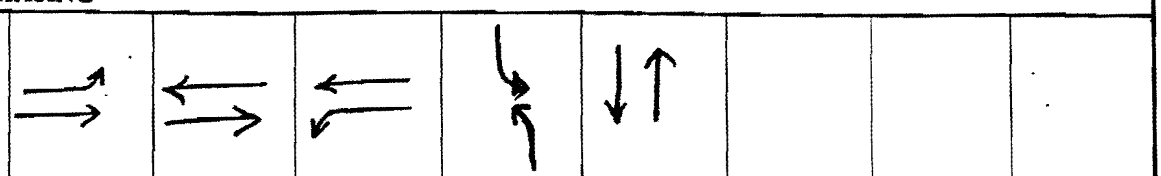
TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	10	N			1		N		3
WB	0	10	N			1		N		3
NB	0	5	N			1		N		3
SB	0	5	N			1		N		3

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

D I A G R A M



Timing	G = 29, Y+R = 5	G = 27, Y+R = 5	G = 10, Y+R = 5	G = 18, Y+R = 5	G = 17, Y+R = 5	G = , Y+R =	G = , Y+R =	G = , Y+R =
Pretimed or Actuated								

Protected turns | Permitted turns | -------- Pedestrian | Cycle Length ____ Sec

Figure 5-3. HCM Worksheet for the Example Intersection

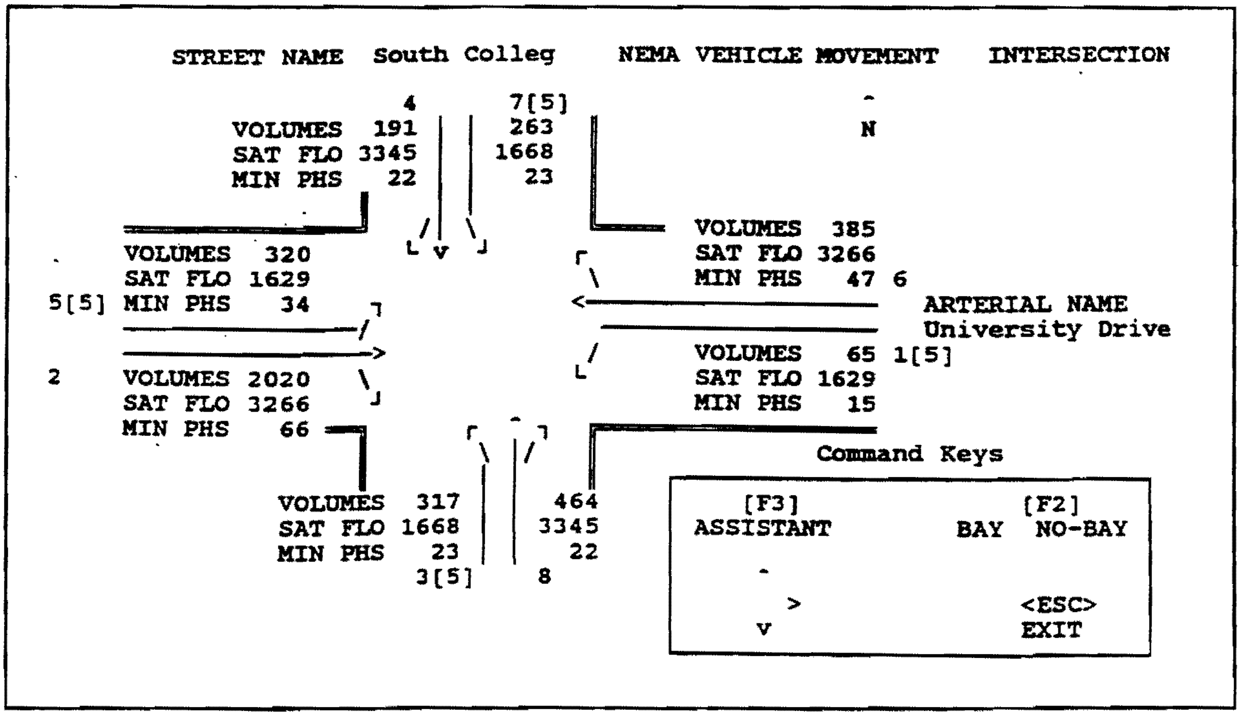


Figure 5-4. PASSER II-90 Screen Illustrating the Intersection Layout

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                    TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
PASSER II-90     MULTIPHASE ARTERIAL PROGRESSION - 145101      VER 1.0  DEC 90

                       **** BEST PROGRESSION SOLUTION SUMMARY ****
*** INT.    1                     ART   ST PHASE SEQ IS LT 5 LEADS   (2+5)
                                  CROSS ST PHASE SEQ IS DUAL LEFTS   (3+7)

                          ARTERIAL STREET                  CROSS STREET
CONCURRENT PHASES     2+5    2+6    1+6    TOTAL      3+7   3+8   4+8     TOTAL
PHASE TIME (SECS)    34.0   32.0   15.0     81.0     23.0    .0  22.0      45.0
PHASE TIME (%)       27.0   25.4   11.9     64.3     18.3    .0  17.5      35.7
                   -------------- MEASURES OF EFFECTIVENESS ---------------
PHASE (NEMA)          5[5]   6      1[5]   2          3[5]   4      7[5]   8
PHASE DIRECTION     EBLTPR WBTHRU WBLTPR EBTHRU    NBLTPR SBTHRU SBLTPR NBTHRU
PHASE TIME (SEC)     34.0   47.0   15.0   66.0       23.0   22.0   23.0   22.0
*V/C-RATIO            .83    .35    .46   1.26       1.26    .40   1.05    .97
LEVEL OF SERVICE       D      A      A      F          F      A      F      E
DELAY (SECS/VEH)     57.8   31.1   56.7  215.8      269.6   49.5  130.6   86.8
LEVEL OF SERVICE       E      C      E      F          F      D      F      F
QUEUE (VEH/LANE)     11.3    4.7    2.3  103.1       40.8    3.0   19.0   11.5
STOPS (STOPS/HR)     308.   347.    58.  4011.       703.   173.   374.   530.
TOTAL INTERSECTION DELAY         FUEL CONSUMPTION         MINIMUM DELAY CYCLE
   158.92 SECS/VEH                156.52 GAL/HR                > 120 SECS
```

Figure 5-5. PASSER II-90 Results of the Measures of Effectiveness for CASE 1

CASE 2: Optimizing the green splits.

The green splits for the intersection of University Drive and South College were optimized for the existing cycle length of 126 seconds. Analysis of this alternative was accomplished by changing the minimum phase times from existing splits to realistic minimum phases. The phasing sequence remained unchanged. The existing phasing sequence was east bound left leading with overlap (lead-lag) on University Drive and dual lefts leading on South College.

Figure 5-6 illustrates the measures of effectiveness generated by PASSER II-90 after optimizing the green splits for the intersection. Based on the measures of effectiveness table, the volume to capacity ratio for the east bound through and north bound left turn movements decreased to 1.17 and 1.12 respectively. Also, the total intersection delay has decreased from 159 seconds per vehicle to 115 seconds per vehicle. Note that the volume to capacity ratios are more balanced than for the existing conditions; however, volume to capacity ratios and delays for several movements, especially the east bound and north bound movements, are still unacceptable.

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                      **** BEST PROGRESSION SOLUTION SUMMARY ****
*** INT.    1                     ART   ST PHASE SEQ IS LT 5 LEADS   (2+5)
                                  CROSS ST PHASE SEQ IS DUAL LEFTS   (3+7)

                        ARTERIAL STREET                  CROSS STREET
CONCURRENT PHASES    2+5   2+6   1+6    TOTAL     3+7   3+8   4+8    TOTAL
PHASE TIME (SECS)   49.3  21.4  10.0    80.7     25.4    .0  19.9     45.3
PHASE TIME (%)      39.1  17.0   7.9    64.0     20.2    .0  15.8     36.0
                   ------------- MEASURES OF EFFECTIVENESS --------------
PHASE (NEMA)         5[5]   6     1[5]   2        3[5]   4      7[5]   8
PHASE DIRECTION    EBLTPR WBTHRU WBLTPR EBTHRU  NBLTPR SBTHRU SBLTPR NBTHRU
PHASE TIME (SEC)    49.3  31.4   10.0   70.7     25.4   19.9   25.4   19.9
V/C-RATIO            .55   .54    .84   1.17     1.12    .45    .93   1.10
LEVEL OF SERVICE     A     A      D      F        F      A      E      F
DELAY (SECS/VEH)    33.2  44.6  106.1  137.2    166.1   51.7   84.4  142.8
LEVEL OF SERVICE     D     D      F      F        F      D      F      F
QUEUE (VEH/LANE)     7.7   5.6    4.0   70.7     27.6    3.0   12.8   17.8
STOPS (STOPS/HR)    234.  351.    84.  3212.     513.   174.   300.   685.
TOTAL INTERSECTION DELAY     FUEL CONSUMPTION       MINIMUM DELAY CYCLE
   114.96 SECS/VEH             117.86 GAL/HR           > 120 SECS
```

Figure 5-6. PASSER II-90 Results of the Measures of Effectiveness for CASE 2

CASE 3: Adding a through lane for the east and west approaches on University Drive.

In the two earlier cases, the east bound approach experienced an extremely high volume to capacity ratio and average delay. Thus, one improvement might be the addition of a through lane to the east and west bound approaches on University Drive which would result in three lanes for through movements on these approaches. Analysis of this alternative was accomplished by increasing the east bound and west bound saturation flow rate (increasing the number of east bound and west bound through lanes) on the intersection movement data screen. As in Case 2, the green splits were optimized for the given traffic conditions for a cycle length of 126 seconds and the existing phase sequence.

Figure 5-7 illustrates the measures of effectiveness generated by PASSER II-90 after an additional through lane was added to the east and west bound approaches on University Drive. Note that the traffic conditions at the intersection show significant improvement. The additional lane significantly increased the capacity of these two approaches and the volume to capacity ratios have decreased below 1.0 for all approaches. The total intersection delay has decreased to 49 seconds per vehicle; however, the average delay for several movements is still unacceptable, indicating that further improvements may be needed.

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                     **** BEST PROGRESSION SOLUTION SUMMARY ****
*** INT.    1                     ART   ST PHASE SEQ IS LT 5 LEADS   (2+5)
                                  CROSS ST PHASE SEQ IS DUAL LEFTS   (3+7)

                         ARTERIAL STREET                  CROSS STREET
CONCURRENT PHASES   2+5    2+6    1+6    TOTAL     3+7    3+8    4+8    TOTAL
PHASE TIME (SECS)   49.0   13.4   10.0    72.4     30.2    .0   23.4     53.6
PHASE TIME (%)      38.9   10.6    7.9    57.5     24.0    .0   18.6     42.5
                    -------------- MEASURES OF EFFECTIVENESS ---------------
PHASE (NEMA)        5[5]    6     1[5]     2       3[5]    4     7[5]     8
PHASE DIRECTION    EBLTPR WBTHRU WBLTPR EBTHRU   NBLTPR SBTHRU SBLTPR NBTHRU
PHASE TIME (SEC)    49.0   23.4   10.0   62.4     30.2   23.4   30.2   23.4
V/C-RATIO            .55    .54    .84    .93      .92    .37    .76    .90
LEVEL OF SERVICE     A      A      D      E        E      A      C      E
DELAY (SECS/VEH)    33.5   49.9  106.1   39.6     74.9   48.1   55.3   69.8
LEVEL OF SERVICE     D      D      F      D        E      D      E      E
QUEUE (VEH/LANE)     7.8    4.2    4.0   17.0     13.7    2.9    8.7    9.3
STOPS (STOPS/HR)    235.   351.    84.  1936.     344.   173.   244.   482.
TOTAL INTERSECTION DELAY       FUEL CONSUMPTION          MINIMUM DELAY CYCLE
    48.87 SECS/VEH                58.23 GAL/HR                > 120 SECS
```

Figure 5-7. PASSER II-90 Results of the Measures of Effectiveness for CASE 3

CASE 4: Adding a left turn lane for the north and south approaches on South College.

In CASE 1 and CASE 2, the north bound left-turn movement experienced high volume to capacity ratios and long delays. In CASE 3, the volume to capacity ratio for the north bound left-turn movement was reduced to less than 1.0 as the green time taken from the east-west movements (when a lane was added to those approaches) was added to the north-south movements. The addition of a second left turn lane to the north-south approaches, however, is likely to further improve the traffic conditions at the intersection.

Because dual left-turn lanes are being considered, dual lefts leading phasing for the north and south bound approaches on South College should probably be changed to lead-lag phasing. Thus, the analysts selected a lead-lag phasing pattern (the north bound movements leading the south bound movements) without overlap (split phasing) and determined that the minimum delay cycle length was 96 seconds. Analysis of this alternative was accomplished by changing the cycle length from 126 to 100 seconds, adding a north and south bound left-turn lane, and changing the cross street phase sequence.

Figure 5-8 illustrates the measures of effectiveness generated by PASSER II-90 after the input data was changed. Note that a further decrease in the volume to capacity ratios for all the movements occurred as there has been a transfer of green time from the north-south approaches to the east-west approaches. The overall intersection delay has decreased to about 33 seconds per vehicle, and measures of effectiveness for the intersection and each of the individual movements are probably acceptable.

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**** BEST PROGRESSION SOLUTION SUMMARY ****
*** INT. 1 ART ST PHASE SEQ IS LT 5 LEADS (2+5)
CROSS ST PHASE SEQ IS LT 3 LEADS (3+8)

	ARTERIAL STREET				CROSS STREET			
CONCURRENT PHASES	2+5	2+6	1+6	TOTAL	3+8	4+8	4+7	TOTAL
PHASE TIME (SECS)	42.0	10.4	10.0	62.4	20.2	.0	14.4	34.6
PHASE TIME (%)	43.3	10.7	10.3	64.3	20.8	.0	14.8	35.7

-------------- MEASURES OF EFFECTIVENESS ---------------

PHASE (NEMA)	5[5]	6	1[5]	2	3[5]	4	7[5]	8
PHASE DIRECTION	EBLTPR	WBTHRU	WBLTPR	EBTHRU	NBLTPR	SBTHRU	SBLTPR	NBTHRU
PHASE TIME (SEC)	42.0	20.4	10.0	52.4	20.2	14.5	14.4	20.3
V/C-RATIO	.50	.49	.65	.87	.62	.53	.80	.83
LEVEL OF SERVICE	A	A	B	E	B	A	C	D
DELAY (SECS/VEH)	23.1	37.0	56.4	25.1	39.6	42.4	53.6	47.8
LEVEL OF SERVICE	C	D	E	C	D	D	E	D
QUEUE (VEH/LANE)	5.7	3.2	2.2	11.4	4.2	2.4	4.4	6.5
STOPS (STOPS/HR)	223.	350.	69.	1893.	275.	177.	266.	464.

TOTAL INTERSECTION DELAY FUEL CONSUMPTION MINIMUM DELAY CYCLE
33.00 SECS/VEH 44.72 GAL/HR 96 SECS

Figure 5-8. PASSER II-90 Results of the Measures of Effectiveness for CASE 4

CASE 5: Implementing protected-permitted phasing on east-west approaches on University Drive.

As mentioned earlier, left-turn capacity can be increased and left-turn delay can be decreased by allowing permissive left turns under low and moderate volume conditions. To illustrate the effects of this alternative, the analysts implemented protected-permissive phasing on the east-west approaches on University Drive. To implement this alternative, the city installed new five section signal heads, and maintained the cycle length at 100 seconds. This alternative was not considered for College Avenue because of the presence of the dual left-turn lanes. Analysis of this alternative was accomplished by changing the left-turn treatment code on the *Intersection Movement Data* screen from protected only to protected plus permitted phasing. It should be emphasized that the user must make the determination whether protected-permitted phasing is appropriate; **the program has no way of knowing what is unsafe or counter to standard practice.**

Figure 5-8 illustrates the measures of effectiveness generated by PASSER II-90 after allowing protected-permitted phasing. Note that there has been a significant improvement in the delay associated with the left-turn movements on University Drive. The east bound as well as west bound left-turn delays have decreased. The overall intersection delay has decreased to 32 seconds per vehicle. It should be noted that the combination of all five alternatives represent the current signalization and geometric conditions at this intersection.

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                     **** BEST PROGRESSION SOLUTION SUMMARY ****
*** INT.    1                     ART   ST PHASE SEQ IS LT 5 LEADS   (2+5)
                                  CROSS ST PHASE SEQ IS LT 3 LEADS   (3+8)

                         ARTERIAL STREET                 CROSS STREET
CONCURRENT PHASES    2+5    2+6    1+6    TOTAL     3+8    4+8    4+7    TOTAL
PHASE TIME (SECS)   36.9   14.7   10.0    61.6     20.1     .0   14.3    34.4
PHASE TIME (%)      38.4   15.3   10.4    64.2     20.9     .0   14.9    35.8
                    -------------- MEASURES OF EFFECTIVENESS ---------------
PHASE (NEMA)        5[6]   6      1[6]   2         3[5]   4      7[5]   8
PHASE DIRECTION     EBLTPP WBTHRU WBLTPP EBTHRU    NBLTPR SBTHRU SBLTPR NBTHRU
PHASE TIME (SEC)    36.9   24.7   10.0   51.6      20.1   14.4   14.3   20.2
V/C-RATIO            .51    .38    .40    .87       .61    .53    .80    .82
LEVEL OF SERVICE     A      A      A      E         B      A      C      D
DELAY (SECS/VEH)    11.4   32.3   39.4   25.3      39.1   41.9   53.2   47.1
LEVEL OF SERVICE     B      C      D      C         D      D      E      D
QUEUE (VEH/LANE)     3.8    3.0    1.5   11.5       4.1    2.4    4.4    6.4
STOPS (STOPS/HR)    223.   348.    60.  1898.      274.   177.   266.   463.
TOTAL INTERSECTION DELAY       FUEL CONSUMPTION          MINIMUM DELAY CYCLE
   31.32 SECS/VEH                43.33 GAL/HR                  96 SECS
```

Figure 5-9. PASSER II-90 Results of the Measures of Effectiveness for CASE 5

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6.0 IMPLEMENTATION

The next step in the retiming of a traffic signal is implementation of the improved timing plan. After a "best solution" has been determined using SOAP-84 or PASSER II-90, the results should be transferred to a controller worksheet for use in the field. This text does not address all entries to the controller sheet, rather only those entries directly related to the computer output. Note that electromechanical pretimed and actuated control require different controller data sheets, worksheets may vary with the brand of controller, or a self-made worksheet may be used. These guidelines address the current TxDOT controller standard specifications as much as possible.

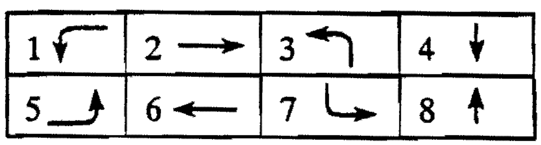
6.1 Terminology

Before discussing an implementation procedure, some definitions of the terminology commonly used in traffic signal timing must be given (16).

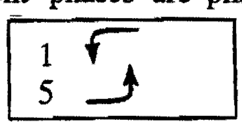
Split - The portion of the cycle length allocated to each of the various phases, expressed in seconds or as a percent of the entire cycle length.

Interval - A discrete portion of the signal cycle during which the signal indications do not change, including the green, yellow, and all red clearance intervals per movement.

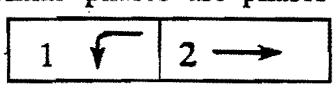
Phase - Individual movement; for example, at a typical intersection, eight movements are usually given some green time.



Concurrent phases are phases that are timed together:



Sequential phases are phases that follow one another:



Phase Reversal -	For an eight phase dual ring controller (see Figure 2-3), phases 1 and 2 could be reversed (change from leading left-turn to lagging left-turn). The existing 1 + 5 phase (dual left leading) becomes 2 + 5 (lead-lag phasing).
Yellow Clearance -	Time to clear each phase based on the traffic speed.
Red Clearance -	Additional red time needed for clearing the intersection, dependent on the intersection width.

The following terms relate to actuated control:

Minimum Green -	This parameter is the minimum green time for each phase and is the larger of pedestrian times and detector placement requirements.
Initial Gap -	Also known as the passage time, extension, or preset gap. The initial gap is added to the minimum green as a vehicle is detected. The time is based on speed and the detector location.
Maximum Green -	The total phase time minus any yellow and red clearance time (MAX I).
MAX II -	The total phase time minus any yellow and red clearance time (MAX I when externally activated).
Recall -	A phase may be set on recall to allow for constant detection. The phase will be given green time whether a vehicle is there or not.

6.2 Signal Settings for Each Intersection

A controller pin setting report, or "*PIN.SET*" report (Figure 6-1), is presented for the PASSER *Best Signal-Timing Solution*. Note that the *PIN.SET* report is presented in phase intervals; these can be used directly by future delay-based programs. The controller pin-setting report is designed for direct implementation of the PASSER *Best Signal Timing Solutions* on quad-left, microcomputer-based traffic signal controllers.

The names and directions of entry are indicated as specified in the input data. Controllers use the phase numbers 1, 3, 5, and 7 for protected left-turn movement volumes only; "protection" must be provided by a separate left-turn lane or bay and by a protected left-turn signal phase. Left-turn movements not protected by an exclusive lane and phase are combined in the same phase with the adjacent through movements.

```
(PIN.SET)
                TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
PASSER II-90     MULTIPHASE ARTERIAL PROGRESSION - 145101      VER 1.0  DEC 90

       **** SUMMARY OF PASSER II-90 BEST SIGNAL TIMING SOLUTION ****
College Stat  University Drive              DISTRICT  1 11/29/92    RUN NO.  1

CYCLE =  96. SECONDS             SPLIT = 1 2 3.           OFFSET = 1 2 3.

DEFAULT(1) : SAME MASTER & SYS INT, OFFSET TO BEGINNING OF MAIN STREET GREEN
MAST INT =  1 SYS INT =  1 SYS OFFSET =    .0 REF MOVMNT = 0 REF PNT = BEGIN

INTRSC  1 : South Colleg       COORD PHASE : 0 OFFSET :     .0    SEC :    .0%
*-[ISOLATED OPERATION]
DUAL-RING PHASE #      5      6      1      2      3      4      7      8
PHASE SPLIT (SEC)     36.9   24.7   10.0   51.6   20.1   14.4   14.3   20.2
PHASE SPLIT (%)       38.%   26.%   10.%   54.%   21.%   15.%   15.%   21.%
PHASE REVERSAL         --     --     2      1      --     --     8      7
LEFT TURN             LEAD    --    LAG     --    LEAD    --    LAG     --

CONCURRENT PHASES      2+5    2+6    1+6    3+8    4+8    4+7   MAIN  CROSS
DURATION    (SEC)     36.9   14.7   10.0   20.1     .0   14.3   61.6   34.4
CYCLE COUNT (SEC)       .0   36.9   51.6   61.6   81.7   81.7     .0   61.6
CYCLE COUNT ( % )      0.%   38.%   54.%   64.%   85.%   85.%    0.%   64.%
```

Figure 6-1. PINSET Report Presented by PASSER II

Phase interval timings are listed by:

1. Seconds;
2. Percent of cycle; and
3. Cumulative percent of cycle.

These phase intervals prove useful in developing phase (or cam) interval charts for traffic control purposes. Each phase interval consists of the allocated green, yellow, and red clearance times for the compatible NEMA movement, expressed both in seconds and percent or fraction of the cycle length. The cycle count percentages are the cumulative percentages of each phase interval from the beginning of the cycle.

6.3 Implementing Pretimed Settings

When implementing timing plans for pretimed control, both the splits and pin settings can be determined from the PASSER II-90 output. The splits, also referred to as green time, for concurrent and individual phases appear in the output table titled *GREEN TIME.* The pin settings can be read directly from the PASSER output under the title *PIN.SET* in either seconds or percent of the cycle. The *PIN.SET* equals zero for the first phase; for every subsequent phase, the *PIN.SET* equals *PIN.SET* of the previous phase plus the total time for the previous phase. The term PIN in the output gives the green time (splits) for each concurrent phase.

The phase times in the PASSER II-90 output include the yellow and the red clearance for each phase. If protected left turns with overlap are used, PASSER II-90 automatically selects the heavier left-turn movement as being the first left turn to be serviced. In a dual-ring traffic actuated controller, the heavier left-turn movement will always be serviced in the overlap phase. In a pretimed controller, however, one of the two left-turn movements must be designated as the leading movement.

The SOAP-84 output requires some manipulation before entering data into the controller sheet. SOAP-84 reports the combination of movements (phases) in percent of cycle. Each movement must be converted to seconds by multiplying the percent of the cycle by the cycle length.

The green splits generated by both PASSER and SOAP do not represent absolutes for either pretimed or actuated control, and should be used with engineering judgement. Some modifications may be necessary in the field.

6.4 Implementing Actuated Settings

When implementing timing plans for actuated control, the input data - minimum green times, yellow and red clearances, Max I, Max II must be determined to implement the desired cycle length, phase times, pedestrian requirements, and change intervals. A benefit of TxDOT's practice of only purchasing solid state NEMA full actuated controllers is that signal settings can be programmed into the controller or downloaded from a lap-top or a remote computer using a telephone modem.

Minimum Green. Analysts base the minimum green duration for Phases 2, 4, 6, and 8 on several factors: pedestrian walk time, driver expectancy, operational mode and 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 driver expectancy; however, for actuated phases using presence control as is done for low speed approaches and left-turn lanes, the minimum green is often set to 0 seconds so that the green is maintained only as long as the detector registers the presence of a vehicle. 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 (i.e. no stop line detection). The minimum green time should be based 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 and red clearance.

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

For Phases 2, 4, 6, and 8 (Through Phases)	P_{min} = larger of	$D/L_Q \times 3600/S + (l_1 + l_2)$ or $W + FDW$

where:

G_{min}	=	minimum green interval duration for phase, in sec;
P_{min}	=	minimum duration of phase, in sec;
Y	=	yellow interval duration of phase, in sec;
RC	=	red clearance interval duration of phase, in sec;
D	=	distance from stop-line to nearest edge of detector serving phase, in ft;
L_Q	=	space occupied by queued vehicle, in ft; (use 20 ft/veh);
S	=	saturation flow rate of critical movement, in vphgpl;
l_1	=	start-up lost time in phase, in sec; (use 2.0 sec);
l_2	=	ending lost time in phase, in sec; (use 2.0 sec);
W	=	steady WALK interval for phase, in sec; and
FDW	=	flashing DON'T WALK for phase, in sec (see Table 6-1).

Detector Location. The approach speed to signalized intersections should establish the detector configurations and location. In left-turn lanes and low speed approaches with speeds less than 30 miles per hour, a long presence detector is very efficient. For approaches with higher speeds, it is desirable to install multiple point detectors operating in the pulse mode. Table 6-2 illustrates the stopping distances for speeds up to 55 miles per hour, for application in the proper placement of loops on approaches.

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

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

W = curb-to-curb width of street being crossed, ft;
f = fraction of time that pedestrian calls occur. Calculated as: $f = 1 - e^{-P * C/3600}$;
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

Table 6-2. Stopping Distances for Various Approach Speeds

Speeds (mph)	Speeds (fps)	Stopping Distances (feet) (1 sec reaction time)
20	29	54
25	36	75
30	44	99
35	51	129
40	58	163
45	65	201
50	73	261
55	80	327

Low Speed Approaches. In left-turn lanes and low speed approaches, a presence detector of length equal to the required stopping distance for the approach speed is appropriate. Such a detector will not only hold the phase till the queue clears, but will also hold the phase if another vehicle enters the detection zone. The phase will be terminated once the last vehicle leaves the detection zone. Thus, the vehicle extension (passage interval), is set to zero seconds. A vehicle entering the detection zone just after phase termination has adequate time to decelerate and stop. For an approach speed of 30 miles per hour (44 feet per second), a presence detector about 99 feet in length is desirable. In such a case, a vehicle observing a change in the signal can maintain a headway of at least 2.25 seconds (99/44) and have at least 99 feet to come to a halt, both of which are comfortable to the driver.

For an approach speed of 25 miles per hour (36 feet per second), a presence detector of about 75 feet in length is provided. A vehicle observing the signal change maintains a headway of at least 2.1 seconds (75/36) and has at least 75 feet to decelerate and come to a stop, both of which are comfortable for the driver. Consider the case of an approach speed of 20 miles per hour (29 feet per second). If a presence detector of 54 feet in length is provided, a vehicle facing a signal change has at least 54 feet to come to a halt comfortably; however, a driver will have to maintain a headway of less than 1.85 seconds (54/29), which is uncomfortable to most drivers. Even a headway of about 2.00 seconds, the lower limit of the headway maintained by most drivers, will not be adequate to extend the phase. Thus, the length of the detector will have to be increased to 65 feet in order to maintain a headway of about 2.25 seconds. Therefore, the stopping distance, as well as the headway, have to be considered when selecting the appropriate detector lengths.

Consider the case of an approach speed of 40 miles per hour (58 feet per second). When using a long presence detector, the length of the detector should be at least 163 feet (from Table 6-2). The minimum green time required to clear the queue is about 18 seconds which is very high for a minimum green and should be reduced. Thus, for approach speeds of 40 miles per hour or greater, one should use an alternative method of detection, such as multiple point detectors.

High Speed Approaches. For a vehicle travelling at a speed of 40 miles per hour, a detector will have to be placed at a distance of 163 feet (stopping distance for 40 miles per hour). Positioning the detector any nearer to the stop bar than that to reduce minimum green requirements might place the driver in the dilemma zone; i.e., a zone where a driver can neither clear the intersection safely or stop at the intersection using a comfortable deceleration. If a dilemma zone exists, it should be eliminated through the use of proper yellow timings and red clearance times.

To eliminate the need for a long minimum green, a point detector (6 feet by 6 feet) should be placed 163 feet from the stop line. A vehicle travelling at a speed of 40 miles per hour needs 2.8 seconds (163/58) to enter the intersection area. A second point detector is placed at a distance of 99 feet (stopping distance for 30 miles per hour) from the stop line.

An extension (passage interval) of one second is considered appropriate. A vehicle travelling at a speed of 30 miles per hour, 1.0 second after crossing the first detector 163 feet, is 95 feet from the intersection [163 - 6 (loop length) - 18 (vehicle length) - 44 (distance travelled in 1 second)]. A vehicle travelling at a speed of 30 miles per hour needs 2.25 seconds to enter the intersection area after crossing the detector at 99 feet.

A third detector is placed at a distance of 54 feet (stopping distance for 20 miles per hour). A vehicle travelling at a speed of 20 miles per hour, 1.0 second after crossing the second detector at 99 feet, is 46 feet from the intersection [99 - 6 (loop length) - 18 (vehicle length) - 29 (distance travelled in 1 second)]. A vehicle travelling at a speed of 20 miles per hour needs 1.86 seconds to enter the intersection area after crossing the detector at 54 feet. Thus, the vehicle enters the intersection area on yellow. The minimum green requirement, obtained by calculating the time needed to clear the vehicles stored between the detector at 54 feet and the stop bar, is an acceptable 7.5 seconds.

Positioning the detectors at such spacings, with an extension of about 1 second and a minimum green of about 8 seconds, will result in better traffic operations. The resulting minimum green falls to a tolerable level. Fewer vehicles get caught in the dilemma zone, and while stragglers are eliminated, they are provided with adequate stopping distances.

For higher speeds, as many as five detectors may be needed, with the farthest being placed at 327 feet (for a speed of 55 miles per hour). The next detector can be placed at about 245 feet, which is where a vehicle travelling at 40 miles per hour would be about one second after passing the detector at 327 feet. Thus, with an extension of one second, the vehicle could extend the green for another second. The same vehicle would be 163 feet away from the stop line one second after crossing the second detector at 245 feet. This point equals the distance at which a detector was placed for an approach speed of 40 miles per hour. Vehicles travelling at speeds above 40 miles per hour will be able to enter the intersection; however, vehicles travelling at speeds below 40 miles per hour will not be able to extend the phase. Thus, stragglers are eliminated, minimum green times are limited to 8 seconds, which is acceptable, and the dilemma zone is reduced.

Maximum Green. When determining the green splits for maximum green times, if the degree of saturation (v/c) for the critical movement is less than 0.85, use the splits calculated by PASSER II-90 or SOAP-84 as the maximum green times. If the critical movement has a degree of saturation greater than 0.85, the overflow E(x) should be estimated and used to determine the maximum green time.

$$G_{max} = G + 3600/S * E(x) = (3600 * X^2 / 2(1-X)*S)$$

where:

G_{max}	=	maximum green time, (sec);
G	=	optimized phase green time, (sec);
X	=	degree of saturation; and
S	=	saturation flow on approach, in (vphg).

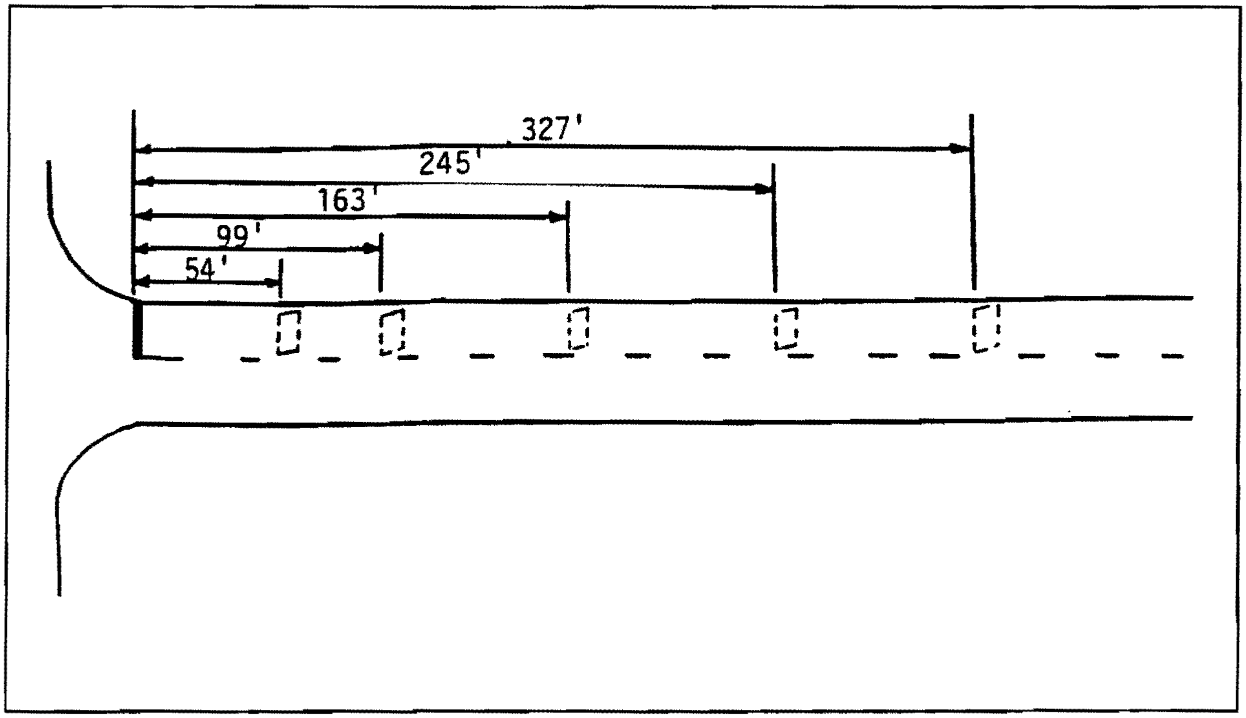


Figure 6-2. Detector Spacing for a High Speed Approach

For extremely high values of X (approaching 1.0), unrealistic maximum green times may result from this equation; therefore, for higher volume to capacity ratios, the following equation is suggested:

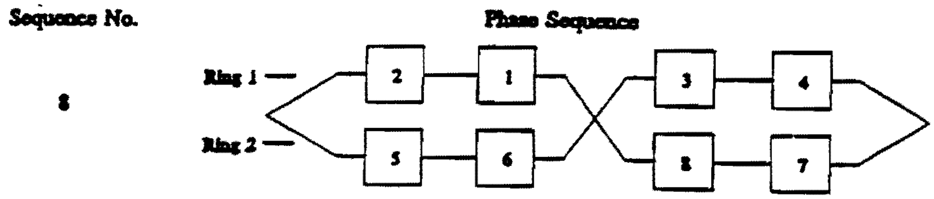
$$G_{max} = G + (3600\ X^2)/(4\ *(1-X)*S)$$

where:

where:	G_{max}	=	maximum green time, (sec);
	G	=	optimized phase green time, (sec);
	X	=	degree of saturation; and
	S	=	saturation flow on approach, in (vphg).

6.5 Multiple Timing Plans

Computer controlled controllers are capable of implementing a number of different signal timing plans. In these cases, a two-digit timing plan number is often used to identify the different cycle and split combinations. For example, a timing plan number 11 corresponds to cycle 1, split 1. In addition, a dual ring controller with a desired phase sequence of Phase 5 leading with overlap, followed by dual lefts on the cross street, is given sequence number 8.



6.6 Fine Tuning the Timing Plan

The following discussion follows suggestions and guidelines presented by Yauch and Gray (17). 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 operates 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 implementing process.

Fine Tuning In-house. Before actual field observation, one should check the data used in the analysis for errors. The simulation of existing conditions should be verified before the optimization runs are made. 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. After an optimized solution has been reached, data input and results should be thoroughly scrutinized. The transposed data from the computer output to controller settings should be reviewed for accuracy. If one takes these steps before field implementation, adjustments in the field will be minor.

The public should be notified of proposed signal changes in advance of their implementation. This notification may be accomplished by the media or appropriate signing. When actual field modifications begin, proper traffic control should be used to protect the traveling public and workers implementing the new timing plans.

Fine Tuning in the Field. Fine tuning traffic signal timing plans in the field involves the verification of plan implementation of cycle length, phase splits, and offsets. 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 first. Before actual field fine tuning takes place, the traffic should be allowed to "settle." Drivers may react hesitantly or erratically due to the change in signal timing and/or phasing. The true effect of the new control strategy on the traffic flow may not be apparent immediately due to driver behavior. Therefore, observations and measurements should not be made until drivers become familiar with the new changes.

Cycle Length and Phase Splits. After modifying the controller settings, field observations should be made to ensure that the proper settings have been implemented. One can verify cycle lengths and phase lengths in the field with a stopwatch. For most full actuated controllers, the maximum green for each phase may be observed by locking "On" VEH DET or MAX RECALL for each phase. Once the settings have been verified in the field, the functions should be locked "Off." Otherwise, the maximum times will be assigned to each phase whether or not it is needed, and most benefits from actuated control will be lost.

Implementation of new timing plans, especially those involving new phase sequences, may cause drivers to be hesitant; or, drivers may refer back to the old phase sequence out of habit. Analysts should take this into account when observing the overall effectiveness of the changes. If other problems observed in the field, such as excessive queues and poor green time allocation, are not predicted by PASSER II-90 or SOAP-84, the traffic signal analyst should check the input data and controller settings again.

Fine tuning timing plans in the field can significantly affect 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 signalized intersection. **Engineering judgment, combined with signal timing tools and public feed back, is key to developing a good retiming plan.**

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7.0 PROJECT DOCUMENTATION

After successful implementation of the new signal timing plan, documentation of the results is desirable. The following sections discuss two types of documentation. First, traffic signal analysts are interested in the benefits obtained from implementing a new timing plan because often times, traffic control improvement plans require justification to decision makers before they allocate expenditures. Verifying estimated improvements or better operations at an intersection, assist the analyst (i.e., benefits attributable) 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. The following sections discuss estimation of benefits, benefit-cost analysis, and documentation of decisions.

7.1 Estimation of Benefits

To document the benefits of a new timing plan, "before" and "after" studies are often used. Traffic signal analysts use measures of effectiveness such as delay, stops, fuel consumption, queues, and volume to capacity ratios as a base for comparison. The objectives or goals of the project should first be established before undertaking it. Some objectives may include:

1. Improved safety at the intersection;
2. Reduced system delay at the intersection;
3. Improved air quality;
4. Reduced fuel consumption; and
5. Increased intersection operational efficiency.

From some combination of these or other goals, the analysts choose measures of effectiveness for use in the before and after analyses. PASSER II-90 or SOAP-84 can be used to estimate chosen measures for both the before (existing) and after (optimized) conditions. The benefits of the new signal timing plan demonstrate the differences in the before and after conditions. Because both programs' analysis period is one hour, the benefits should be multiplied by unit costs and then converted 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., benefits should only take into account the actual number of vehicles at the intersection. 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 PASSER II-90 and SOAP-84 can be used 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 II-90 when volume to capacity ratios at the intersection are less than 0.95. The improvement due to actuated control is an approximate 15 percent reduction in individual MOEs when volume to capacity ratios are less than 0.85. The benefits are lesser as volume to capacity ratios approach capacity.

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 study conducted on retiming signals 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 note that retiming signals benefit the citizens directly by reducing fuel consumption, delay time, and the number of stops at signalized intersection.

Example Calculations. In an example problem that represents an actual signal retiming project, analysts used PASSER II-90 to evaluate existing conditions at an isolated intersection and then to produce an optimized timing plan. PASSER II-90 reports the following measurements of effectiveness: stops, delay, and fuel consumption.

To estimate the total benefits of an optimized signal system, the delay reduction (or other improvements reported by an analysis tool such as PASSER II) is multiplied by the number of hours a timing plan is in operation. If one uses three or four timing plans in a day, typically the a.m. and p.m. peak timing plans will be used for two to three hours each, and the off-peak timing plan will be used for eight to ten hours for estimating benefits; (i.e. twelve to fifteen hours of the day used for estimating benefits). 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, the improvement in measures of effectiveness, such as delay, stops, and fuel consumption, are calculated. 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 (delay, stops, or fuel consumption) by the number of hours that the timing plan operates. As discussed previously, the a.m. peak reduction may be multiplied by 2 hours, the p.m. reduction by 3 hours, the noon reduction by 2 hours, and the off-peak delay reduction by 8 hours.

3. **Compute Daily Benefits.** Next, sum the reductions (stops, delay, and fuel consumption) for each timing plan; (a.m. reduction * 2) + (p.m. reduction * 3) + (noon reduction * 2) + (off-peak reduction * 8). This sum will be 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, multiply these reductions per day by 300 days per year (not counting weekends). The yearly reductions will be 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 these reductions, select a cost from a reference such as the AASHTO Manual on User Benefit Analysis of Highway and Bus-Transit Improvements (19) per stop, per vehicle-hour of delay, and per gallon of fuel.

Example calculations showing the before and after conditions at an isolated are intersection illustrated in Table 7-1. The difference in the before (existing) conditions and the after (optimized) conditions is:

Stops	=	1873 stops/day	(5.1 percent reduction)
Delay	=	50 veh-hrs/day	(13.8 percent reduction)
Fuel	=	70 gallons/day	(9.7 percent reduction)

Typically, benefits for retiming signals range from 5 to 20 percent reductions in delay, stops, and fuel consumption, depending on the type of retiming strategy used (20). Generally, optimization of green splits or cycle length optimization produces improvements of around 5 percent, while geometric and signal hardware improvements may show as much as a 20 percent overall improvement. The improvement in MOEs also depends on the quality of the signal timing plan before it was retimed.

Table 7-1. Benefits of a Signal Retiming Project

		STOPS		TOTAL SYSTEM DELAY (veh-hrs)		FUEL (gals)	
		BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
HOURLY VALUES	AM	2910	2629	39	31	65	55
	OFF	1928	1899	13	12	34	32
	NOON	1931	1840	14	13	34	32
	PM	3828	3529	51	43	85	75
DIFFERENCES	AM		281		8		10
	OFF		29		1		2
	NOON		91		1		2
	PM		299		8		10
HRS/DAY	AM		2		2		2
	OFF		8		8		8
	NOON		2		2		2
	PM		3		3		3
DAILY TOTALS	AM		562		16		20
	OFF		232		8		16
	NOON		182		2		4
	PM		897		24		30
	TOTAL		1873		50		70
UNIT VALUES			$0.014		$10.00		$1.00
ANNUAL SAVINGS			$7,867		$150,000		$21,000
PROJECT COST :		$40,676		TOTAL SAVINGS (FIVE YEARS)		$894,333	

7.2. Benefit-Cost Analysis

When considering the question of how much to budget for signal retiming projects, one should consider total costs as well as potential benefits. 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 (approximately 10 percent) is used primarily for signal timing, this expenditure would equal $356 per signal per year, or $1067 per signal every three years. One can see that even a small reduction in stops, delay and fuel consumption would easily pay for the cost of retiming.

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 man-hours used to collect 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. Note that this cost estimate applies to retiming six intersections and includes the purchase of new hardware.

Some estimates of retiming costs given by various agencies range from $500 to $1800 per intersection (21). Some agencies estimate 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 computing benefits and costs of the signal retiming project, it is a simple matter to calculate a benefit-cost ratio for the project. Typical ranges from past projects are from $20 to $100 dollars in motorist benefits for every dollar spent in signal retiming projects. It should be noted from the previous example (Table 7-1) that motorists saved 22 dollars for every dollar spent in the signal timing project. It should be noted that the intersection received minor geometric improvements in addition to signal retiming. Thus, the benefit to cost ratio was computed for a period of five years.

After implementing, fine tuning, and documenting the new signal retiming plan, the need for future field observation does not end. Further fine tuning may be necessary as time progresses. If events cause future traffic volume shifts, the process of evaluation, optimization and implementation will need to be repeated. Careful planning of new signal design projects will ease the problems of future traffic growth. If possible, the most versatile controller equipment and signal hardware should be installed to accommodate future growth and fluctuations. As demonstrated by this example, retiming signals can be a cost-effective means for improving intersection capacity and movement.

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

Table 7-2. Cost Estimate for a Typical Retiming Project

COST ITEM	LEVEL/TYPE	TIME	COST	COMMENTS
Personal	Director	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 percent 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 II Training
Total Local Costs			$34,029.00	
Consulting	Timing Plans		$7,250.00	
	Install Controllers		$15,000.00	
	Total		$22,250	
Total Project Cost			$56,279.00	

7.3 Documentation of Decisions

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

It is recommended that documentation include when timing plans are implemented and fine tuned, including traffic control and safety procedures taken to protect the traveling public. It is further 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. Also, two copies of the signal's maintenance records should 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 files.

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