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16. Abstract

This document provides guidelines and procedures for the retiming of freeway corridors, including procedures for data collection, and the types and amounts 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, Freeway Corridors, discusses the characteristics of a freeway corridor, 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 in a freeway corridor.

Section four, Evaluation, provides information needed for the simulation of existing corridor operations, including the use of various software programs currently available. Section Five, Optimization, provides the guidelines to optimize timings in a typical freeway corridor. 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.

17. Key Words
Traffic Signal Retiming, Freeway Corridor, PASSER III, PASSER IV, TRANSYT-7F.

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IMPLEMENTATION GUIDELINES FOR

RETIMING FREEWAY CORRIDORS

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TEXAS TRANSPORTATION INSTITUTE
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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

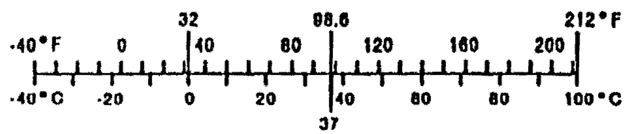
Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

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	milliliters	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, as well as 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 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 freeway corridors.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation and is NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES.

ACKNOWLEDGEMENTS

The research reported herein was performed as part of a study entitled "Implementation Guidelines for Traffic Signal Retiming Projects" by the Texas Transportation Institute and sponsored by the Texas Department of Transportation. This study was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Dr. Daniel B. Fambro, P.E. No. 47535 (Texas) of the Texas Transportation Institute served as the research supervisor, and Mr. Herman H. Haenel and Mr. Charles Riou of the Texas Department of Transportation served as technical coordinators.

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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) at freeway-frontage road interchanges, as well as the initiation of the Primary Arterial Street System (PASS) program for larger cities adds to the magnitude of the problem.

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

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

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

1.2 Objectives

This study places implementation guidelines for traffic signal retiming projects in a single set of documents. These documents include the types and amounts of data to be collected, as well as the procedures for doing so; the analytic procedures and software packages 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 include field implementation and evaluation guidelines. The specific types of retiming projects addressed 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
1165-5	Implementation Guidelines for Retiming Freeway Corridors.

This document provides individual guidelines and procedures for retiming signalized freeway corridors. This document includes the procedures for data collection, as well as the types and amounts of data to be collected; the analytical procedures and software packages available for each type of freeway corridor traffic signal retiming project.

1.3 Organization

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

1.0 Introduction
- 1.1 Background
- 1.2 Objectives
- 1.3 Organization
- 1.4 When to Retime Signals Along a Freeway Corridor

2.0 Freeway Corridors
- 2.1 Characteristics
- 2.2 Types of Phasing
- 2.3 Types of Control
- 2.4 Signal Timing Methods
- 2.5 Coordination Methods
- 2.6 Alternative Methodologies
- 2.7 Measures of Effectiveness
- 2.8 Incident Detection and Diversion

1.4 When to Retime Signals Along a Freeway Corridor

Public complaints are usually the first signs of signal operational problems. Traffic signal analysts cannot address all complaints but the complaints indicate a need for at least a field observation and/or possibly an engineering study. Some common complaints include: excessive approach delay, left-turn delay, poor progression, and excessive queues. Analysts can make field observations to determine the legitimacy of complaints. Major problems will seem obvious to the observer, such as long queues, ineffective use of green times, and excessive cycle lengths (defined here as greater than 150 seconds). In some cases, equipment such as detectors may need repair. After ruling out these problems, retiming may present a solution to improving signal system efficiency. As a rule of thumb, analysts should make field observations or studies every three to five years to determine if signal retiming is necessary.

Changes in traffic flow caused by land use and population changes, the addition or deletion of signals in the area, changes in major traffic generators, and changes in the geometrics of the roadway or intersection may also 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 documentation will help identify operational problems before they become severe.

2.0 FREEWAY CORRIDORS

2.1 Characteristics

A freeway corridor system is comprised of a freeway having adjacent frontage roads and arterial streets running in the general direction of the freeway. One can define a freeway as an expressway with full control of access (1). Freeways in arterial corridors are intended to provide high levels of safety and efficiency in the movement of high volumes of traffic at high speeds.

One can obtain mobility along a freeway corridor by maintaining high speeds on freeways and by providing progression along frontage roads and parallel arterial streets. Good progression along frontage roads and parallel arterials is essential when the need arises to divert traffic from the freeways. Some of the problems that occur along frontage roads and parallel arterial streets (as a result of poor or outdated signal timing and increasing or changing traffic demand) include:

1. Lack of progression, causing excessive stops and low average travel speed;

2. Inadequate capacity at intersections or interchanges, causing an additional amount of overflow traffic delay to occur and creating a bottleneck in the system; and

3. Overflow demand on the intersection and interchange approaches, resulting in the back-up or queuing of traffic. This stoppage of flow can block upstream intersections or can back up the traffic onto the freeway.

The following sections describe the types of phasing and controllers used at arterial street intersections and diamond interchanges; methods of timing intersections and diamond interchanges and coordinating the signals along arterial streets and frontage roads; and the measures of effectiveness used to evaluate the traffic conditions.

2.2 Types of Phasing

One can divide descriptions of the type of phasing at intersections and diamond interchanges into three parts: individual movement numbers, phasing sequence, and the type of left-turn treatment. The following paragraphs describe each of these components.

Movement Numbering. Most methods of signal timing analysis use the NEMA configuration for numbering movements. Each NEMA movement corresponds to a separate phase in an 8-phase dual-ring NEMA controller. Combinations of these movements are also considered as phases. Number the movements for intersections, start with Movement 1 (usually a left-turn movement) and move clockwise, numbering each left-turn movement 3, 5, and 7. Movement 2 (usually a through movement) always lies across from Movement 1 and is usually chosen to represent the forward direction along the arterial (i.e., the direction in which the intersections are numbered or sequenced). After selecting the direction for Movement 2, proceed clockwise to number the remaining through Movements 4, 6, and 8.

Figure 2-1 illustrates the NEMA numbering scheme for intersections. 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 are known as conflicting movements, and as a result of this conflict, they 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.

The movements at a diamond interchange are assigned numbers, as shown in Figure 2-2. Basically the arterial or cross-street movements are assigned Phases 2 and 6, the frontage roads are assigned Phases 4 and 8, and the interior left-turns are assigned Phases 1 and 5. The interior through movements are assigned Overlap A (OVLA) and Overlap B (OVLB). Overlap A is concurrent with both Phases 1 and 2, and Overlap B is concurrent with both Phases 5 and 6. The controller operates as two independent, four-phase rings and has the capability of switching between four-phase and three-phase diamond operation.

Intersection Phasing. One can classify phasing schemes by the type of left-turn treatment that exists at the intersection. Left-turn movements can be permitted only, protected only, or protected-plus-permitted (combined), as indicated in Figure 2-3 and described below.

Permitted - Left-turn vehicles must yield to opposing traffic. They may proceed on a green ball after yielding to oncoming traffic.

Protected - Left-turn vehicles proceed separately from opposing traffic. They may proceed either during a green left-turn arrow or during the time that opposing movements are stopped (i.e., split phase).

Protected/Permitted - Left-turn vehicles are protected during part of the cycle and permitted in another part of the cycle. They may proceed either during the protected left-turn phase or on a green ball after yielding to oncoming traffic.

Phase Sequence. The order in which the phases are displayed at a signalized intersection is called the phase sequence. Protected left-turns phases may be leading, lagging, lag-lead or lead-lag for the arterial and cross-streets. It is not unusual for the phasing sequence on the arterial street to differ from the phasing sequence on the cross-street. It also is not unusual for different intersections to have different arterial phasing sequences in order to improve progression along the arterial.

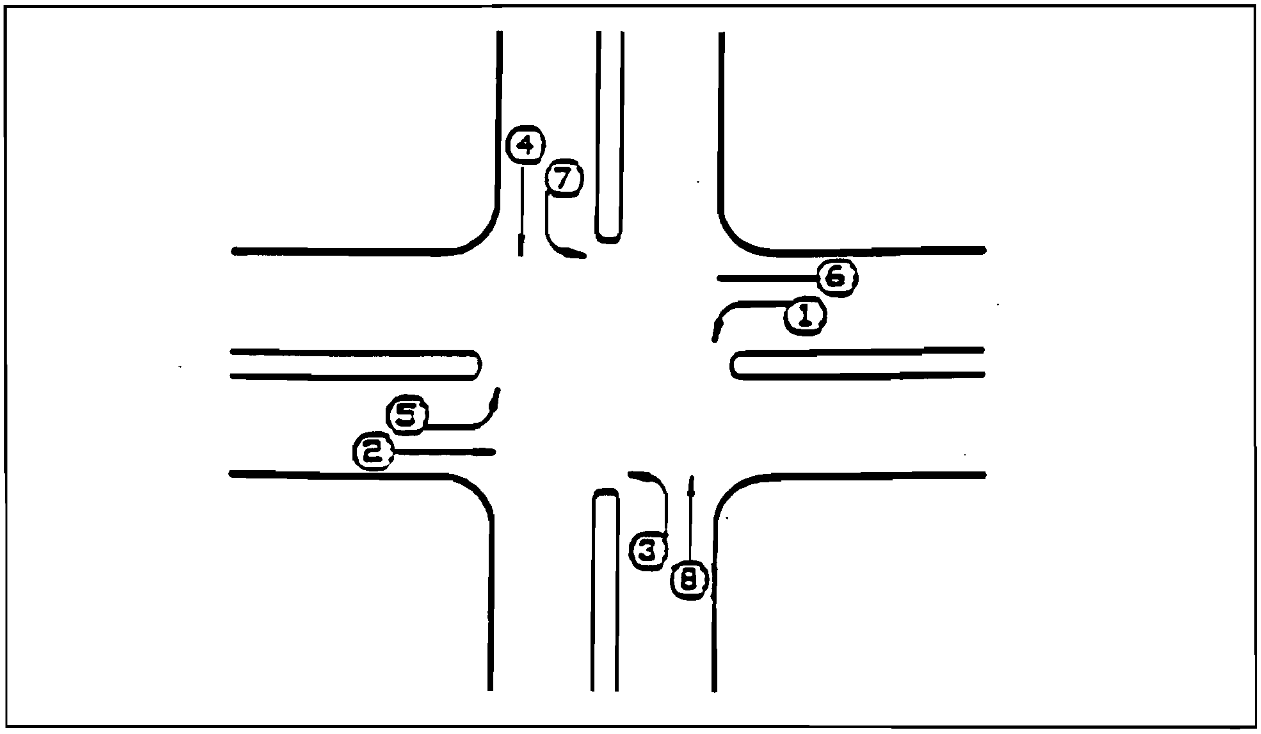


Figure 2-1. Intersection Movement Numbering Scheme for an 8-Phase NEMA Controller

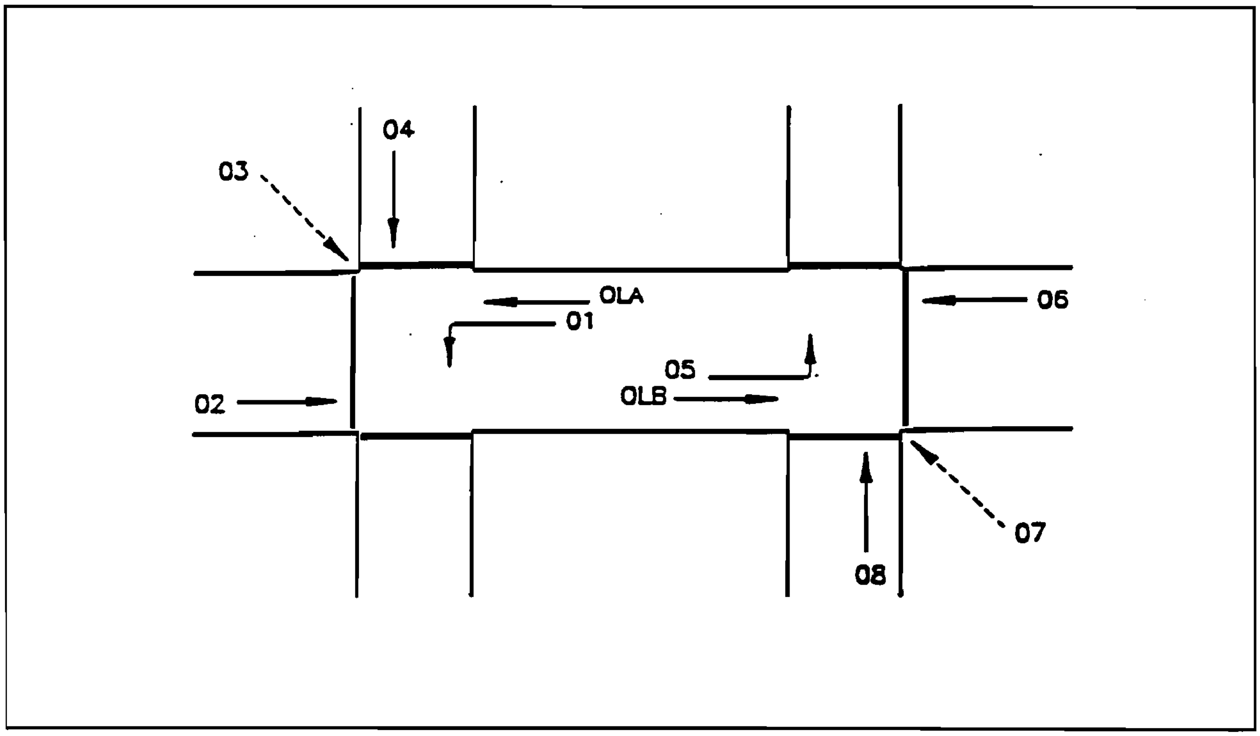


Figure 2-2. Movements at a Diamond Interchange

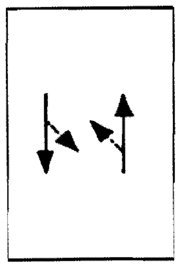
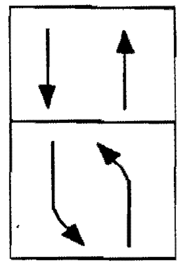
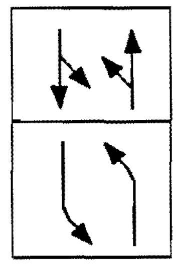
Geometry	Permitted Phasing (PM)	Protected Phasing (PR)	Combined Phasing (PP)
No Bay	1	2	3
Bay	4	5	6
Phasing			

Figure 2-3. Left-Turn Treatment Alternatives

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

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

Lag-lead/lead-lag - One protected left-turn movement and its adjacent through movement proceed before or after the opposing protected left-turn movement 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 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. One should note that the terminology for describing phase sequences may vary between signal timing software and traffic controller hardware.

Diamond Interchange Phasing. Several phasing pattern strategies exist which may be used at signalized diamond interchanges. One may classify each phase pattern by the number of basic phases and the sequence of movements at the diamond interchange. The basic phase configurations are two-phase, three-phase, and four-phase.

One differentiates two-phase, three-phase and four-phase control at a diamond interchange by the number of basic phases and the method in which one calculates green splits. For two-phase control, one treats the diamond interchange as two separate intersections, each having two basic phases. These two phases are the arterial or cross-street phase (Phase 2 or 6) and the ramp or frontage road phase (Phase 4 or 8). Protected left-turn phases for the interior movements are not provided. Although not required, the arterial phase lengths at both intersections usually equal one another, which means that the ramp phase lengths at both intersections also will be the same duration.

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

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

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

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

1. Lead-lead: left-turn vehicles from the interior lanes lead the opposing arterial phase at both intersections;

2. Lead-lag: left-turn vehicles from the interior lanes lead the opposing arterial phase at the left intersection and lag the opposing arterial phase at the right intersection;

3. Lag-lead: the mirror image of the lead-lag phasing pattern; and

4. Lag-lag: left turns from the interior lanes lag the opposing arterial phase at both intersections.

For additional information about phasing patterns for diamond interchanges, refer to Research Report 1164-3 (2).

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

2.3 Types of Control

The types of controllers at intersections along arterial streets and frontage roads affect the number of implementable timing plans, as well as the number of allowable phases and sequences. A coordinated system is typically controlled in one of two ways. First, one may operate the system by predetermined signalization schemes designed to accommodate the anticipated traffic demands on the system. This type of control is referred to as pretimed operation. Second, one may also control the system by signalization operations that maintain progression but respond to the current traffic demands in the system. This type of control is referred to as actuated or semi-actuated operation.

Pretimed Coordinated Control. Under pretimed control, right-of-way is assigned in a predetermined manner, and the cycles at intersections and the interchanges in the system are coordinated to provide for progression. The major elements of pretimed control are fixed cycle length, fixed phase sequence, fixed phase length, and fixed offsets. Depending on equipment, analysts may use several timing plans; the appropriate phase is implemented automatically at fixed times of the day, usually with one timing plan for the a.m. peak period, one timing plan for the p.m. peak period, and one timing plan for the off-peak period.

Pretimed coordination, are recommended for certain types of design and operational conditions. Design conditions that benefit from coordination are predominantly associated with intersection spacing. Generally, as the distance between signals decreases, the need for coordination increases. Signal spacing may range from 150 to 5000 feet, but spacings between 800 and 1000 feet prove ideal for coordination.

Traffic conditions that benefit from pretimed coordination include circumstances where there are a limited number of traffic patterns and there is no significant changes in these patterns. Peak periods with heavy directional traffic volumes provide additional conditions where pretimed coordinated control proves advantageous. If more than three phases are required, however, pretimed control generally is not recommended.

For pretimed operations two types of controllers exist: electromechanical (no longer being manufactured) and solid state. The electromechanical controller is comprised of one or more dials (usually no more than three) driven by a synchronous motor. Each dial corresponds to a different cycle length; for example, 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 that the mechanical parts (the dial units, camshafts, and keys) are replaced by solid state components. A microprocessor controls operations. 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, and instead only purchases fully actuated controllers and uses them as pretimed controllers when conditions warrant.

Semi-Actuated Coordinated Control. Semi-actuated coordinated control is suited for systems where traffic proves unpredictable and demand varies. Traffic actuated control attempts to adjust green times, and in some cases skips minor phases to provide additional green time where vehicular demand warrants it. Detectors placed in the approach lanes provide demand information to the controller. The basic timing parameters are yellow plus red clearance times, minimum green, green extension interval, and maximum green times 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 or, in the case of multiple detection, time to travel from the point detection to the next detector.
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.

When specifying controller settings for coordinated semi-actuated controllers, the following signal timing parameters may be encountered:

Yield Point -	The yield point is the earliest point at which the coordinated phase may end to give right of way to one or more of the conflicting phases. The yield point normally occurs at the beginning of the yellow interval (end of the green interval) of the coordinated phase.
Force-off -	The force-off point is the fixed point(s) in the background cycle length used to give right of way to one or more of the conflicting phases. One calculates force-off points by adding the splits for each phase beginning with the start of the interval to which the yield point is referenced.

Figure 2-4 illustrates the above definitions of yield and force-off points.

Two basic hardware designs for actuated controllers exist: 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. Changes in traffic conditions are accommodated by updating the software. NEMA (National Electrical Manufacturers Association) controllers meet specifications whose standards reflect 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 they have developed a set of standard specifications for their controllers.

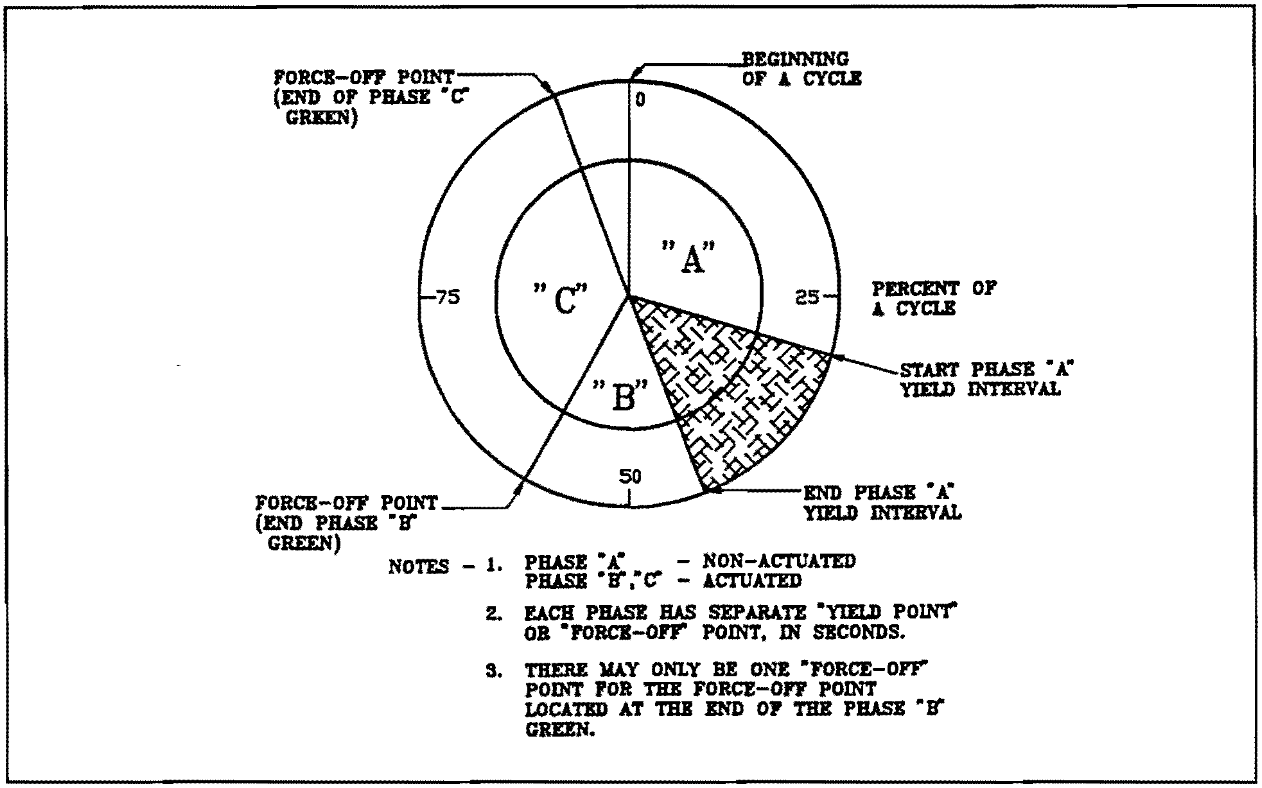


Figure 2-4. Yield Point and Force-Off Point for Semi-Actuated Controller

Actuated coordinated control of arterial street systems is predominantly at the semi-actuated operational level. The fundamental premise of semi-actuated arterial control is to provide the main-street with as much green as possible and only service the intersecting cross-streets in response to traffic demand. Actuated arterial signals operate on a common background cycle and synchronization is provided through force-off points. Coordination is provided for the arterial street phase (sync phase), which is commonly not actuated. Actuated phases consist of those serving the cross-street and the main-street left-turn movements.

For diamond interchanges, one may use two standard NEMA full-actuated units to implement three-phase or four-phase configurations. Analysts typically use actuated controllers for isolated diamond interchanges, where traffic demands and/or traffic patterns vary significantly during the day. The common traffic signal controller used by TxDOT is the Texas Diamond Controller, a special full-actuated controller developed to provide phasing that changes with changing traffic demands. One software modified eight-phase NEMA controller unit with special internal programming logic is used to provide a combination of either four-phase or three-phase operation at a diamond interchange. The change from one type of phasing to the other is made by time clock or by external traffic-responsive logic (3).

The type of control selected may affect intersection operation. At low to moderate volume to capacity ratios, actuated controllers are almost always more efficient, than pretimed controllers 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 demand, 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 optimal pretimed controller cycle lengths.

2.4 Signal Timing Methods

For a freeway corridor to operate effectively, one must optimize the signal timings at the individual intersections along the corridor. The objective of signal timing at individual intersections is to find a "best" solution characterized by a cycle length and green splits that accomplish a traffic engineering objective, such as minimizing delay, fuel consumption, or excessive queuing, or increasing the capacity of an intersection or interchange. One must then coordinate the timing for the individual intersections to achieve the traffic engineering objective for each arterial or frontage road. The best cycle length for all intersections may not be the best cycle length for each individual intersection or interchange.

Analysts must consider the traffic characteristics at different times of the day when coordinating signals. During peak hours, a directional distribution of the traffic may exist; i.e., during the morning peak, most of the traffic could move in one direction, while in the afternoon peak, most of the traffic could move in the opposite direction. In such cases, progression can be provided exclusively for the traffic moving in the peak direction. During off-peak conditions, one can provide balanced progression in both directions, or operate intersections in an isolated mode. The best solution depends on the characteristics of the intersections, intersection spacing, development along the arterial or frontage road, traffic characteristics, and the objective of the traffic signal analyst.

One signal timing method often used for retiming intersections is based on equations developed by Webster (4). Equations for estimating delay, minimum delay cycle length, and green splits are based on Webster's original work. The following sections discuss each of these equations.

Minimum Delay Cycle. Analysts often use delay as a measure of effectiveness to determine the efficiency of a signalized intersection's operation. Figure 2-5 illustrates the variation in delay with cycle length, and the equation and location for Webster's minimum and minimum delay cycle lengths. Figure 2-6 illustrates the variation in minimum delay cycle lengths at various volume levels. Note that, a minimum delay cycle exists for every volume level and if traffic volumes fluctuate during the day, an intersection may have a different minimum delay cycle and phasing sequence during different time periods. Note also that adjacent intersections with different traffic volumes may have different minimum delay cycle lengths. Webster's equation for calculating the minimum delay cycle is as follows:

$$C_o = (1.5\ L + 5) \ / \ (1 - \Sigma Y) \qquad [2\text{-}1]$$

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

The range of cycle lengths that provides acceptable average delays (near minimum delay) at a typical intersection is $0.8\ C_o < C < 1.3\ C_o$. Cycle lengths shorter than this range measure too short to handle the traffic volumes, and cycle failures (oversaturation) will occur. Cycle lengths longer than this range generally prove longer than necessary to handle the traffic volumes, and wasted time, resulting in long delays, will occur.

For diamond interchanges, one may consider each side of the interchange an intersection which will have a minimum delay cycle. The side of the interchange with the largest minimum delay cycle controls the cycle length for the entire interchange; i.e., the cycle length must measure long enough to handle traffic at the higher volume intersection. The equation for calculating the minimum delay cycle is the same as discussed earlier for intersections.

After calculating the minimum delay cycle length for all intersections or interchanges, select the largest minimum delay cycle length (i.e., the maximum of the minimums) as the cycle length for the system. Although this cycle length may measure too long for the other intersections in the system, shortening the cycle length at the critical intersection may result in oversaturation and a bottleneck on the arterial. The longest allowable cycle length should measure no more than 10 to 15 seconds longer than the shortest allowable cycle length to minimize the excess delay at non-critical intersections.

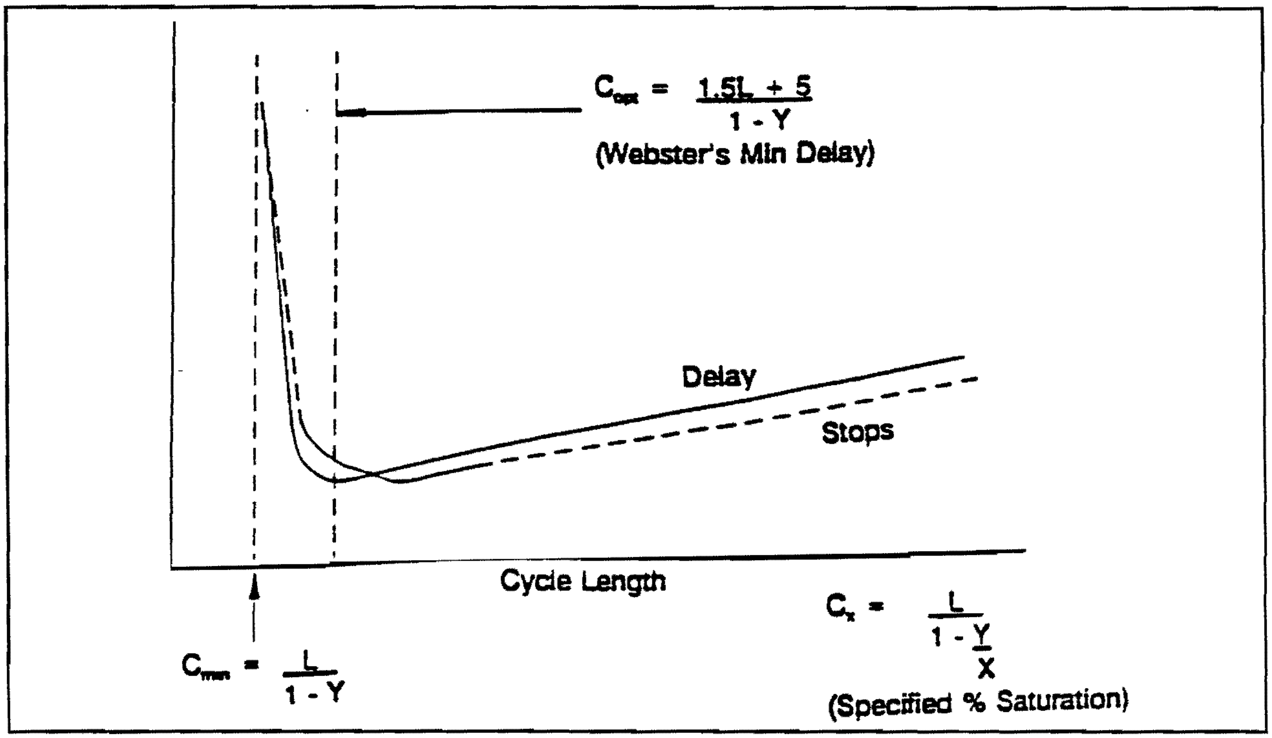


Figure 2-5. Minimum Delay Cycle Length

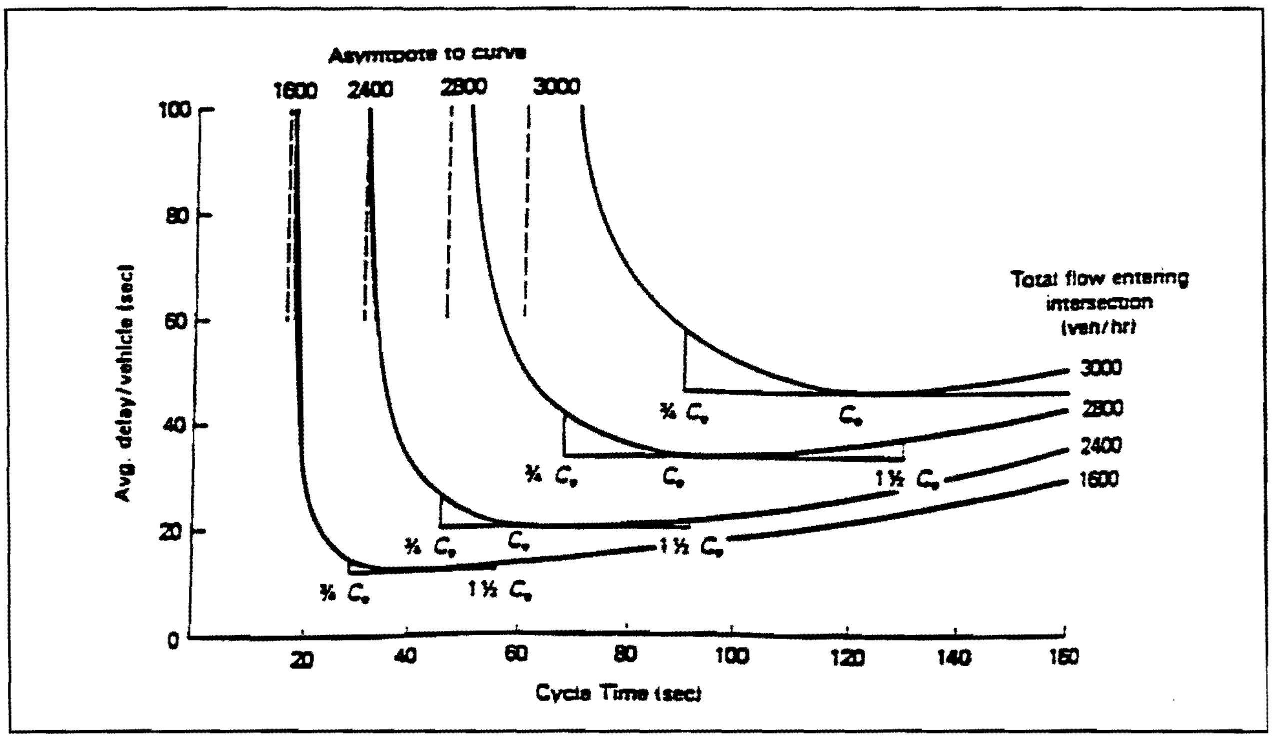


Figure 2-6. Variation in Delay with Cycle Length

Calculating Green Splits for Intersections. After determining the cycle length, green time must be allocated to each phase. In order to determine the optimum green splits at a particular intersection, the critical movements must be identified and their flow ratios calculated. The first step in calculating the green splits is the calculation of the overall volume to capacity ratio, X_I. Webster's method assumes X_I to equal the volume to capacity ratio for each critical movement (the volume to capacity ratio is the same for each movement, as well as for the entire intersection). The equation for calculating the intersection volume to capacity ratio is as follows:

$$X_I = (Y * C) / (C - L) \qquad [2\text{-}2]$$

where: X_I = intersection volume to capacity ratio;
Y = sum of the critical flow ratios;
C = cycle length, in seconds; and
L = total lost time at the intersection in 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:

$$g_i = (y_i * C) / X_I \qquad [2\text{-}3]$$

where: g_i = phase time G + Y + RC - lost time in seconds;
y_i = flow ratio for movement i;
C = cycle length, in seconds; and
X_i = intersection v/c ratio.

Calculating Green Splits for Diamond Interchanges. The diamond interchange operates very similarly to two closely spaced signalized intersections. The distance between the two frontage roads determines the efficiency and phasing strategy of the signal system and the number of vehicles that can be stored before spillback or gridlock occurs. The travel time to get from one frontage road to the next is called Φ (phi) and represents the function of the distance between frontage roads. The problem with diamond interchanges, and at many other intersections, is that generally high volumes of left-turning vehicles exist which, in most cases, require a protected phase. This additional phase reduces the time available for through vehicles.

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

$$G = (y/Y) * (C - \Sigma L) + L \qquad [2\text{-}4]$$

where:

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

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

For the four external movements:

$$Y_x = [X_x (C + 2\Phi - L_x)] / C \qquad [2\text{-}5]$$

where:

Y_x	=	$\Sigma\, y_i$ for the four external movements;
X_x	=	average v/c ratio for the 4 exterior movements;
C	=	cycle length, in seconds;
L_x	=	total lost time, usually 4 seconds per phase multiplied by the number of phases, in seconds; and
Φ	=	overlap or interior travel time, in seconds.

For four-phase with dual overlaps, the following expressions hold true in terms of average delay per vehicle:

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

For the two interior movements:

$$Y_n = [X_n \, (C - 2\Phi - L_n)] \; / \; C \qquad [2\text{-}6]$$

where:

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

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

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

$$G_1 + G_5 = C - 2\Phi \qquad [2\text{-}7]$$

where:

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

Four exterior movements feed the interchange, and their relation of phase times to cycle length and Φ are as follows:

$$G_2 + G_4 + G_6 + G_8 = C + 2\Phi \qquad [2\text{-}8]$$

where:

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

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

2.5 Coordination Methods

To provide an efficient coordinated system, one must develop cycle lengths, phase sequences, and green splits for each intersection in the system, and determine offsets to progress vehicles along the corridor. Analysts achieve coordination by altering the initiation of green phases at each intersection, such that the phase changes correspond with the arrival of a platoon of vehicles. Many of the objectives of system coordination prove similar to the objectives for intersection signal timing, such as minimizing overall delay, stops, and fuel consumption. A coordinated solution also should attempt to maximize the average travel speed along frontage roads and arterials by providing progression (minimizing stops and delay) along the freeway corridor. Good coordination along the frontage roads and arterials parallel to the freeway results in the development of efficient alternate routes for freeway users in case of any major diversions off the freeway.

The analyst must determine optimal values for the following components to develop a coordinated traffic signal plan.

System cycle length - This cycle length is based on the intersection within the system that has the longest cycle length required to satisfy the demand for that intersection. For coordinated operation, the system must operate at one cycle length and usually is constrained by the individual intersection with the maximum (longest) minimum delay cycle length.

Phasing and splits - One must determine the phase sequences and lengths for each intersection and interchange in the system.

Offsets - The analyst must calculate offsets relative to a master intersection for each intersection. One can also define an offset as the difference between the green initiation times at two adjacent intersections, and express it as a positive number between zero and the cycle length.

Time-Space Diagram. A time-space diagram is simply a plot of signal indications as a function of time for two or more signals. One scales the diagram with respect to distance, in order to plot the vehicle positions as a function of time. Figure 2-7 shows a time-space diagram for four intersections (6).

The time-space diagram in Figure 2-7 illustrates a northbound vehicle travelling at a speed of 40 feet per second. Analysts use standard conventions here to illustrate the various indications of the signals in Figure 2-7. A simple line indicates a green signal, a shaded line indicates the amber (yellow) signal, and a thick solid line indicates a red indication.

Offset, defined earlier, is the green initiation time (of the phase of interest) at the downstream intersection minus the green initiation time (of the phase of interest) at the master intersection. *Ideal offset* is defined as the offset that will cause the specified objective to be best satisfied. Often, the ideal offset equals the offset, such that, as the first vehicle of a platoon arrives at the downstream signal, the downstream signal turns green. One can calculate the ideal offset by Equation 2-9:

$$o_{id} = \frac{L}{v} \qquad [2\text{-}9]$$

where:
- o_{id} = ideal offset in seconds;
- L = block length in feet; and
- v = vehicle speed in feet per second.

If vehicles had to stop, and then accelerate after some initial start-up delay, the ideal offset could be represented by the above equation plus some term representing start-up lost time at the first intersection; however, Equation 2-9 is generally used without an added term.

Bandwidth. From Figure 2-7, one may see a window or band of green available for the movement of through vehicles. The size of this window is called the bandwidth. It is a measure of how large a platoon of vehicles can pass through the arterial system without having to stop. The bandwidth is limited by the minimum green in the direction of interest. The concept of bandwidth is popular with traffic signal analysts, as it provides easy visual images of the signal system; however, a significant shortcoming of designing offset plans to maximize bandwidth is that one may overlook internal queues. When internal queues exist, bandwidth-based solutions can be misleading and erroneous.

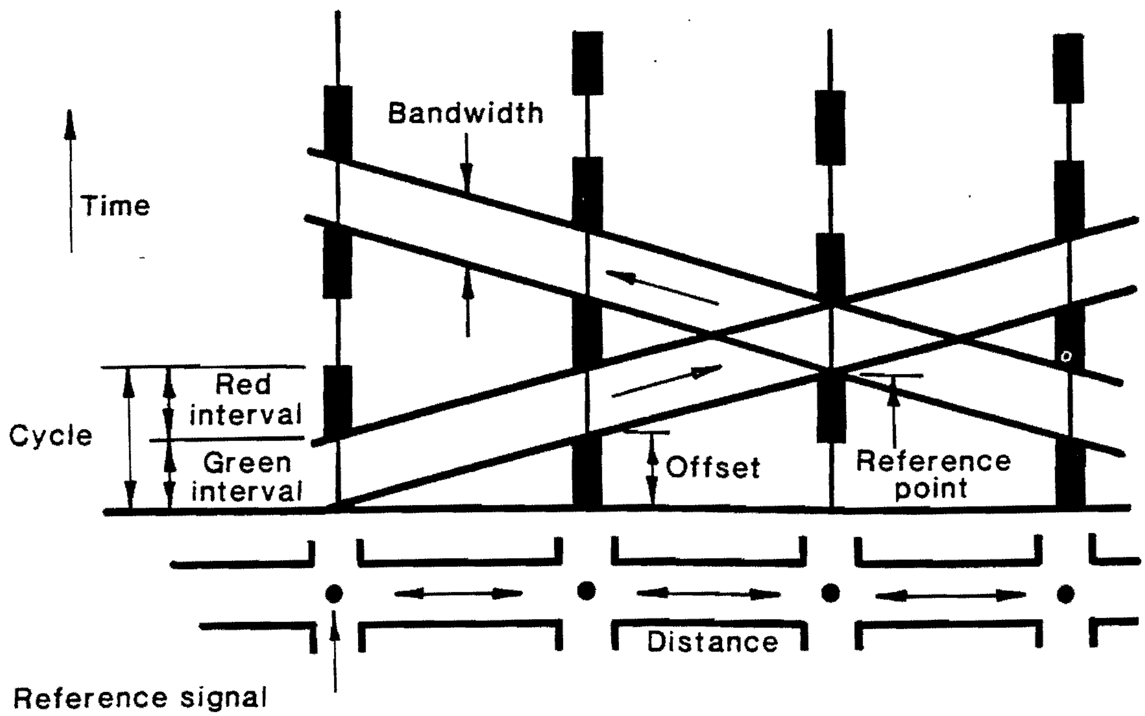


Figure 2-7. Typical Time-Space Diagram

Maximizing the bandwidth in the direction of the peak traffic movement provides good progression for the through movement. For off-peak periods, emphasis is placed on providing equal bandwidth for both directions. One can achieve this latter objective by the process of half-integer synchronization. Half-integer synchronization generally assumes that the speeds are equal in both directions and results in equal bandwidths for both directions. Analysts can maximize bandwidth by centering either the green indications or the red indications and by minimizing bandwidth reduction effects known as interferences (7).

2.6 Alternative Methodologies

The different methodologies for developing optimal arterial timing plans can be divided into one of two categories, those that minimize delay or other traffic performance measures, and those that maximize bandwidth. Each of these alternatives is discussed in the following sections.

Delay-Offset Method. Hilliar of the Transport and Road Research Laboratory in the United Kingdom developed the delay-offset method of selecting optimal offsets for pretimed signals (8). This method minimizes the overall delay but does not necessarily provide uninterrupted progression or minimize stops. The delay-offset method does not exclusively favor the arterial through movement or a maximum progression solution. The method attempts to minimize the overall delay of the system, and therefore, individual movements as well as overall system performance are of concern (9). Figure 2-8 illustrates the variation in delay with change in offset.

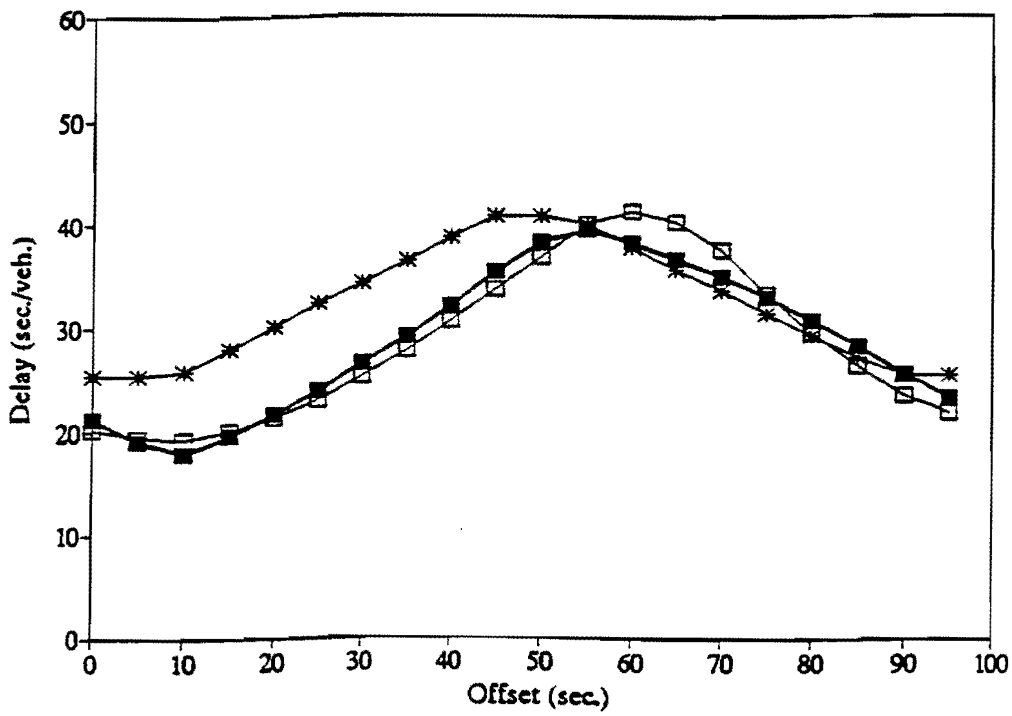


Figure 2-8. Variation in Delay with Change in Offset

Maximum Bandwidth Method. One of the most popular methods of timing arterial systems uses the philosophy of maximizing the progression bandwidth. Analysts give preference to the through movements on the arterial streets, which may cause more delay to the cross-streets. The maximum bandwidth solution attempts to develop a signal timing plan that allows a platoon of vehicles to leave an intersection and progress through the green of downstream intersections without stopping. The maximum obtainable bandwidth depends on the minimum green times for the inbound and outbound directions, and the minimum interference. The maximum bandwidth is calculated as follows:

$$B_{max} = G_{o\,min} + G_{i\,min} - I_{i\,min} \qquad [2\text{-}10]$$

where:
- B_{max} = maximum bandwidth, in seconds;
- $G_{o\,min}$ = minimum green in the outbound directions, in seconds;
- $G_{i\,min}$ = minimum green in the inbound directions, in seconds; and
- $I_{i\,min}$ = minimum inbound interference in seconds.

Efficiency and attainability provide the primary measures of effectiveness for evaluating arterial progression. Efficiency (E) describes the proportion of the cycle length used for the progression of through movements. Attainability (A) is the ratio between the sum of the bandwidths and the sum of the minimum green times in both directions; that is, the percentage of the shortest arterial through green time in each direction occupied by the band. Efficiency and attainability are directly calculated using Equation 2-11 and Equation 2-12 respectively. Several studies by various authors discuss the theory of maximum bandwidth; for further reference, see Little, et al. (10), Brooks (7), and Messer (11). Efficiency and attainability are calculated as follows:

$$E = \frac{B_a + B_b}{2C} \qquad [2\text{-}11]$$

where:
- E = progression bandwidth efficiency ratio;
- B_a = progression bandwidth in "A" direction, in seconds;
- B_b = progression bandwidth in "B" direction, in seconds; and
- C = cycle length, in seconds.

$$A = \frac{B_a + B_b}{G_{min(a)} + G_{min(b)}} \qquad [2\text{-}12]$$

where:
- A = progression bandwidth attainability ratio;
- $G_{min(a)}$ = minimum green time in "A" direction, in seconds; and
- $G_{min(b)}$ = minimum green time in "B" direction, in seconds.

2.7 Measures of Effectiveness

The measures of effectiveness (MOEs) that one uses in the evaluation of signal timing alternatives on arterial streets and frontage roads include average travel speed, critical volume to capacity ratio, total system delay, queue length, and fuel consumption. Each of these MOEs are discussed below.

Average Travel Speed. The *1985 Highway Capacity Manual* (HCM) (12) uses average travel speed as the basic measure of effectiveness for arterial streets. Such factors as the number of signalized intersections per mile and the average delay at these intersections greatly influence the average travel speed. Table 2-1 contains the arterial level of service ranges presented in terms of the average travel speed of all through vehicles on the segment. The "arterial classification" concept utilized in Table 2-1 is based upon the functional and design category of the arterial under evaluation and is described further in the 1985 HCM (12).

Table 2-1. Level of Service Ranges for Arterial Streets

ARTERIAL CLASS	I	II	III
Range of Free Flow Speeds (mph)	45 to 35	35 to 30	35 to 25
Typical Free Flow Speed (mph)	40 mph	33 mph	27 mph
LEVEL OF SERVICE	AVERAGE TRAVEL SPEED (MPH)		
A	≥35	≥30	≥25
B	≥28	≥24	≥19
C	≥22	≥18	≥13
D	≥17	≥14	≥ 9
E	≥13	≥10	≥ 7
F	<13	<10	< 7

One computes average travel speed by taking the distance between Points A and B and dividing by the travel time between the same two points. Travel time includes the average delay at each of the signalized intersections and the running time between intersections. The basic relationship between average travel speed and intersection spacing is as follows: short spacings between intersections result in running time being a lesser percentage of the travel time, and as a result, lower average speeds and levels of service. Longer spacings between intersections result in running time being a higher percentage of travel time, and as a result, higher average speeds and level of service. When computing average travel speed, however, one should realize that if **one** or more of the intersections on the arterial is oversaturated, the arterial will operate at Level of Service F even if the calculated average travel speed indicates otherwise.

Critical Volume to Capacity Ratio (v/c). A measure of effectiveness for assessing operations along a freeway corridor is the critical volume to capacity ratio for the sequence of signalized intersections along the corridor. According to the *Highway Capacity Manual* (12), the volume to capacity ratio equals the actual or projected rate of flow on an approach (or designated group of lanes during a peak 15 minute interval) divided by the capacity of the approach or designated lane group. One can use Equation 2-13 to compute the volume to capacity ratio (X_i) for a specific lane group or approach i.

$$X_i = V_i/c_i = V_i /[(g_i/C) * S_i] \qquad [2\text{-}13]$$

where: V_i = volume of approach i or lane group in vehicles per hour;
c_i = capacity of lane group or approach;
g_i = effective green time for lane group or approach i in seconds;
C = cycle length in seconds; and
S_i = saturation flow rate for lane group or approach i in vehicles per hour green.

It is important to note that capacity at intersections is defined as the maximum rate of flow (for the subject approach) which may pass through the intersection under prevailing traffic, roadway, and signalization conditions. A volume to capacity ratio of less than 1.0 indicates that the approach probably operates at an acceptable level; a volume to capacity ratio near 1.0 indicates that the approach operates near its capacity, and operational problems may occur; and a volume to capacity ratio of greater than 1.0 indicates that more demand than capacity exists; and if the capacity has not been underestimated, the approach is not operating at an acceptable level and operational problems will occur for as long as this condition persists.

The critical volume to capacity ratio for an arterial in a freeway corridor is considered to be the highest volume to capacity ratio found among the intersections along the arterial. One must realize, however, that analysis utilizing critical volume to capacity ratios can provide misleading results if the operational conditions resulting from the critical volume to capacity ratio are not taken into consideration. In other words, if one or more of the intersections operates at an unacceptable level, the arterial will also operate at an unacceptable level, even if the average speed indicates an acceptable level of service. **Thus, it is extremely important in the analysis of arterial streets to check both average travel speed and volume to capacity ratios for arterial movements at each of the intersections.**

Total System Delay. The sum of the delay for each individual movement at each of the intersections along the arterials and frontage roads is considered the total delay for the freeway corridor. Note that because total delay represents the delay for the total number of vehicles in the system, it is not comparable to another system with a different number of total vehicles. One can estimate the delay at individual intersections along an arterial and frontage roads with delay equations based on Webster's delay theory (4), or the analyst can measure delay directly in the field.

Delay is a measure of effectiveness commonly used to estimate the level of service at signalized intersections. Two types of delay can be estimated: stopped delay and total delay. Stopped delay is the amount of time a vehicle actually waits for a green indication and/or the queue of vehicles to clear. It is more easily measured than total delay. Total delay estimates the time lost to stops and speed reductions because of acceleration, deceleration, and interferences due to other vehicles. One typically determines the total system delay for an arterial using estimates for total delays for the individual intersections.

The *Highway Capacity Manual* (12) 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 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:

$$d_1 = \frac{0.38C[1 - (g/C)]^2}{[1 - (g/C)(Min(X,1.0))]} \qquad [2\text{-}14]$$

where:

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

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:

$$d_2 = 173X^2 \left[(X - 1) + \sqrt{[(X - 1)^2 + mX/c]}\ \right] \qquad [2\text{-}15]$$

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

The intersection stopped delay is as follows:

$$d = d_1 * DF + d_2 \qquad [2\text{-}16]$$

Total delay can be related to stopped delay as follows (13):

$$D = 1.3 * d \qquad [2\text{-}17]$$

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

Queue Length. Queue length provides another basic measure of performance. It is of particular importance when a limited queue storage space exists. A heavy left turn demand or a short left-turn lane can cause queues of left-turning 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 lanes and, in some cases, can block upstream intersections or can backup onto the freeway. Presence of queues at the intersection impedes the smooth flow of platooned traffic; causes more stops, delays, and driver frustration; and in turn can cause safety problems.

According to Akcelik (14), the average number of vehicles in the queue at the start of the green period can be calculated as follows:

$$N = qr + N_o \qquad [2\text{-}18]$$

where: N = average number of vehicles in queue, in vehicles;
q = arrival flow rate in vehicles per second, in vehicles per second;
r = effective red time in seconds, in seconds; and
N_o = average overflow queue in vehicles and given by:

$$N_o = \frac{QT_f}{4} (z + \sqrt{z^2 + \frac{12(x - x_o)}{QT_f}}) \qquad [2\text{-}19]$$

where: Q = capacity in vehicles per hour;
T_f = flow period, in hours;
z = $(x - 1)$;
x = degree of saturation (q/Q); and
x_o = $(0.67 + sg/600)$, where s = saturation flow, in vehicles per second, and g = effective green time for the lane group, in seconds.

The above-mentioned equation for queue length is based on the theoretical model, which assumes that vehicles join the queue when they reach the stop-line. Since vehicles actually join the queues before reaching the stop-line, the equation underestimates the maximum queue length. Maximum queue length can be calculated as follows:

$$N_m = \frac{qr}{1 - y} + N_o \qquad [2\text{-}20]$$

where: N_m = maximum length of the queue; and
q = arrival flow rate in vehicles per second;
r = effective red time in seconds; and
y = flow ratio, (volume/saturation flow rate).

Stops. The average number of stops is another basic measure of performance from which one obtains other (secondary) measures of performance (like fuel consumption). Note that every vehicle which comes to a complete stop at an intersection experiences a small delay. According to Akcelik (14), the average number of stops per vehicle is called the stop rate and can be calculated as follows:

$$h = 0.9 \left(\frac{1 - u}{1 - y} + \frac{N_o}{qC}\right) \qquad [2\text{-}21]$$

where:
- h = average number of complete stops per vehicle (stop rate);
- u = green time ratio (g/c);
- y = flow ratio (q/s);
- q = flow in vehicles per second;
- C = cycle length in seconds; and
- N_o = average overflow queue in vehicles.

One obtains the number of stops per movement by multiplying the stop rate (h) by the demand volume in vehicles per hour.

Fuel Consumption. Faced with fuel shortage, increased fuel prices, and increased environmental awareness traffic signal analysts have become more interested in fuel consumption estimates. One divides fuel consumption in an arterial system into three components: fuel consumed travelling from Point A to Point B, fuel consumed while stopped at an intersection, and fuel consumed while decelerating to a stop and accelerating back to a desired speed. The following model is used to calculate the fuel consumption in both PASSER II-90 and TRANSYT-7F.

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

where:
- F = estimated total system fuel consumption, in gallons per hour;
- TT = total travel, in vehicle miles per hour;
- D = total delay, in vehicle hour per hour;
- S = total stops, in stops per hour;
- V = cruise speed, in miles per hour; and
- A_{ij} = regression model beta coefficients, and is given by:

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

2.8 Incident Detection and Diversion

In a freeway corridor, freeways serve as the major carriers of traffic. Urban freeways are characterized by 6 to 8 lanes, high access control, and high speeds. Often freeways serve as the major traffic carriers, although alternate routes are available. Freeways are subject to regular peak-period congestion, as well as congestion experienced due to the occurrence of an accident or other lane-blocking incidents. These accidents reduce the freeway capacity.

The effects of lane blocking prove significant. For a six lane freeway, while 50 percent of the capacity is lost when one lane is blocked, 79 percent of the capacity is lost when two lanes are blocked (15). Similarly, the time at which an incident occurs is also significant. More delay is experienced if an incident occurs before the peak hour than after the peak hour. Incidents during peak hours cause the capacity of the freeway to fall below the demand volumes. Another factor affecting the amount of congestion is the duration of the incident. The longer the duration, more severe the congestion experienced. Under such conditions, the extra capacity available along frontage roads and arterials in the freeway corridor can be utilized.

Delay on the freeway due to congestion can be reduced, and extra capacity along the arterials in the corridor can be utilized only after advising motorists early about the congestion ahead and diverting them to the available alternate routes. One can detect incidents through electronic surveillance, closed-circuit television, aerial surveillance, emergency call boxes and emergency telephones. Each incident detection method has advantages and disadvantages. One can inform motorists about congestion through changeable message signs and radio announcements.

One analyzes the arterials parallel to the freeway for normal traffic conditions. This analysis will identify any bottlenecks along the arterials. The analyst can improve the timings along the arterials to provide progression for the major through movements. Some transportation system management measures, like adding a left-turn lane or right-turn lane, changeable lane assignment signs, or advance warning signs can be implemented to alleviate the identified bottlenecks. One can develop a number of timing plans to be used at different times of the day for the freeway corridor network. Analysts can develop separate timing plans for the peak periods and the off-peak periods. Plans for the peak periods can emphasize providing progression along arterials parallel to the freeway corridor. Such progression improves mobility along the corridor and encourages users to divert to the alternate routes and make efficient use of available capacity. For off-peak periods, however, the timing plan should emphasize providing progression along all the arterials in the network to improve overall traffic conditions in the freeway corridor network.

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3.0 DATA REQUIREMENTS

A major component of good signal timing plans is adequate data for analysis and optimization purposes. Poor or incomplete data results in less than desirable timing plans. Analysts may categorize the type of data required for freeway corridor systems into two groups: intersection data and freeway system data. Intersection data involves data collection at each individual intersection within the system while freeway system data corresponds to the overall characteristics of the freeway corridor. Knowing what data is needed before going to the field will save both time and extra trips to the project site for the traffic signal analyst.

The first question to be asked is how many timing plans will be needed? The number of timing plans necessary depends on the fluctuation of traffic demand throughout the day and the type of control equipment available. Analysts should collect data during the periods of interest. For example, one-way progression may be desirable in the inbound direction for the a.m. peak and in the outbound direction for the p.m. peak. Two-way progression may be desirable for the off-peak or a.m. and p.m. peaks as well. Data for developing the a.m. peak timing plan should be collected during the a.m. peak period and data for developing an off-peak timing plan should be collected during the off-peak time period, etc.

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

There are four types of data to be collected:

1. Traffic Data;
2. Signal Data;
3. Geometric Data; and
4. Travel Time Data.

The type of data needed for analyzing a freeway corridor varies somewhat for each corridor. A worksheet for recording individual intersection data such as the one specified in the *1985 Highway Capacity Manual* (12) and illustrated in Figure 3-1, proves helpful as a starting point.

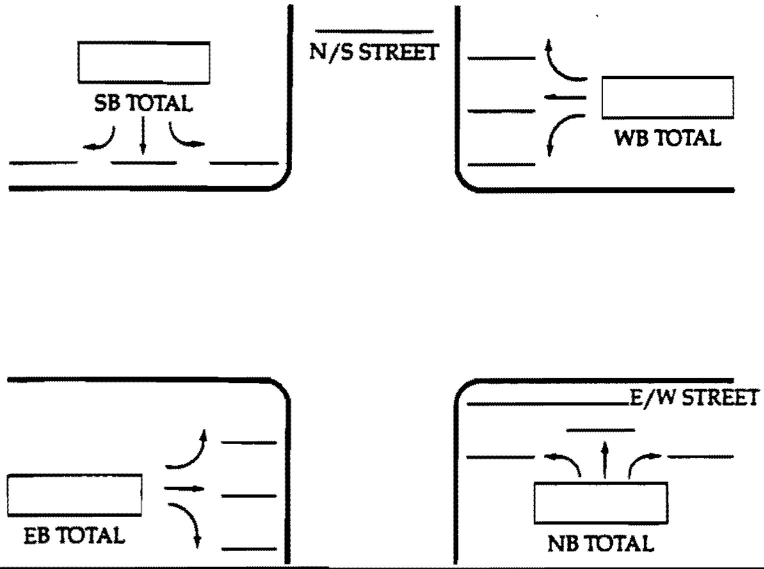
INPUT WORKSHEET

Intersection: ______________________ Date: __________

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

Project No.: ______________________ City/State: ______________________

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		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

3.1 Traffic Data

Traffic data identifies both the demand and capacity of intersections, as well as the factors affecting the smooth flow of traffic along the freeway. The quantification of demand requires observation of the 24-hours traffic volumes at the intersections, the traffic volumes during the peak period, and the traffic volumes for specific turning movements during the peak period. The analysts should also record mid-block traffic generators and their influence on freeway traffic. The capacity of an intersection is calculated based on the saturation flow rate and available green time for each movement. The number of heavy vehicles using the intersection, as well as bus stops and parking near the intersection, and the number of pedestrians that cross the intersection affect the saturation flow rate. The following text elaborates on the collection of traffic data.

Traffic Volumes. One should obtain traffic volumes for each intersection in the freeway system for the a.m., p.m., and off-peak time periods. The first step in this process, analyst should make a 24-hour count at each intersection 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 at the intersection.

Turning Movements. After determining the peak period, make the necessary manual counts to record the volumes for individual movements or lane groups during the peak period or period of interest. Twelve possible movements should be counted at each signalized intersection, as shown in Figure 3-3. Generally, one should make turning movement counts in 15-minute intervals during the two hour a.m. or p.m. peak periods, and for one hour during the off-peak period.

For diamond interchanges, there are 18 movements to be counted, as shown in Figure 3-4. Note that these movements are defined by both their origin and destination. For example, straight through movements on the exterior approaches to the interchange are subdivided into those that travel straight through both intersections and those that turn left at the downstream intersection.

Figure 3-5 shows a turning movement data sheet for a diamond interchange, the various movements being illustrated with time divided into 15 minute intervals. When counting turning volumes, consider the interior movements separately from the other turning movements, even though they were previously counted on the frontage road or cross-street. As mentioned previously, one should make these counts in 15 minute intervals within the peak period.

TRAFFIC COUNT SHEET

LOCATION: GREGORY EAST of ALPINE DATE: OCTOBER 4-5, 1984

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

TIME	COUNT	HOUR COUNT	TIME	COUNT	HOUR COUNT	TIME	COUNT	HOUR COUNT
24:00	17	54	8:00	93	275	16:00	69	292
:15	18		:15	52		:15	71	
:30	11		:30	60		:30	73	
:45	8		:45	70		:45	79	
1:00	9	21	9:00	49	220	17:00	106	335
:15	7		:15	58		:15	76	
:30	3		:30	61		:30	80	
:45	2		:45	52		:45	73	
2:00	1	9	10:00	62	254	18:00	83	272
:15	6		:15	63		:15	58	
:30	2		:30	74		:30	68	
:45	0		:45	55		:45	63	
3:00	2	9	11:00	74	253	19:00	59	202
:15	0		:15	62		:15	53	
:30	4		:30	46		:30	50	
:45	3		:45	71		:45	40	
4:00	2	27	12:00	80	285	20:00	46	142
:15	8		:15	55		:15	38	
:30	4		:30	68		:30	23	
:45	13		:45	82		:45	35	
5:00	6	47	13:00	61	262	21:00	27	117
:15	15		:15	71		:15	31	
:30	9		:30	71		:30	36	
:45	17		:45	59		:45	23	
6:00	16	158	14:00	63	273	22:00	31	92
:15	23		:15	74		:15	15	
:30	49		:30	68		:30	30	
:45	70		:45	68		:45	16	
7:00	53	352	15:00	56	260	23:00	18	57
:15	81		:15	66		:15	10	
:30	116		:30	76		:30	18	
:45	102		:45	62		:45	11	

A.M. PEAK: (07:15 -08:15)

81	392
116	
102	
93	

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

79	341
106	
76	
80	

24-HOUR TOTAL: (11:00 THUR-11:00 FRI)

4268

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

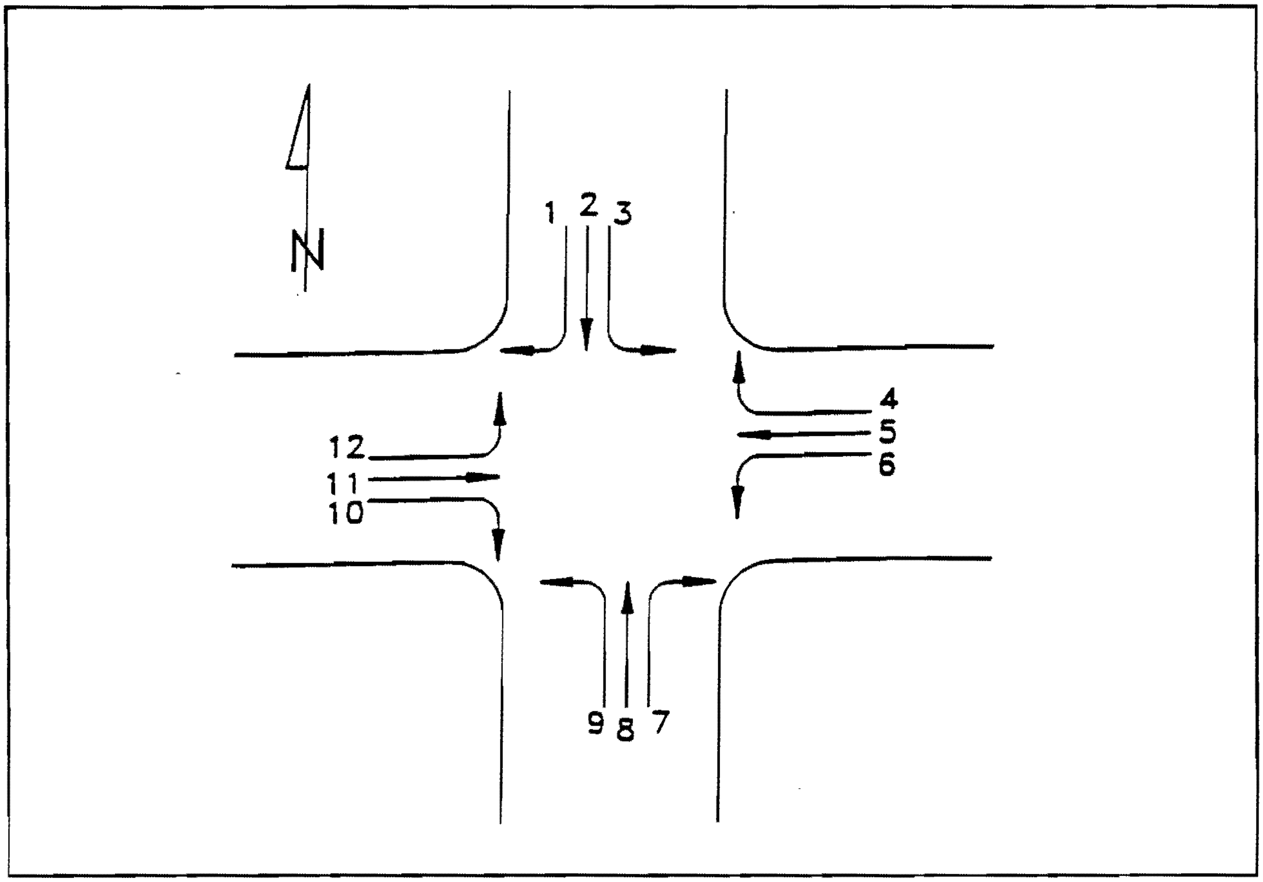


Figure 3-3. Turning Movements to be Counted at a Typical Intersection

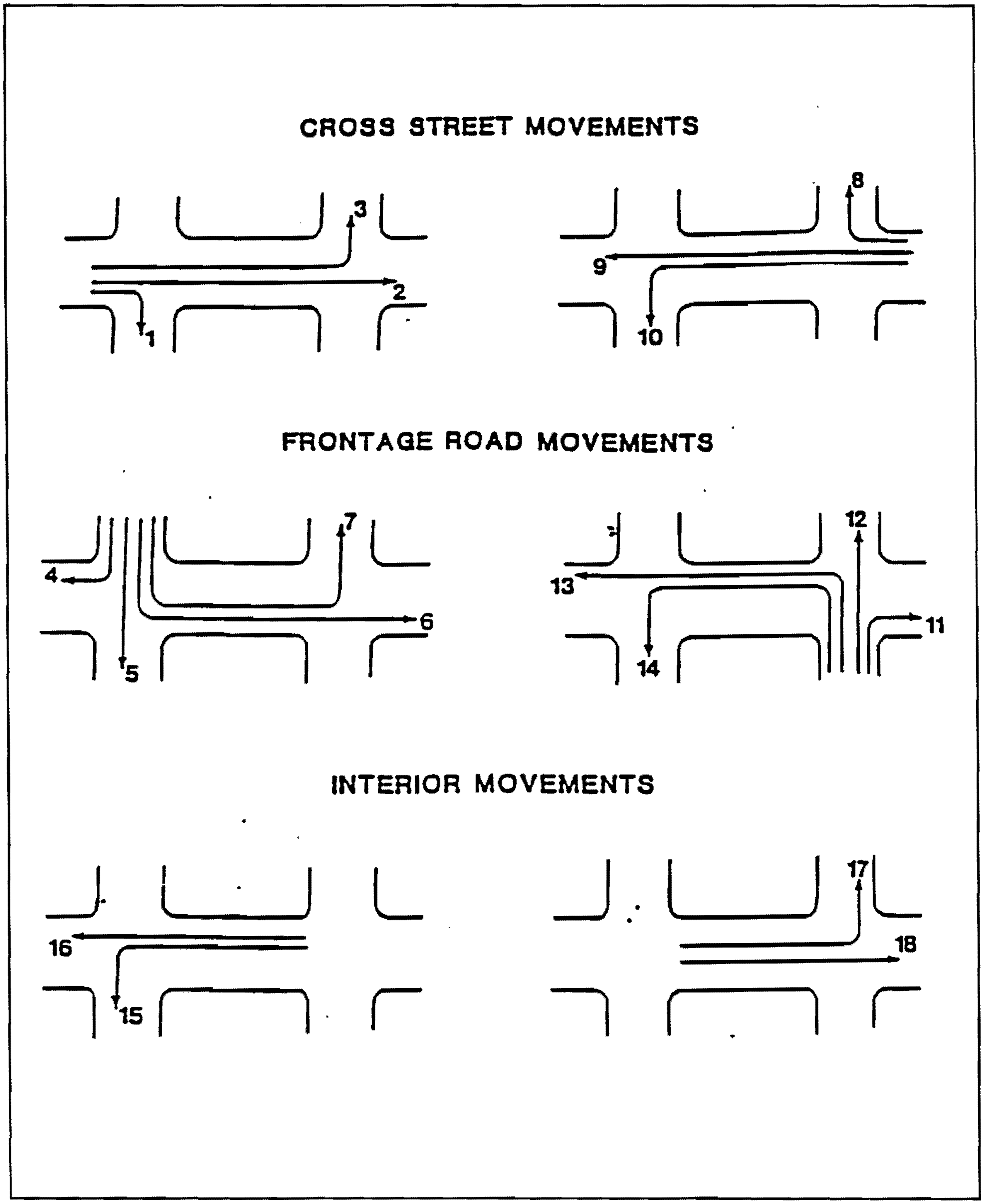


Figure 3-4. Turning Movements to be Counted at a Diamond Interchange

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

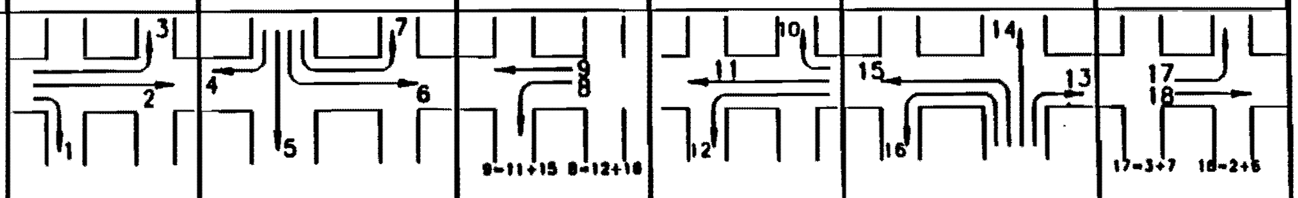


Figure 3-5. Turning Movements Data Sheet for a Diamond Interchange

Add the highest four consecutive 15-minute volumes together to determine the highest peak or off-peak hour flow rate. It may be helpful to record intersection data on a worksheet, such as the one in the *Manual of Traffic Engineering Studies* (16) shown in Figure 3-6. A sketch illustrating the freeway corridor system, as well as the orientation of intersections along the arterials and frontage roads and their geometric basic features, also will prove helpful for recording system data (see Figure 3-7).

During congested periods, it is important that the volume counted equal 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 be those vehicles that arrive at the back of the queue rather than those that depart during a green signal. One should note however, that this procedure is for counting only and is 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, this report suggests that the analysts record the number of vehicles making right turns on the red interval. This number will be subtracted from the total right-turn volume when modeling (analyzing) the existing conditions at any intersection along the arterial.

Mid-block Volumes. Heavy traffic generators may exist between intersections along the arterial street. These generators may include major shopping centers, large parking lots, etc. One should use engineering judgment to determine the significance of the volumes generated. Most sources of large traffic generation have signalized access points; however, this generalization may not be so in all cases. Significant mid-block traffic affects the quality of progression on the arterial streets and frontage roads.

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

$$PHF = \frac{V}{(V_{15} * 4)} \quad [3\text{-}1]$$

where: PHF = peak hour factor;
V = highest hourly volume, in vehicles per hour; and
V_{15} = highest 15 minute count within that hour, in vehicles per 15 minutes.

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 3-6. Worksheet to Record Intersection Data

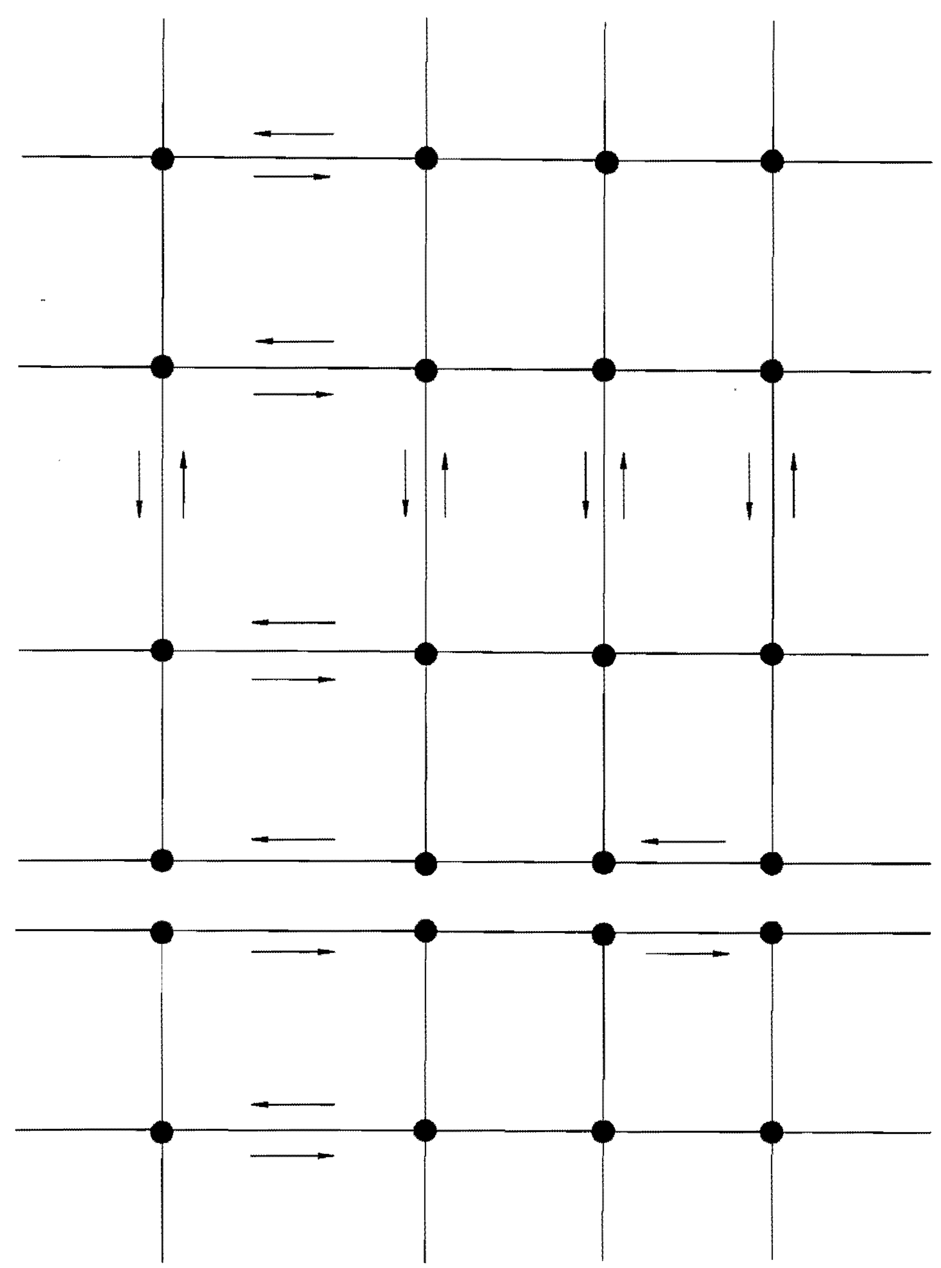


Figure 3-7. Typical Freeway Corridor

Demand volumes are generally stated in terms of vehicles per hour for a peak hour. For signal timing analysis, one normally adjusts peak hour volumes to flow rates in vehicles per hour for a 15-minute period. For example, analysts conducted a 24-hour count and determined the a.m. peak hour to be between 7:00 a.m. and 8:00 a.m. The analysts also determined the hourly volume to equal 900 vehicles per hour and the peak 15-minute flow rate to equal 300 vehicles in 15 minutes or 1200 vehicles per hour.

$$PHF = \frac{900}{(300 * 4)} = 0.75 \qquad [3\text{-}2]$$

Thus, the peak hour factor for the a.m. peak equals 0.75 in this example. For timing purposes, one can calculate the peak hour flow rate 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 flow rate equals 1200 vehicles per hour. One should note that if the peak 15-minute flow rate was multiplied by four to arrive at a peak hour flow rate, the correct peak hour factor is 1.0; i.e., the adjustment for peak flows within the hour have already been accounted for.

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

Percent heavy vehicles - The number of heavy vehicles operating within an intersection should be counted. A heavy vehicle is characterized as having 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; i.e., for example heavy vehicles accelerate from a stop at a slower rate.

Parking - Parking in the vicinity of an 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 an 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 schedules are posted at the bus stop, and further information on bus frequency may be obtained from the bus company.

Pedestrians - The number and type of pedestrians crossing each intersection should be noted. Elderly pedestrians and children require more time to cross the street. Analysts 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-turn movements may require reduction. Pedestrian volumes can be recorded 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 to base the data on field observations.

Because saturation flow rate is a critical factor one should measure it in the field if possible. Figure 3-8 shows a worksheet for determining the saturation flow rate for an approach using direct observation. If field measurements are not available, one can estimate the saturation flow rate using the procedure outlined in the *1985 Highway Capacity Manual* (12).

The analyst should note 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 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 needed will be allocated to that movement. Neither condition is efficient or desirable.

FIELD SHEET – SATURATION FLOW STUDY

Location:____

Date:____/____/____ Time:______________ City:______________

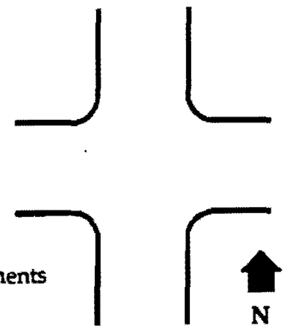
______Bound Traffic; Approaching From the________________________

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 3-8. Worksheet for Calculating Saturation Flow Rates

3.2 Signal Data

One may take most information classified as signal data (cycle lengths; green, yellow, and red intervals; phasing; and offsets) directly from the controller for pretimed control plans. For actuated control plans, the fixed yellow and red intervals remain constant, and yield, hold and force-off settings may be obtained from the actuated controller. Average green intervals 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 of each intersection remains constant. Actuated control systems also must operate at the same cycle or some multiple of the system cycle length, normally called the background cycle length. One may obtain the system cycle length from existing timing plans, the controllers, or by field measurement with a stopwatch.

Green Splits. The analyst should record green splits for each intersection. The green split, i.e., green, yellow, and red clearance for each phase, will remain constant for pretimed control and can be obtained from controller settings, existing timing plans or by field measurement. For actuated control, the green interval will be a variable length interval and one should be determined by recording 10 to 30 measurements of the green interval in the field and calculating average green interval length. The yellow and red intervals remain constant for both pretimed and actuated control. One may obtain this information from timing plans or/by field measurement with a stopwatch.

Phasing. The analyst 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 order that the left-turn phases occur describe the phasing sequence. Types of phasing include permitted, protected, and protected/permitted.

Permitted - Left-turn movements must yield to opposing traffic. They may proceed during green ball indication after yielding to opposing traffic.

Protected - Left-turn movements proceed separately from opposing traffic. They are protected with a green arrow or a split phase. Left-turning vehicles are not allowed to proceed otherwise.

Protected/Permitted - Left-turn movements are protected during part of the cycle and permitted in another part of the cycle. They may proceed during the protected left-turn phase or on 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 Left - Both left-turns proceed before the through movements.

Lagging Left - 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.

Offsets. In pretimed control, a parameter called the offset, references signals to a master intersection. Offsets usually reference to the start or end of main-street green; however, they may be referenced to the start or end of the phase for the coordinated movement. Existing offsets may be obtained from timing plans or master controller settings. Actuated controllers use force-off, yield, and hold commands to provide progression. One may also obtain these settings from controller settings.

Type of Controller. Analysts may control freeway corridor systems by pretimed or actuated controllers. Section 2.3 previously discussed the characteristics and capabilities of these controllers, but, in general, the following attributes should be noted for each type of controller.

Pretimed - Is the controller electromechanical or digital? How many dials does it have?

Actuated - Is the controller a single or dual-ring? How many timing plans, cycle lengths, and split patterns will it accommodate?

Master Control Unit. A master controller coordinates the individual intersections in the freeway corridor system. Local controllers may be interconnected with cable or a time-base coordinator may provide coordination. Some local coordinators have the time clocks or master systems built into the cabinet, or the master controller may be a separate controller to which all other intersections are referenced. One should record the existing arrangement and equipment.

3.3 Geometric Data

The analyst may determine geometric data from site plans or through field inspections. Geometric conditions can affect traffic operational data, such as the saturation flow rate. Additionally, the geometric conditions that exist at the intersections along the arterial dictate the signal data, particularly with regard to phasing and left-turn treatments.

Number of Lanes. One should record the number of lanes per approach for each intersection. Note that the counting of the number of lanes at the stop bar, not upstream or downstream of the intersection. The type of movements allowed from each lane also should be recorded, including exclusive turning lanes and shared lanes. The analyst should also note the following information 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, whether storage is adequate for the expected queue.
Through lanes -	the number of lanes, whether the through lanes accommodate left or right turns.
Right-turn lanes -	the number of lanes, whether right turns have an exclusive lane.

For diamond interchanges, the number of lanes on the ramp or frontage road, arterial or cross-street, and the interior of the interchange should be recorded. Some diamond interchanges have U-turn lanes, and the analyst should note them if they exist.

Lane Widths. The analyst should measure the lane widths for each lane on the approach or obtain them from existing plan sheets. For diamond interchanges, the lane width for each lane of the cross-street, frontage road, and the interior portion of the interchange is required. Lane width affects the saturation flow rate. Lanes less than 12 feet in width reduce the saturation flow rate and thus, the available capacity for that movement.

Percent Grade. The analyst should record the percent grade at each approach. This information should be obtainable from existing plan sheets or be measurable in the field. Percent grade will also affect the saturation flow and can cause lost time due to longer start-up times on an uphill grade.

Location. The location of the intersections with respect to the surrounding area should be noted. Is the intersection in a central business district (CBD)? An example of a CBD would be a downtown area where arterial streets cross each other to form a grid system or network. Each arterial is of equal importance with regards to progression. Businesses and shops are prevalent, and heavy pedestrian traffic, parking maneuvers, and turning movements occur.

Intersection Spacing. One needs to know the distance from intersection to intersection in the freeway corridor system. The distance is usually measured from stop-line to stop-line from field measurements or plan sheets. Distances between intersections are usually the same in both directions; however, for skewed or offset intersections, distances between intersections may differ in different directions.

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

3.4 Travel Time Data

Analysts can use the test car method to determine the average speed for the arterial streets and frontage roads in a freeway corridor system. The steps for conducting travel time studies include making an inventory of the arterial geometrics and frontage roads, identifying the segments, determining segment lengths, identifying the access control in each segment, determining the existing signal timing, and identifying the peak 15 minute flow periods and an off-peak period. The appropriate free flow speed along the arterials and frontage roads is determined. The analyst determines the free flow speed by making runs during low volume periods and noting the speed at mid-block locations in a test car equipped with a calibrated speedometer. These readings can be supplemented with spot speed studies at mid-block locations. From the information obtained, one can determine the arterial classification.

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

The next step is to conduct travel-time runs. The analyst should drive the test car as if it were a through vehicle. Note the location of all desired check points and their distances from the starting point. Start the stop watch or a timer at the first intersection, either at the instant the test vehicle comes to a stop, or at the midpoint of the intersection if the vehicle does not stop at the intersection. Note the cumulative travel time, stopped delay time, and the reason for the delay at each signalized intersections and other check points, such as stop signs. The information obtained should be filled in a tabular form, as indicated in Figure 3-9.

Make six to twelve travel time runs for each study period. These runs should start at different times with respect to the cycle to avoid being a "first in the platoon vehicle" for all the trips. Also note and compare the mid-block speedometer readings with the free flow speeds measured earlier. Finally, calculate the average travel time and travel speed for each segment along the arterial, and the average travel time and travel speed for the entire arterial.

TRAVEL TIME FIELD WORKSHEET

Arterial____________________ Date__________

Driver__________ Recorder__________ Direction__________

		Run No. __ Time		Run No. __ Time		Run No. __ Time	
LOCATION	DIST (mi)	STOP TIME (sec)	CUM TT (sec)	STOP TIME (sec)	CUM TT (sec)	STOP TIME (sec)	CUM TT (sec)

S—Signal (lower box)
LT—Left Turn (upper box)
P—Pedestrian (upper box)
PK—Parking (upper box)
4W—4-Way Stop (upper box)

Figure 3-9. Travel Time Field Worksheet

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4.0 EVALUATION

After collecting the data the next step toward retiming a freeway corridor is to evaluate the existing conditions. Evaluating an existing traffic control strategy requires field observation as well as analysis of existing conditions with computerized traffic models. A recommended methodology for assessing the operational efficiency of a traffic signal control strategy for traffic signals in a freeway corridor is summarized below.

Field Evaluation

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

2. At critical intersections, check that the green intervals are short enough that no long periods of time exist when no vehicles move through the intersection. Longer than necessary green intervals result in wasted time, unnecessary delay, and longer queue lengths for the other movements; however, note that longer than necessary green intervals will occur at all non-critical intersections because of the need for a common cycle length for coordination.

3. Check that left-turn queues do not exceed the left-turn storage. If so, left-turning vehicles may block through lanes and reduce their 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.

4. Check the queue lengths in the inner lanes on the frontage roads to ensure that the queues do not back up on to the freeway. Such a scenario can arise if the off ramp is very close to the interchange and the lane assignment on the interchange approach is not compatible with the traffic demand (turning volume demand).

5. Check that the platoon arrives during the green interval so that through vehicles do not stop at the intersection. Unnecessary and sometimes excessive delay occurs if the platoon arrives during the red interval.

Computer Analysis

1. Check that individual movements are not delayed disproportionately to one another. If they are, green splits may 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 one should correct it. If not, the green splits and/or cycle length may measure too short and should be lengthened. If through movements are oversaturated, a capacity bottleneck exists, and, thereby no progression through the intersection(s).

3. 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. Check that the progression bands are large enough to move a reasonable platoon of vehicles. This check is met if the efficiency and attainability are high, and capacity bottlenecks do not exist. If the progression bands are small, i.e., if the efficiency and attainability are small, then the offsets and phasing sequence need changing.

4.1 Evaluation Software

Analysts can perform, evaluate or simulate existing conditions using various computer simulation programs, such as the Highway Capacity Software (17), PASSER II-90 (18), PASSER III-90 (19), PASSER IV-94 (20), TRANSYT-7F (21, 22), and TRAF-NETSIM (23). The measures of effectiveness calculated by some of these programs can locate problems within the freeway corridor network and pinpoint areas requiring improvements. The following sections briefly describe each of the programs.

Highway Capacity Software (HCS). Courage and Wallace at the University of Florida developed the HCS software for the Federal Highway Administration. The program calculates saturation flow rates, average stopped delay, average travel speed, level of service, and other measures of effectiveness based on *Highway Capacity Manual* (HCM) (12) methodologies, the widely accepted standard for analysis of signalized intersections. The HCM program is straightforward and easy to use; however, one can use the program for evaluation only, and can only evaluate one intersection or one direction on the arterial at a time. For further information, consult the *Highway Capacity Software User's Manual* (17).

PASSER II-90. The Texas Transportation Institute at Texas A&M University developed the Progression Analysis and Signal System Evaluation Routine for the Texas Department of Transportation. The program analyzes and optimizes isolated intersections, and arterial streets. Features include provisions for actuated and pretimed control, an engineer's assistant key for calculating saturation flow rates using HCM methods, and the capability for modeling permitted left turns. For evaluation purposes, PASSER II estimates the measures of effectiveness for movements corresponding to NEMA phases at individual intersections, as well as overall measures of effectiveness for the entire arterial network. The measures of effectiveness used by the program include v/c ratios, delay, queues, stops, and fuel consumption. Additionally, PASSER II evaluates the progression bandwidth efficiency and attainability for the existing or optimum signal timing conditions. For further information on the program, refer to *Arterial Signal Timing Optimization Using PASSER II-90* (18).

PASSER III-90. The Progressive Analysis and Signal System Evaluation Routine, Model (PASSER) III is a fixed-time based traffic signal optimization model. The Texas Transportation Institute at Texas A&M University developed the program for the Texas Department of Transportation to determine and evaluate the optimal signal timing plan at diamond interchanges. PASSER III analyzes isolated diamond interchanges (with or without frontage roads) and/or progression for a series of diamond interchanges connected by frontage roads. The program analyzes different phasing patterns and varies the offset to minimize delay within the interchange. Researchers developed PASSER III to analyze fixed time and fixed sequence control, but the program can be used to approximate actuated control. Input requirements include turning movements, distance between intersections, average link speeds, queue clearance interval, phasing sequence and minimum green times. PASSER III-90 has a built in assistant function to calculate saturation flow rates based on the Highway Capacity Manual methodology. One can obtain further information for using PASSER III from the *PASSER III-90 User's Manual* (19).

PASSER IV-94. The Progressive Analysis and Signal System Evaluation Routine, Model (PASSER) IV is an advanced network signal timing optimization model. The program is being developed by the Texas Transportation Institute at Texas A&M University for the Texas Department of Transportation. This program is the only practical computer program that can optimize signal timings for large multi-arterial networks based on maximizing platoon progression. PASSER IV maximizes progression bandwidth on all arterials (one-way and two-way) in closed networks and explicitly handles one-way streets. The program complements PASSER II and TRANSYT-7F. In the present version of PASSER IV, the user can specify the splits and phasing sequences. Offsets however, cannot be specified. Hence, the existing conditions cannot be simulated. It is expected however, that in future versions of PASSER IV, the offsets can be specified and the program can be used as a simulation tool. One can obtain further information for using PASSER IV from *PASSER IV - A Program for Optimizing Signal Timing in Grid Networks* (20).

TRANSYT-7F. Dennis Robertson of the Transport and Road Research Laboratory in England (21) developed the **Traffic** Network **Study** Tool. Version 7 was modified to reflect North American nomenclature by the University of Florida for the Federal Highway Administration. The program can analyze and optimize signal timing on coordinated arterials and grid networks. Features include provisions for actuated and pretimed control, the capability of modeling permitted left-turn movements, and provisions for including stopped controlled intersections along the frontage roads and arterials in the freeway corridor. TRANSYT estimates measures of effectiveness for each of the movements at individual intersections as well as overall measures of effectiveness for the entire network in the freeway corridor. The measures of effectiveness used by the program include delay, queues, stops, fuel consumption, total travel, total travel time, average travel speed, and total operating cost. Additionally, TRANSYT graphically illustrates the flow profiles of the through and left-turn movements along the arterials in the network. For further information, refer to the *TRANSYT-7F Users Manual* (22).

TRAF-NETSIM. TRAF-NETSIM is a microscopic traffic network simulation model developed by Federal Highway Administration. This model can simulate traffic control systems in great detail; however, it cannot optimize signal timing. The TRAF-NETSIM model can handle isolated intersections, diamond interchanges and coordinated arterial street networks. The model can simulate uncontrolled, stop/yield controlled, pretimed and semi-actuated systems. One can also simulate fully actuated signals in isolated mode. The output includes detailed statistics on delay, stops, queues, emissions, and other variables. For further information, refer to the *TRAF User Reference Guide* (23).

These guidelines will address PASSER III, PASSER IV, and TRANSYT-7F due to their ability to both simulate and optimize network timing plans. PASSER III can evaluate and optimize isolated diamond interchanges or a series of diamond interchanges connected by one-way frontage roads. One can use PASSER IV and TRANSYT separately or in combination with one another for optimizing network signal retiming plans. All three programs run on IBM PC compatible microcomputers. For additional information, refer to the user's manuals for the programs mentioned above.

4.2 Input Requirements

This section of the guidelines discusses the coding and simulation procedures for PASSER III, PASSER IV, and TRANSYT-7F. Table 4-1 presents some of the common input requirements utilized by PASSER III, PASSER IV, and TRANSYT for coding existing conditions along frontage roads and arterial streets. One must initially code a data set containing the existing traffic signal operational conditions for the freeway corridor network to be analyzed through one of the programs. After developing the initial data set, the selected program is utilized to simulate the existing conditions in a freeway corridor. A simulation run models the arterial conditions coded in the data set and generates a performance evaluation of the conditions with respect to the modelling parameters of each program. The following sections outline the basic requirements for coding existing conditions for simulation with PASSER III, PASSER IV, and TRANSYT.

PASSER III-90. A traffic signal analyst codes data in PASSER III using three data screens: *Freeway Identification*, *Interchange and Signal Phasing Data*, and *Interchange Movement Data*. To begin entering data, the user must first access the *File Screen*. The *File Screen* allows the user to set up the file name and path for the new data set. After entering the proper path and file name, the user hits <ESC> return to the *Main Menu*, then selects the *Edit Command* to enter new data.

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

1. **Run Number.**
2. **Freeway Name.**
3. **District.** TxDOT District Number.
4. **City Name.**
5. **Number of Interchanges.** Number of interchanges being considered.
6. **Cycle Lengths.** Set a lower and upper cycle lengths appropriate to the system.
7. **Band Split Percentage.** Percentage of the band width for the peak direction.

Figure 4-1 illustrates a filled-in Freeway Identification Screen for evaluating existing conditions.

Table 4-1. Common Coding Requirements for PASSER III, PASSER IV, and TRANSYT

ARTERIAL IDENTIFICATION	Name of Arterial Segment Orientation of Segment (NS or EW) Number of Intersections Distance Between Intersections
SYSTEM-WIDE PARAMETERS	Ideal Saturation Flow Rate Operational Defaults System Cycle Length
TRAFFIC/PHASING DATA	Speeds Volumes Phasing Sequences and Duration Additional Timing Intervals

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

                        FREEWAY IDENTIFICATION

            Run Number: 04   Freeway Name: Users Manual Example
            District  : 14      City Name: San-Antonio·
            Number of Interchanges  (1 for isolated analysis) :  4
                    --------CYCLE LENGTHS----------
                    Lower: 65  Upper:  75 Incr:  5

            Calculate Band Split Proportional to Traffic?(Y/N): N
            "A" direction Percentage             ( 0 to 100 ):  50
            Speed Search?                               (Y/N): Y


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

                         Escape key to exit screen
```

Figure 4-1. Freeway Identification Screen

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

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

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

3. **Interior Travel Time.** The interior travel time is the running time from the left (right) side of the intersection to the right (left) side. The analyst may measure the travel time in the field or estimate it based on the width of the interchange, as discussed in the Data Requirements Chapter. Table 4-2 illustrates the corresponding travel times for intersection widths.

4. **Interior Queue Storage.** The interior queue storage is the number of vehicles that may be stored in the interior of the interchange. The user enters storage separately for the through and left movements. One can obtain an estimate of queue storage by assuming a vehicle occupies 25 feet of lane space. Multiple lane storage must be added and a single lane may store left and through vehicles. The storage for left-turn and through vehicles sharing a lane may be based on proportions of left-turn and through vehicles to total volume or by field observation.

5. **Signal Phasing Data.** For simulating existing conditions, answer 'N' to the optimized delay-offset for all phase sequences, but enter the existing internal offset for the corresponding existing phase sequence. Figure 4-2 illustrates the signal data screen and gives hints for simulating existing conditions. Figure 4-3 shows PASSER III phase sequence codes.

Table 4-2. Estimated Travel Times for Various Diamond Interchange Widths

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

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

                            INTERCHANGE  2 DATA

               Cross Street Name        Permitted Left Turns (Y/N)
                 Dallas-Ave--           Left-side: Y  Right-side: Y

              Interior-Travel-Time       Interior-Queue-Storage
               --Rgt-->   <--Lft--        Left-Side  Right-Side
                (Sec)       (Sec)         Left Thru  Left Thru
                  8          12             6   10    10   10

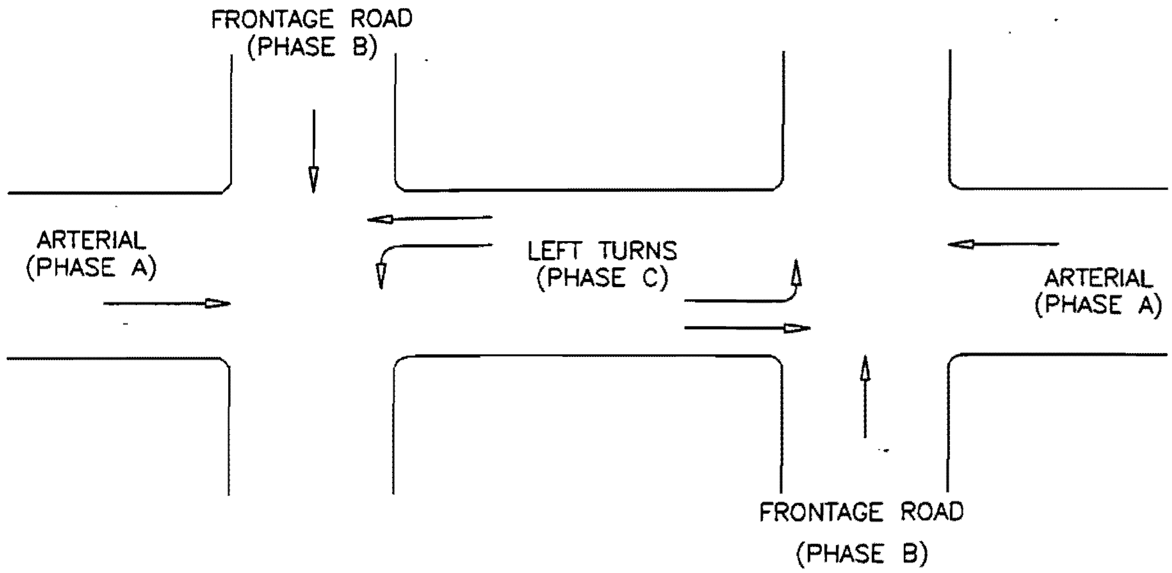
                           SIGNAL PHASING DATA

         Type of      Run Delay-Offset   ----- Seconds Offset -----
         Phasing      Analysis (Y/N)     Forced Int.    Forced Ext.
         -------           -----         -----------    -----------
         Lead-Lead           N              10
         Lag-Lead            N               0
         Lead-Lag            N              35
         Lag-Lag             N               0
         TTI-Lead            N              10

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

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

Figure 4-2. Signal Data Screen - Simulating Existing Conditions



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

Figure 4-3. PASSER III Phase Sequence Codes

Interchange Movement Screen. To enter volumes, number of lanes, and minimum phase times, this report suggests that the assistant function <F3> be used. When the data entry into the assistant window is complete, the saturation flow rates for each movement are automatically calculated when the user hits the <F3> key again or hits the <ESC> key. Figure 4-4 shows the *Interchange Movement Screen* with and without the Assistant Window.

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

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

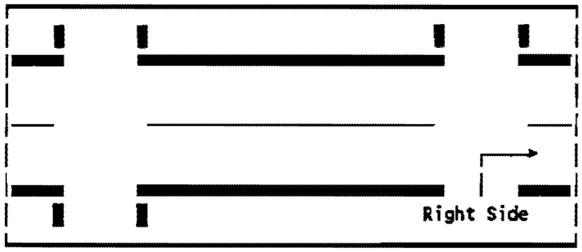
   An R for right turns, L for left turns, T for throughs and/or a U for U-turns must be assigned to at least one lane if non-zero volumes were entered for the movements. At least a partial lane for each movement must exist, and there must be **no** allowable lanes for movements that do not exist. The user should note that one may enter more than one movement per lane.

   If short, right or left-turn lanes impact an adjacent through lane's capacity, the number of through lanes should be reduced by an amount which corresponds to the loss in capacity; e.g. if a 10 percent loss in capacity occurs, the number of through lanes input to the program should be reduced by 10 percent.

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

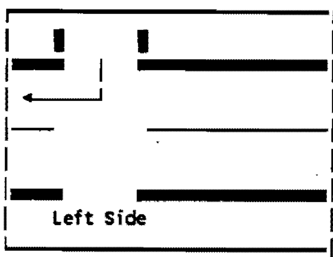
4. **Minimum Green.** For simulating existing conditions, the existing phase times (G + Y + RC) are entered instead of true minimum values. The sum of the phase times for each side should equal the existing cycle length for simulation purposes.

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




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

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




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

Figure 4-4. Interchange Movement Screen

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

Frontage Road Progression. If the number of diamond interchanges specified in the freeway identification data input screen is more than one, PASSER III assumes that the user intends to provide progression along the frontage roads. The program assumes the two one-way frontage roads as a divided arterial street. The *Frontage Road Progression Data Input Screen* gives information about frontage road progression for each direction, as illustrated in Figure 4-5. The program requires information about the distances between each interchange. The distance from stop-line to stop-line is measured. Average running speed between the interchanges is found out by the floating car technique for each of the time periods. The queue formed from side streets and parking lots on each frontage approach is estimated, and the time required to clear the queue is calculated. This will allow the calculation of the offset such that the frontage road platoon arrives after the queue has just cleared.

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

```
                    P A S S E R   III  -  88                     Version 1.0
            Texas Department of Highways & Public Transportation

                    FRONTAGE ROAD PROGRESSION DATA
        Link Name             From -----------> To    To <----------- From
  From: HOUSTON AVE           Dist  Speed  Que Clr    Dist  Speed  Que Clr
  To:   DALLAS AVE            (Ft)  (MPH)   (Sec)     (Ft)  (MPH)   (Sec)

HOUSTON AVE   DALLAS AVE      1900   35      0        1900   35      0
DALLAS AVE    EL PASO ST      2650   30      0        2650   30      0
EL PASO ST    ELGIN BLVD      2220   38      0        2220   38      0



            Esc:exit screen         PgUp/PgDn:next link
```

Figure 4-5. Frontage Road Progression Data Input Screen ("A" Direction)

PASSER IV-94. Two components make up PASSER IV; a *User Interface* and an *Optimization Module*. The *User Interface* module allows data editing and file manipulation functions, whereas the *Optimization Module* actually performs bandwidth optimization for the given traffic network. The PASSER IV *User Interface* is extremely easy to use with a minimum amount of practice. The first screen in this program is the main menu as shown in Figure 4-6. The various options in the main menu allow the user to select the function that needs to be performed. The function keys in the *File* menu allow the user to select corresponding file functions outside the *File* menu. An input data file may be created, or an existing data file can be loaded into the current screen using these options.

The *Edit* function allows easy data entry/editing of a loaded data file. It uses a free format in the sense that the user can go to any arterial or signal data entry screen in a selected order (that is, the program does not force the user to go through a fixed data entry sequence). Within the data entry hierarchy, one can use the function keys (F2 and F3) to move from one data entry scheme to another, as shown in Figure 4-7. The user can enter an arterial or a frontage road in the freeway corridor network in any order (the program will attach a sequence number to each arterial in the order entered); however, one must enter signal data within an arterial starting from the first to the last intersection in the "Forward or A-Direction," without skipping any signal data. In addition, each signal must be assigned a unique node identification number used by the program to find linkages in the network.

The traffic signal analyst needs to enter data for an intersection falling on two arteries only once. For example, if Signal 1 on Arterial 2 and Signal 3 on Arterial 5 have the same node identification number, signal data will need only be entered while entering Arterial 2 data. Figure 4-8 shows the input screens for global data. The format is slightly restrictive as compared to other network programs available to the traffic community, however, it also is very simple, since the data entry sequence informs the program about the linkage of signals, which the user has no need to provide.

The *Parameters* screen lets the user specify optimization data, including optimization steps, number of solutions to be saved at each step, and the upper limit on the number of iterations and re-inversions for branch and bound procedure and linear programs. One-Step or global optimization gives the best possible solution but also is the slowest; it applies, however to all problems. Two and Three-Step optimization are heuristic procedures which will give very good solutions in a short amount of time. The Two-Step optimization process applies to arterial as well as closed loop networks. The Three-Step optimization procedure is only valid for multi-arterial-close-loop networks. The *Information* function allows the user to view the network on the screen, as illustrated in Figure 4-9.

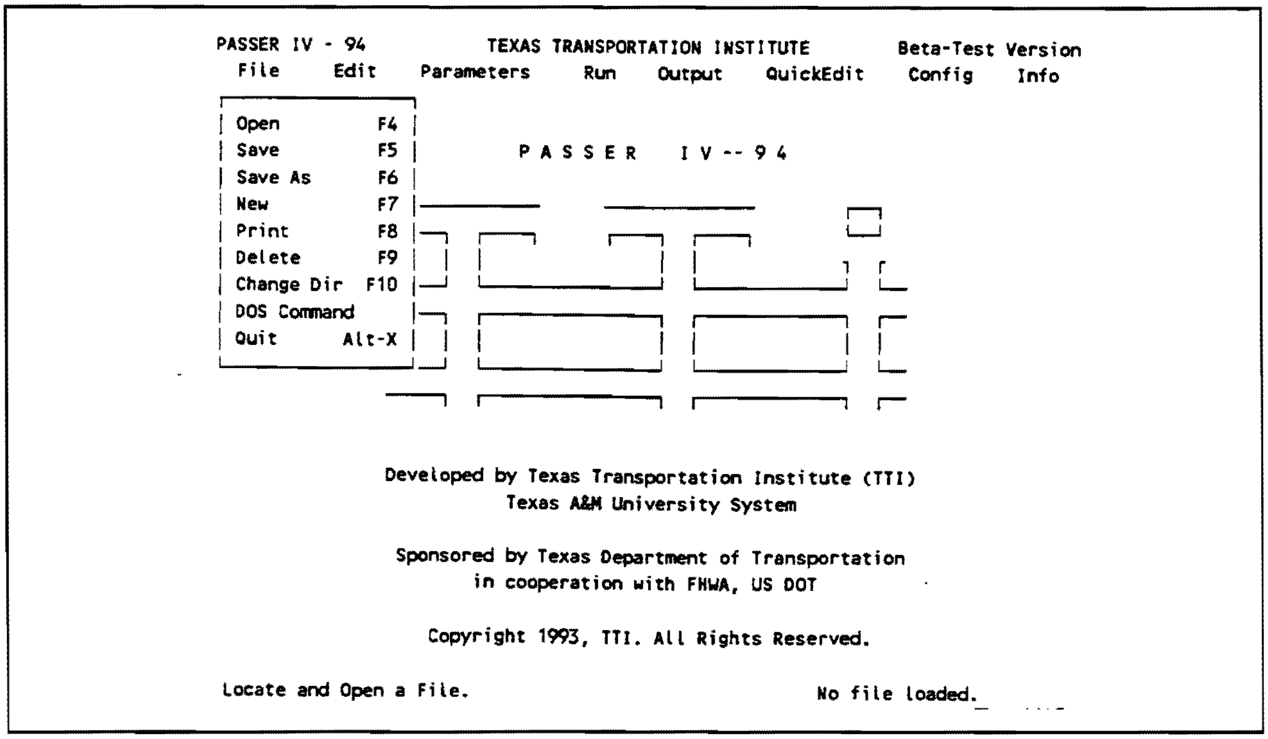


Figure 4-6. PASSER IV Main Menu Screen

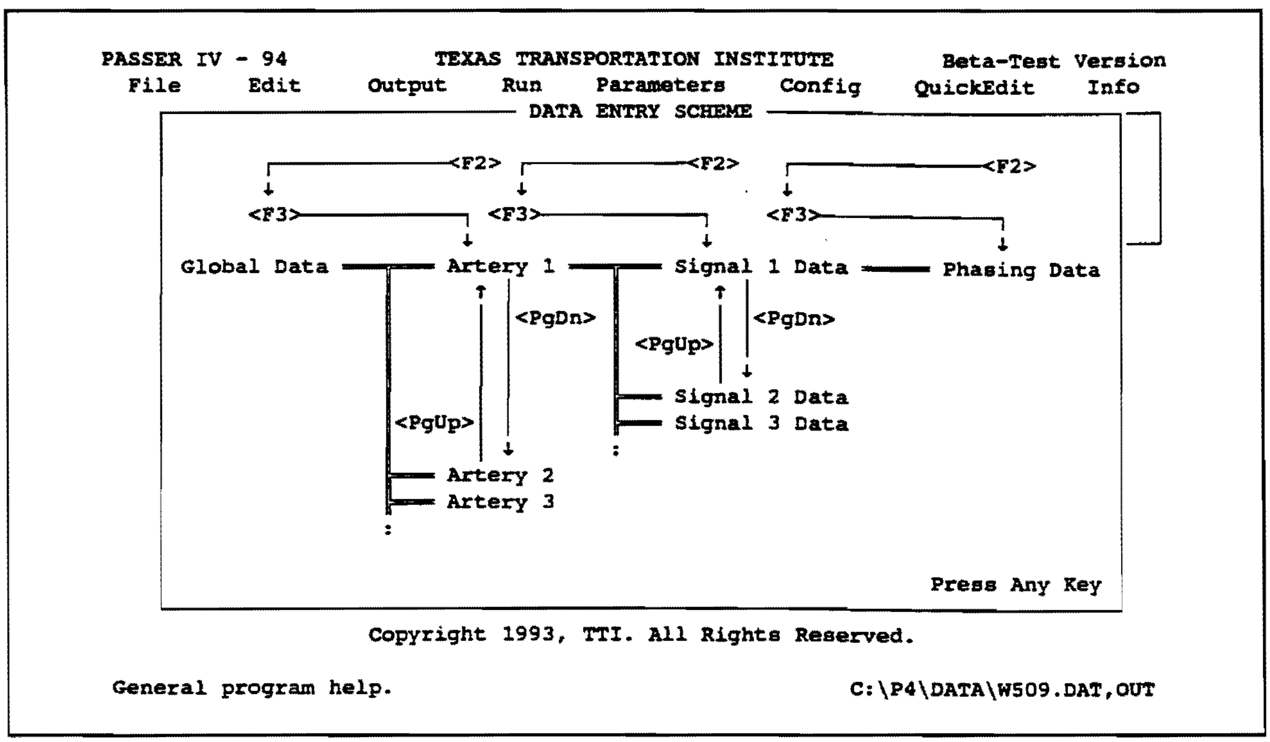


Figure 4-7. PASSER IV Data Entry Scheme

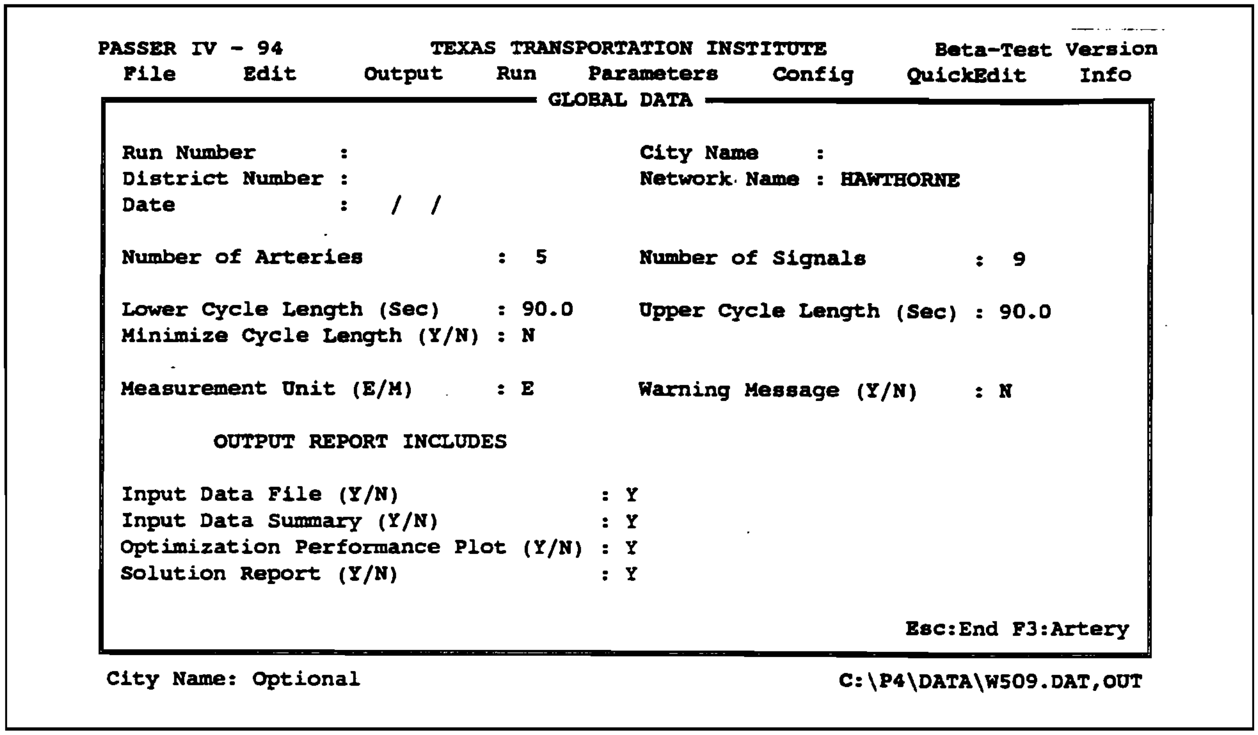


Figure 4-8. PASSER IV Global Data Entry Screen

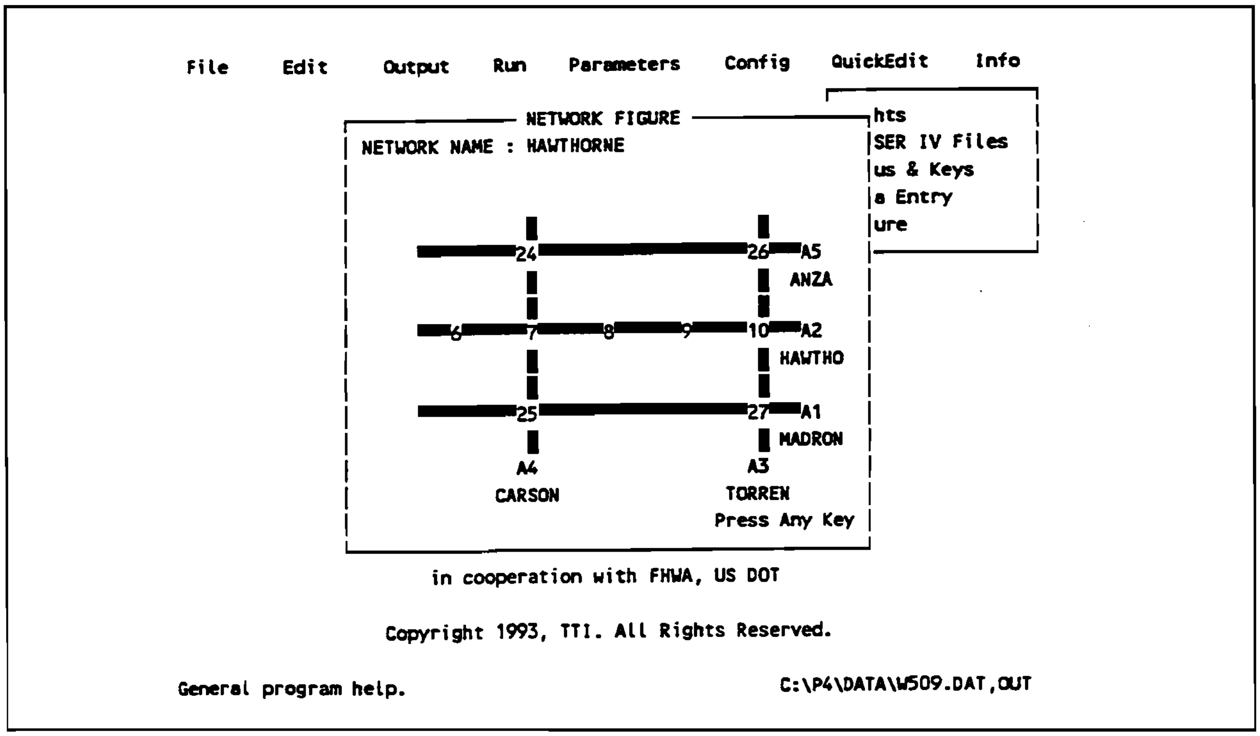


Figure 4-9. PASSER IV Screen for Network Layout

The number of steps for a given problem should be selected as the highest number possible, since that selection will require the least amount of CPU time. For example, use the Three-step optimization procedure for a network with multi-phase signals. In addition, the number of solutions saved in the last step must equal one, except when an output report for more than one signal timing solution is needed.

TRANSYT-7F. One may code freeway corridor network data through coding screens and windows accessible from TRANSYT's main menu, shown in Figure 4-10. The correct menu choice to create or edit an arterial street network data set is the TRANSYT *Data Input Manager* (DIM). After making this selection, a blank screen will appear in the create mode and a screen containing data will appear if an existing data set was loaded. Figure 4-11 shows an example of an existing data set; however, one should note that only the first 21 lines of data appear in the figure. One accesses the remainder of the data set by moving the cursor down to the bottom of the screen.

Note that a TRANSYT data set is made up of several lines of data with each line containing 16 five column fields. To view column headings or enter data, the analyst can move the cursor to the line of interest and press the [Esc] key. This key causes a window with column headings and current data to appear on the right side of the screen. One can edit data within the window, and update the data file when exiting the window. Figure 4-12 shows a data entry window for Card Type 1 (second line of data). Note that cycle length and analysis period length are entered on this data card.

Card Type 2 is an optimization node list card used to list the nodes for which the signal timing is to be optimized during an optimization run. Card Type 7 is the shared stop-line card used to list the sets of links that share the same stop-line in the network. This card can be used whenever more than one link will end at a single downstream node, and it can be used for any grouping of links that share the same saturation flow rate and signal timing. Card Type 10, the network master controller card, is used to code information about the system controllers at all nodes and/or links in the network. If the default values are used, then the network master card need not be included in the input data deck.

Card Types 15 through 28 contain intersection information, such as phase lengths and sequences, traffic volumes, and link speed and distance data. This type of information will be repeated for each intersection in the arterial system. Figures 4-13, 4-14, and 4-15 illustrate *Interval Timing*, *Signal Phasing*, and *Link Data* respectively. As should be apparent, TRANSYT requires more detailed data and a more complex data entry scheme than PASSER IV.

```
                McT7F - TRANSYT-7F MAIN MENU

          T ---> TRANSYT-7F Program
          R ---> Read Input/Output Files
          A ---> Alter Data Files (T7FDIM)
          N ---> New File (T7FGEN)
          S ---> Send Output File to Printer
          Y ---> Your Special Program
          T ---> TRANSYT-7F Program
          + ---> EZ-TRANSYT PLUS
          7 ---> 7F PPD Program
          F ---> File Management

         Copyright 1988, University of Florida

       Enter your Choice [Quit, eXit  or Esc]
```

Figure 4-10. TRANSYT-7F Main Menu

```
TRANSYT-7F DATA INPUT MANAGER                                  Version 4.1
  1    2    3    4    5    6    7    8    9   10   11   12   13   14   15   16
-RIDGEWOOD AVE     CHAP 7 BWC EX    BANDWIDTH CONSTRAINED OPTIMIZATION       -
[  1  120  120    0    0    0    3    3   -1    0    0   60    0    0    0    0]
[  2    1    2    3    4    0    0    0    0    0    0    0    0    0    0    0]
[  7  105  106    0    0    0  107  108    0    0    0  205  206    0    0    0]
[  7  207  208    0    0    0  405  406    0    0    0  407  408    0    0    0]
[ 10    0    0    0    0    0    0    0    0    1    0    0    0    0    0    0]
[ 13    1    0    1   14    4   56    4   38    4    0    0    0    0    0    0]
[ 21    1    1    1    2    0   10  102  104    0    0    0    0    0    0    0]
[ 22    1    3    3    4    0   15  101  103    0    0    0    0    0    0    0]
[ 23    1    5    5    6    0   15  105 -106  107 -108    0    0    0    0    0]
[ 28  101  100 3400  640    0    0    0   35    0    0    0    0    0    0    0]
[ 28  102  100 1600   46    0    0    0   35    0    0    0    0    0    0    0]
[ 28  103  766 3400  856    0  203  738   35  208   90   35  205   27   35    0]
[ 28  104  766 1600  110    0  203  110   35    0    0    0    0    0    0    0]
[ 28  105    0 3400  426    0    0    0    0    0    0    0    0    0    0    0]
[ 28  106    0    0   48    0    0    0    0    0    0    0    0    0    0    0]
[ 29  106    0    0    0    2    0    0    0  107    0    0    0    0    0    0]
[ 28  107    0 3400  214    0    0    0    0    0    0    0    0    0    0    0]
[ 28  108    0    0   86    0    0    0    0    0    0    0    0    0    0    0]
[ 29  108    0    0    0    2    0    0    0  105    0    0    0    0    0    0]
     [F1] for Command Mode , [Esc] for Window mode. [?] for card list
 File Name:   CHAP7.DAT          BROWSE MODE       Card No. 1  OF 72
```

Figure 4-11. Example of an Existing Data Set

```
TRANSYT-7F DATA INPUT MANAGER                                   Version 4.1
  1    2    3    4    5    6    7    8    9   10   11   12   13   14   15   16
-RIDGEWOOD AVE      CHAP 7 BWC EX      BA
[  1  120  120   0    0    0    3       CARD TYPE   1         SYSTEM CONTROLS
[  2    1    2   3    4    0    0
[  7  105  106   0    0    0  107  10   MINIMUM CYCLE..................[120  ]
[  7  207  208   0    0    0  405  40   MAXIMUM CYCLE..................[120  ]
[ 10    0    0   0    0    0    0      CYCLE INCREMENT................[0    ]
[ 13    1    0   1   14    4   56      SEC/STEP (CYC).................[0    ]
[ 21    1    1   1    2    0   10  10   SEC/STEP (OPT).................[0    ]
[ 22    1    3   3    4    0   15  10   START LOST TIME................[3    ]
[ 23    1    5   5    6    0   15  10   EXT. EFFECTIVE GREEN...........[3    ]
[ 28  101  100 3400  640   0    0      STOP PENALTY...................[-1   ]
[ 28  102  100 1600   46   0    0      OUTPUT LEVEL...................[0    ]
[ 28  103  766 3400  856   0  203  73   INITIAL TIMING (Y=1)...........[0    ]
[ 28  104  766 1600  110   0  203  11   PERIOD LENGTH..................[60   ]
[ 28  105    0 3400  426   0    0      SEC(0) / PERCENT(1)............[0    ]
[ 28  106    0    0   48   0    0      SPEED(0) / TIME(1).............[0    ]
[ 29  106    0    0    0   2    0      U.S.(0) / METRIC(1)............[0    ]
[ 28  107    0 3400  214   0    0      PUNCH (Y=1)....................[0    ]
[ 28  108    0    0   86   0    0
[ 29  108    0    0    0   2    0
     [PgUp] [PgDn] to change cards ... [Esc] to return to Browse Mode.
  File Name:    CHAP7.DAT              WINDOW MODE        Card No. 2  OF 72
```

Figure 4-12. Data Entry Window for Card Type 1

```
TRANSYT-7F DATA INPUT MANAGER                                   Version 4.1
  1    2    3    4    5    6    7    8    9   10   11   12   13   14   15   16
-EXTENDED HAWTHORNE. OPT OFF. MAX SEQ
[  1   90   90    5    5    1    2       CARD TYPE [ 15]       INTERVAL TIMING
[  2    3    4    5    6    7    8
-****4    2   15   40   15    2          NODE NUMBER....................[3    ]
-****6  500  500  500  100    1          OFFSET OR YIELD PT.............[84   ]
[  4   15   40   -2   15   40    2    -  REFERENCE INTERVAL.............[1    ]
[  6 1000 1000  100  100  100   10       INTERVAL LENGTH 1..............[7    ]
[  7  401  402  409    0    0  305  31   .               2..............[3    ]
[  7  407  412                 501  50   . (NOTE: IF THE 3..............[5    ]
[  7  601  609                 603  61   . '1X' IS NEG-  4..............[1    ]
[  7  705  711                 707  71   . ATIVE (-1X)   5..............[47   ]
[  7  801  802  809            803  81   . THE SPLITS    6..............[3    ]
[  7  807  812                1005 101   . WILL NOT BE   7..............[18   ]
[  7 1101 1102 1109           1103 110   . OPTIMIZED.)   8..............[1    ]
[  7 1107 1112                1201 120   .               9..............[12   ]
[  7 1205 1211                1207 121   .              10..............[3    ]
[  7 1305 1311                1307 131   .              11..............[     ]
[  7 1407 1412                1503 150   DOUBLE CYCLE? (Y=1)............[     ]
[ 10    3    3    0    0   35   35  10
[ 15    3   84    1    7    3    5
     [PgUp] [PgDn] to change cards ... [Esc] to return to Browse Mode.
  File Name:    B:HAWT1.DAT            WINDOW MODE        Card No. 20  OF 219
```

Figure 4-13. TRANSYT Coding Screen for Interval Timing

```
TRANSYT-7F DATA INPUT MANAGER                                       Version 4.1
   1    2    3    4    5    6    7    8    9   10   11   12   13   14   15   16
[  1   90   90    5    5    1    2
[  2    3    4    5    6    7    8     CARD TYPE [ 21]          SIGNAL PHASING
-****4    2   15   40   15    2
-****6  500  500  500  100    1        NODE NUMBER....................[3     ]
[  4   15   40   -2   15   40    2   - START INTERVAL.................[1     ]
[  6 1000 1000  100  100  100   10     VARIABLE INTERVAL..............[1     ]
[  7  401  402  409    0    0  305  31 YELLOW INTERVAL................[2     ]
[  7  407  412                 501  50 ALL-RED INTERVAL...............[      ]
[  7  601  609                 603  61 MINIMUM PHASE LENGTH...........[9     ]
[  7  705  711                 707  71 LINKS SERVICED: 1..............[306   ]
[  7  801  802  809            803  81 .               2..............[308   ]
[  7  807  812                1005 101 .  (NEGATIVE    3..............[      ]
[  7 1101 1102 1109           1103 110 .   NUMBERS     4..............[      ]
[  7 1107 1112                1201 120 .     MEAN      5..............[      ]
[  7 1205 1211                1207 121 .  PERMITTED.)  6..............[      ]
[  7 1305 1311                1307 131 .               7..............[      ]
[  7 1407 1412                1503 150 .               8..............[      ]
[ 10    3    3    0    0   35   35  10 ACTUATED OR DESIRED D/S........[      ]
[ 15    3   84    1    7    3    5
[ 21    3    1    1    2         9  30
     [PgUp] [PgDn] to change cards ... [Esc] to return to Browse Mode.
 File Name:    B:HAWT1.DAT          WINDOW MODE         Card No. 21  OF 219
```

Figure 4-14. TRANSYT Coding Screen for Signal Phasing

```
TRANSYT-7F DATA INPUT MANAGER                                       Version 4.1
   1    2    3    4    5    6    7    8    9   10   11   12   13   14   15   16
[  7  401  402  409    0    0  305  31
[  7  407  412                 501  50 CARD TYPE    28                LINK DATA
[  7  601  609                 603  61
[  7  705  711                 707  71 LINK NUMBER....................[301   ]
[  7  801  802  809            803  81 LINK LENGTH....................[500   ]
[  7  807  812                1005 101 SATURATION FLOW................[1500  ]
[  7 1101 1102 1109           1103 110 TOTAL VOLUME...................[90    ]
[  7 1107 1112                1201 120 MIDBLOCK ENTRY.................[      ]
[  7 1205 1211                1207 121 1ST INPUT LINK NO..............[      ]
[  7 1305 1311                1307 131 .           VOLUME.............[      ]
[  7 1407 1412                1503 150 .            SPEED.............[      ]
[ 10    3    3    0    0   35   35  10 2ND INPUT LINK NO..............[      ]
[ 15    3   84    1    7    3    5     .           VOLUME.............[      ]
[ 21    3    1    1    2    0    9  30 .            SPEED.............[      ]
[ 22    3    3    3    4    0    5  30 3RD INPUT LINK NO..............[      ]
[ 23    3    5    5    6    0   22  30 .           VOLUME.............[      ]
[ 24    3    7    7    8    0    9  30 .            SPEED.............[      ]
[ 25    3    9    9   10    0    9  30 QUEUEING CAPACITY..............[      ]
[ 28  301  500 1500   90
[ 28  302  500 1500   80
     [PgUp] [PgDn] to change cards ... [Esc] to return to Browse Mode.
 File Name:    B:HAWT1.DAT          WINDOW MODE         Card No. 26  OF 219
```

Figure 4-15. TRANSYT Coding Screen for Link Data

4.3 Output Evaluation

This section discusses the generation and interpretation of the output from simulation runs by PASSER III, PASSER IV-94, and TRANSYT-7F. After completing a simulation run, an output file of the simulation results is created. Because PASSER III, PASSER IV, and TRANSYT are signal optimization programs, the output generated by each program is oriented toward developing an optimal set of signal timing parameters. This section will focus upon the components of the output critical for evaluating existing conditions. In general, output files consist of an echo of the information coded in the initial input data file and a listing of the MOEs calculated for the existing or optimized traffic operational conditions.

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

The output data contained for each screen in the output is explained below:

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

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

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

Link Geometry Data. The distances between the interchanges, progression speeds, and the queue clearance time at each interchange are listed.

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

Optimal Progression Solution. If the interchanges are analyzed as a system, the Optimal Progression Solution is generated. Figure 4-16 illustrates the *Optimal Progression Solution* screen. The screen illustrates the system cycle length, progression speed and bandwidth for each direction, efficiency, and attainability.

*** OPTIMAL PROGRESSION SOLUTION *** RUN 6 PAGE 11A

N CENTRAL FREEWAY

* *

CYCLE LENGTH (SEC) - - - - - - 65

'A' DIRECTION

PROGRESSION SPEED (MPH) - - 30.0

BANDWIDTH (SEC) - - - - - - 15.0

'B' DIRECTION

PROGRESSION SPEED (MPH) - - 33.0

BANDWIDTH (SEC) - - - - - - 15.0

EFFICIENCY - - - - - - - - - - .23

ATTAINABILITY - - - - - - - - - 1.00

Pg 1 <Esc>exit <PgUp><PgDn> <F2>Print Page <F3>Print Section 1 Pg

Figure 4-16. Optimal Progression Solution

*** FRONTAGE ROAD PROGRESSION INFORMATION *** RUN 6 PAGE 11B

N CENTRAL FREEWAY

INTERCHANGE CROSS STREET NAME	SIGNAL PHASING SEQUENCE	INTERNAL OFFSET (SEC)	EXTERNAL OFFSET (SEC)	'A' DIRECTION TRAVEL TIME (SEC)	'B' DIRECTION TRAVEL TIME (SEC)
SOUTHWESTERN	LAG -LAG	0	.0	25.4	181.4
LOVERS LANE	LAG -LEAD	35	51.1	109.9	118.7
UNIVERSITY	LEAD-LAG	35	69.2	185.0	71.1
YALE	LAG -LAG	0	74.2	225.9	25.0

Pg 1 <Esc>exit <PgUp><PgDn> <F2>Print Page <F3>Print Section 1 Pg

Figure 4-17. Frontage Road Progression Information

Frontage Road Progression Information. For a series of interchanges, the progression information is given on the *Frontage Road Progression Information* screen, as illustrated in Figure 4-17. This screen gives information about the signal phasing sequence at each of the interchanges. The program gives the internal offset at each of the interchanges corresponding to the selected phasing sequence. The external offset (the difference in seconds between the start of the arterial phase on successive interchanges on the left side of the interchange) is given, followed by the travel times along the frontage roads in both the directions.

General Signalization Information. This screen contains the phase times, v/c ratio, delay, and storage ratio for each movement for the left and right side of the interchange. The user also notes the total interchange delay, phase order, and internal offsets. PASSER III also assigns levels of service to various measures of effectiveness, such as delay, v/c ratio, and the storage ratio. Table 4-3 shows level of service criteria used by PASSER III. Figure 4-18 illustrates an example of the *General Signalization Information* screen.

Signal Phasing Information. This screen gives the phase interval number, the left side sequence, the right side sequence, and the corresponding phase interval length. The user also notes the cycle length, phase order, and internal offset. An example of the *Signal Phasing Information* screen is illustrated in Figure 4-19.

Time-Space Diagram. If the interchanges are analyzed as a system, the output generates a Time-Space Diagram. The diagram illustrates the city name, district, run number, freeway name, and cycle length. Additionally, the output also contains the average progression speeds and the bandwidths in the "A" and "B" directions.

Table 4-3. Level of Service Criteria for Operational Measures of Effectiveness at Signalized Diamond Interchanges

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

[a]Source. Guide for Designing and Operating Signalized Intersections in Texas (24).
[b]Highway Capacity Manual, 1985 - stopped delay multiplied by 1.3 for total delay (12).
[c]PASSER III-84 User's Manual.

```
* * * INTERCHANGE  1 SOUTHWESTERN                                  RUN 6   PAGE 12A

              ***   GENERAL SIGNALIZATION INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
 MEASURES OF         LEFT-SIDE INTERSECTION   *    RIGHT-SIDE INTERSECTION
EFFECTIVENESS        A      B      C     A+C  *    A      B      C     A+C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                                              *
 PHASE TIME (SEC)  21.1   15.0   28.9   50.0  *  28.9   21.0   15.1   44.0
                                              *
 V/C RATIO           .73    .56    .73    .39 *    .67    .22    .70    .19
  LEVEL OF SERVICE  C      A      C      A    *   B      A      C      A
                                              *
 DELAY (SEC/VEH)   42.39  27.01   7.42   1.25 *  20.24  18.86   7.90   1.32
  LEVEL OF SERVICE  D      C      B      A    *   C      B      B      A
                                              *
 STORAGE RATIO                     .76    .18 *                  .36    .20
  LEVEL OF SERVICE                 E      C   *                  D      C
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   PHASE ORDER    LAG -LAG      TOTAL INTERCHANGE DELAY   15.08 VEH-HRS/HR
   INTERNAL OFFSET   0 SEC      CYCLE LENGTH   65 SEC

 Pg 1   <Esc>exit  <PgUp><PgDn>  <F2>Print Page <F3>Print Section   8 Pgs
```

Figure 4-18. General Signalization Information

```
* * * INTERCHANGE  1 SOUTHWESTERN                                  RUN 6   PAGE 16A

                      ***   SIGNAL PHASING INFORMATION   ***

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                       *                      *
                * LEFT-SIDE SEQUENCE    * RIGHT-SIDE SEQUENCE  *
                *   A      C      B     *  A      C      B     *
PHASE INTERVAL  * <----  <----    |     * <----     ^     ^    *  PHASE INTERVAL
    NUMBER      *        |----    |     *        ----|    |    *   LENGTH (SEC)
                * ---->  V        V     * ---->  ---->    |    *
                *                       *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
                *                       *                      *
       1        *        A              *         A            *       21.10
       2        *        C              *         A            *        7.80
       3        *        C              *         C            *       15.10
       4        *        C              *         B            *        6.00
       5        *        B              *         B            *       15.00
                *                       *                      *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   INTERNAL OFFSET     0 SEC                        CYCLE LENGTH   65 SEC
   EXTERNAL OFFSET    .0 SEC                        PHASE ORDER LAG -LAG

 Pg 1   <Esc>exit  <PgUp><PgDn>  <F2>Print Page <F3>Print Section   4 Pgs
```

Figure 4-19. Signal Phasing Information

PASSER IV Output. The output report for a PASSER IV optimization run includes input data file, input data summary, optimization performance plot, and the solution report. The input data summary consists of a report of the input conditions coded into the original data file. This input report is typically referred to as an "input echo" and includes the map, speed, length, and volume for each artery in the freeway corridor network. If the program detects coding errors, they will be listed either with the input data or on the error message page generated after the input data, depending on when the errors are detected.

Pages summarizing the optimization performance plots and optimization statistics follow the input echo. The optimization performance plot includes the optimization steps, number of solutions saved at each step, and the upper limit on the number of iterations and re-inversions for branch and bound procedure and linear programs. Optimization statistics include the best objective function value of final solution and the total number of iterations and re-inversions performed. The objective function value is represented in fraction of cycle length, and it equals twice the overall network efficiency. The user obtains the average arterial efficiency by dividing the network efficiency by the number of arterials in the network.

The solution report includes the summary of intersection numbers at artery meetings, network wide cycle length, and the bandwidths for each artery in fraction of the cycle length as well as in seconds. Figure 4-20 illustrates the PASSER IV solution report. For each artery, the solution report consists of artery-wide information, intersection information, phase settings for each node, and progression times and speeds. The time-space diagram provides a graphical illustration (Figure 4-21) of the timing sequence along each arterial and can be used to plot progression bands for through movements along each arterial segment in the network. A summary of the best signal timing solution for each node in the network is also included in the solution report.

The present version of PASSER IV does not calculate MOEs for the freeway corridor network. One can calculate the MOEs by using an evaluation software, such as TRAF-NETSIM. Section 5.0 further discusses this procedure relative to network optimization.

```
              **** PASSER-IV SOLUTION REPORT ****

RUN NUMBER              :     1

DISTRICT                :
NAME OF CITY            : Los Angeles
NAME OF NETWORK         : HAWTHORNE
NUMBER OF ARTERIES      :     5
NUMBER OF SIGNALS       :     9

MEASUREMENT UNITS       : ENGLISH


           INTERSECTION NUMBERS AT ARTERY MEETINGS
           -------------------------------------

  ARTERIES  1 AND  4 INTERSECT AT SIGNALS  1 AND  3, RESPECTIVELY.
  ARTERIES  1 AND  3 INTERSECT AT SIGNALS  2 AND  3, RESPECTIVELY.
  ARTERIES  4 AND  2 INTERSECT AT SIGNALS  2 AND  2, RESPECTIVELY.
  ARTERIES  2 AND  3 INTERSECT AT SIGNALS  5 AND  2, RESPECTIVELY.
  ARTERIES  3 AND  5 INTERSECT AT SIGNALS  1 AND  2, RESPECTIVELY.
  ARTERIES  4 AND  5 INTERSECT AT SIGNALS  1 AND  1, RESPECTIVELY.


                  **** NETWORK SOLUTION ****


NETWORK WIDE CYCLE TIME:   80.00 SECS

        BANDWIDTHS - PERCENTAGE OF CYCLE LENGTH (SECONDS)
        ------------------------------------------------

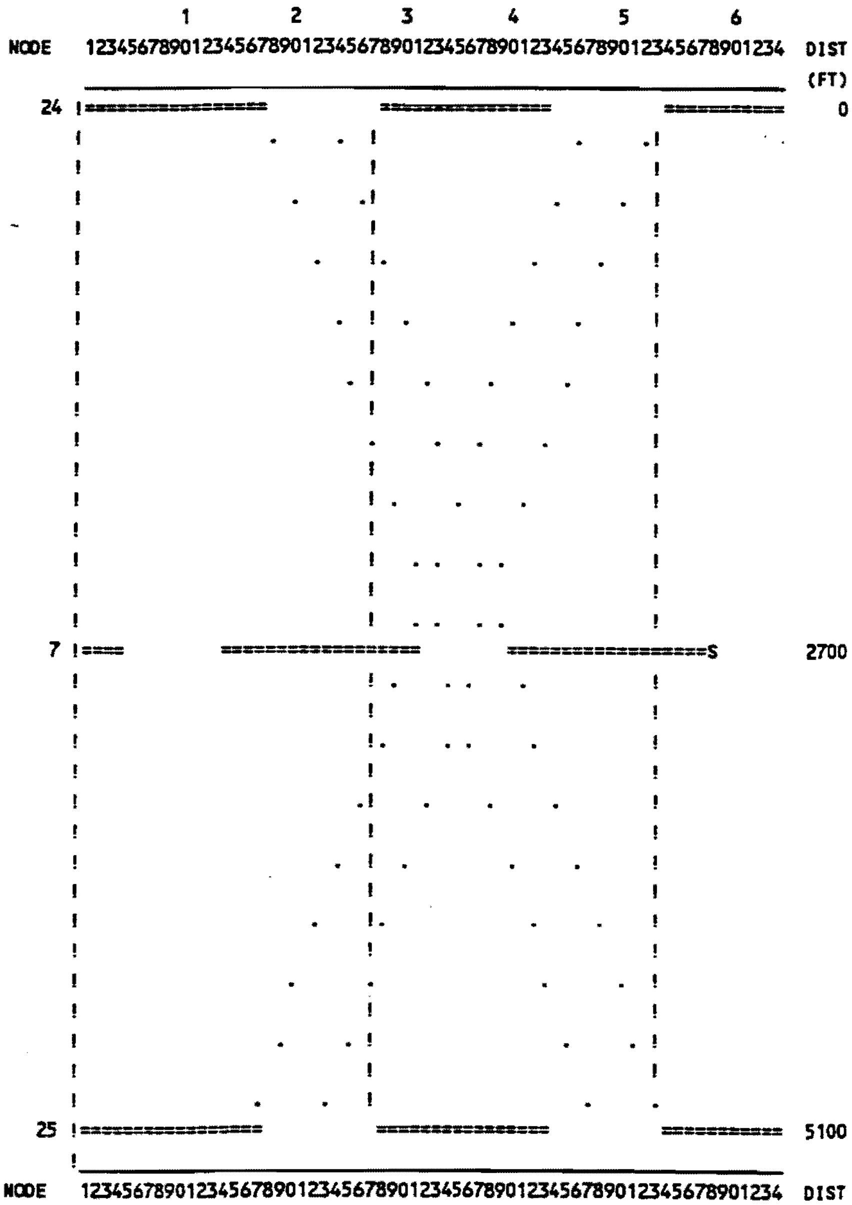
ARTERY 1:    EASTBOUND: .5000 (40.00)   WESTBOUND: .5000 (40.00)
ARTERY 2:    EASTBOUND: .2919 (23.36)   WESTBOUND: .2919 (23.36)
ARTERY 3:   SOUTHBOUND: .3124 (24.99)  NORTHBOUND: .3124 (24.99)
ARTERY 4:   SOUTHBOUND: .2312 (18.49)  NORTHBOUND: .2312 (18.49)
ARTERY 5:    EASTBOUND: .4885 (39.08)   WESTBOUND: .4885 (39.08)

TOTAL BANDWIDTH:        3.648114
```

Figure 4-20. PASSER IV Solution Report

```
                    *** PASSER-IV TIME-SPACE DIAGRAM ***

      NAME OF ARTERY: CARSON                      TIME  SCALE =   3 SEC/CHAR
      CYCLE LENGTH  :  80.00 SECONDS              DIST. SCALE =  150 FT/LINE
```




```
NODE    24 ---> NODE    25: SOUTHBOUND: DIRECTION-DOWNWARDS : GREEN -  SSSSSS
NODE    25 ---> NODE    24: NORTHBOUND: DIRECTION-UPWARDS   : GREEN -  NNNNNN
SOUTHBOUND BAND =  18.5 SECS AT 38.0 MPH           GREEN IN BOTH DIRECTIONS
NORTHBOUND BAND =  18.5 SECS AT 38.0 MPH    ====== RED   IN BOTH DIRECTIONS
```

Figure 4-21. PASSER IV Time-Space Diagram

TRANSYT-7F Output. When entering data using TRANSYT-7F, the user may designate a portion of the plots and diagrams to be included in the output. The TRANSYT-7F user's manual (22) can be consulted for further information relative to the type of output information to be generated. In general, TRANSYT will produce an echo of the input data, intersection and system-wide performance evaluations, intersection controller settings, a time-space diagram, and flow profile diagrams after a simulation run has been completed.

The table(s) of performance with initial settings provides a summary of the operational performance parameters associated with each of the movements for the intersections in the arterial street network. A separate table is produced for each intersection in the network. For simulation and evaluation purposes, one can use the initial settings table(s) to identify specific bottlenecks and locations along the arterial where operational improvements could possibly be made. The program reports the performance measures for the entire arterial street network in the system-wide table, which also includes the performance index calculated by TRANSYT-7F. Flow profile diagrams illustrate the position of the platoon with respect to the arterial through green phase. Ideally, most vehicles would arrive during the green phase.

Figure 4-22 illustrates the individual intersection best solution and performance with initial setting reports generated by TRANSYT-7F. Figure 4-23 shows system-wide performance measures. Individual link performance measures can also be obtained for each arterial in the network (Figure 4-24). Time-space diagrams are generated, which provide a graphical illustration of the timing sequence at the intersections along the arterial. The user can determine the bandwidth by extending two parallel lines through the green window at each intersection and then measuring its width. Flow profile diagrams for TRANSYT-7F, illustrated in Figure 4-25, can be generated for each intersection along the arterial. These diagrams graphically depict the arrival and departure pattern of vehicles during green and red. On the flow profile diagram, an "I" symbol indicates an arrival on red that queues, an "S" symbol indicates the departure of a queued vehicle, and an "O" symbol indicates an arrival as well as a departure on green.

EXTENDED HAWTHORNE. OPT OFF. MAX SEQ 1. CYCLE: 90 SECONDS, 60 STEPS PAGE 12

<PERFORMANCE WITH INITIAL SETTINGS>

NODE NO.	LINK NO.	FLOW (VEH/H)	SAT FLOW (VEH/H)	DEGREE OF SAT (%)	TOTAL TRAVEL (VEH-MI/H)	TOTAL TIME (VEH-H/H)	DELAY UNIFORM (VEH-H/H)	DELAY RANDOM (VEH-H/H)	DELAY TOTAL (VEH-H/H)	AVERAGE DELAY (SEC/VEH)	UNIFORM STOPS (VEH/H;%)	MAX BACK OF QUEUE (VEH/LK)	QUEUE CAPACITY (VEH/LK)	FUEL CONSUM (GA/H)	PHASE LENGTH (SEC)	LINK NO.
3	301	90	1500	54	8.50	1.34	.95	.16	1.10	44.0	81.9(91%)	2	20	1.74	13	301
3	302	80	1500	48	7.56	1.16	.83	.11	.94	42.5	72.7(91%)	2	20	1.52	13	302
3	303	296	4500	22	27.96	2.76	1.94	.02	1.96	23.8	210.1(71%)	5	60	4.06	30	303
3	304	178	1500	40	16.81	1.79	1.24	.06	1.30	26.3	135.0(76%)	3	20	2.60	30	304
3	305	1520	4500P	80	143.56	13.43	8.60	.70	9.30	22.0	1212.0(80%)	35	60	21.31	45	305
3	306	16	1500	16	1.51	.23	.18	.01	.18	41.6	14.6(91%)	0	20	.36	9	306
3	307	442	4500	18	76.90	2.50	.76	.01	.77	6.3	102.3(23%)	3	110	4.46	51	307
3	308	286	2400	77	49.76	5.97	4.27	.60	4.87	61.3	285.5(100%)	7	37	8.88	15	308
3	309	158	1500	95*	14.92	4.19	1.74	2.02	3.76	85.7	151.5(96%)	4	20	4.45	13	309
3	311	152	305S	80	14.36	1.34	.86	.07	.93	22.0	121.2(80%)	305	305S	2.13	45	311
3 :		3218	MAX =	95*	361.84	34.71	21.37	3.75	25.12	28.1	2386.8(74%)			51.51	PI =	27.8

Figure 4-22. TRANSYT-7F Intersection Performance Summary Table

CYCLE: 80 SECONDS, 60 STEPS

<SYSTEM WIDE TOTALS INCLUDING ALL LINKS>

PERFORMANCE MEASURES	TOTAL TRAVEL V-MI	TOTAL TIME V-HR	TOTAL DELAY V-HR	AVG. DELAY SEC/V	UNIFORM STOPS NO. (%)	FUEL CONS. GA	SYSTEM SPEED MI/H	TOTAL COST	PERFORMANCE INDEX
<TOTALS>	5454	226	43	5.8	7655(29)	299	25.7	1015	42.9

NOTE: PERFORMANCE INDEX IS DEFINED AS:

PI = DELAY + STOPS

NO. OF SIMULATIONS = 359 NO. OF LINKS =2890 ELAPSED TIME = 197.5 SEC.

Figure 4-23. TRANSYT-7F Table of System Performance Measures

CYCLE: 90 SECONDS, 60 STEPS

<ROUTE SUMMARY REPORT>

7F HAWTHORNE (ARTERIAL 2)

MOVEMENT/ NODE NOS.	V/C (%)	TOTAL TRAVEL (V-MI)	TOTAL TIME (V-HR)	TOTAL DELAY (V-HR)	AVG. DELAY (SEC/V)	UNIFORM STOPS NO. (%)	MAX. BACK OF QUEUE NO.	CAP.	FUEL CONS. (GA)
605	: 37	278.37	12.55	3.22	8.3	620.(44)	17	168	17.24
705	: 53	343.68	18.01	6.49	14.4	991.(61)	26	179	24.36
805	: 38	264.74	10.76	1.89	4.0	510.(30)	14	133	15.10
905	: 42	215.91	8.53	1.29	2.3	448.(22)	14	90	12.31
1005	: 66	307.96	19.77	9.45	17.6	1354.(70)	35	134	27.03
DOWN	: 66	1410.66	69.63	22.33	9.3	3924.(45)			96.03
607	: 21	166.54	7.03	1.44	6.6	294.(37)	8	179	9.54
707	: 27	123.63	6.94	2.80	12.0	435.(52)	11	125	9.53
807	: 19	85.85	3.72	.85	3.8	221.(27)	6	90	5.37
907	: 18	142.85	5.22	.43	1.7	155.(17)	4	134	7.06
1007	: 34	231.68	11.94	4.17	16.8	556.(62)	14	219	15.65
UP	: 34	750.56	34.84	9.68	8.3	1661.(39)			47.15

Figure 4-24. TRANSYT-7F Table of Link Performance Measures

```
        LINK   203    MAX FLOW  3400 VEH/H     PLT. INDEX  .52

4000+
    :
    :
    :
    :           SSSSSS
    :           SSSSSS
3000+           SSSSSS
    :           SSSSSS
    :           SSSSSS
    :           SSSSSSS
    :           SSSSSSS
    :           SSSSSSS
2000+           SSSSSSS       OOOOO
    :           SSSSSSS     OOOOOOO
    :          SSSSSSSS   OOOOOOOOOO
    :          SSSSSSSS   OOOOOOOOOO
    :          SSSSSSSS  OOOOOOOOOOOO
    :          SSSSSSSS OOOOOOOOOOOOO
1000+          SSSSSSSS OOOOOOOOOOOOOI
    :IIII      SSSSSSSS OOOOOOOOOOOOOII              II        II
    :IIIIIII   SSSSSSSSOOOOOOOOOOOOOOOIIIII         III    IIIIII
    :IIIIIIIII SSSSSSSSOOOOOOOOOOOOOOOIIIIIIIIIIII IIIII  IIIIIII
    :IIIIIIIIIIOOSSSSSSOOOOOOOOOOOOOOOIIIIIIIIIIIIIIIIIIIIIIIIIII
    :IIIIIIIIIIOOOOOSSSOOOOOOOOOOOOOOOIIIIIIIIIIIIIIIIIIIIIIIIIII
     **********                       ***************************
     123456789012345678901234567890123456789012345678901234567890
```

Figure 4-25. TRANSYT-7F Flow Profile Diagram

Output Verification. After using any traffic simulation program to evaluate conditions on an arterial segment, it is especially important to validate the output to insure that the results have been accurately generated. The timing parameters reported by the program, such as phase splits and sequences, should equal the existing parameters recorded in the field. Additionally, one should determine operational measures, such as v/c ratios and queue lengths, and compare them to the v/c ratios and queues predicted by TRANSYT. It is not always feasible to conduct a full-scale verification of the program output, but some degree of validation must always be conducted. A simplified approach for verifying results could be to evaluate the total travel times along each arterial in the network by comparing the field measurements to the program estimates.

If large discrepancies between observed and estimated values are noted, check the input data again. After making corrections, rerun the program and check the results. If inaccuracies persist, evaluate the system-wide default variables to assure that the assumed operational factors are reasonable. Inaccurate data cannot be used to simulate existing conditions accurately and will result in optimization runs generating inaccurate timing plans. The user should be sure that the simulation run is valid before going on to the optimization step.

4.4 Arterial Level of Service and Speed Profiles.

Arterial level of service, as defined by the *1985 Highway Capacity Manual* (12), is based on the average travel speed for the segment, section, or entire arterial under consideration. The average travel speed is computed from the running time on the arterial segments and the average intersection approach delay. The user can estimate these two variables either in the field or by computer programs such as the Highway Capacity Software, PASSER II, PASSER IV, or TRANSYT-7F. Arterial level of service analyses should be done for each major arterial street in the network.

Segment lengths and free flow speeds are computed from existing data, running times are computed using tabular values, and approach delays are estimated using HCM delay equations. Segment travel times are obtained by adding the approach delays and the running times. Figure 4-26 also shows the construction of a speed profile for the arterial. Figure 4-27 shows the necessary calculations to obtain arterial level of service. Delay estimates for the southbound through movement were obtained from the TRANSYT-7F output for NEMA movement 2 by using free flow speeds as input data. The analyst could use delay estimates from field measurements in this analysis.

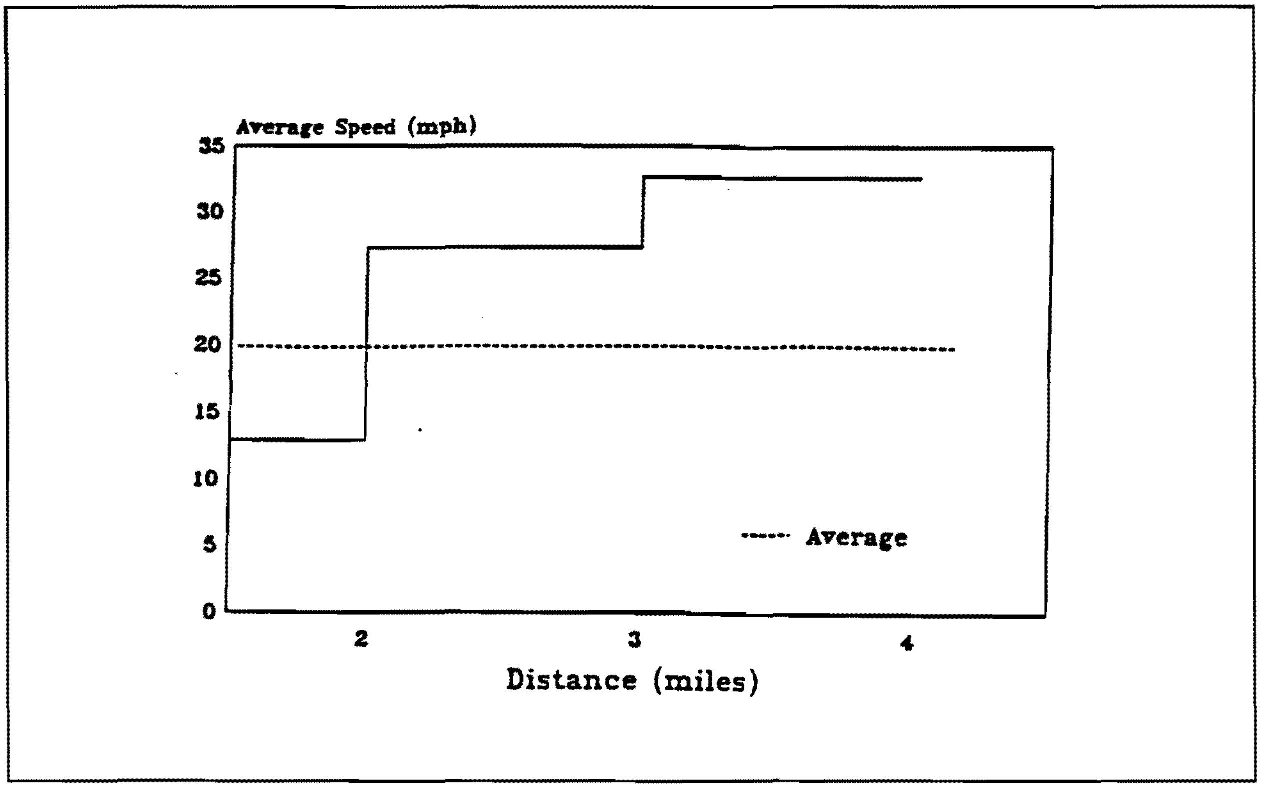


Figure 4-26. Speed Profile for the Arterial

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: Example West-bound

File or Case #: Date: 04-29-93

Prepared by: Sanqineni, Srinivas

$$\text{ART SPD} = \frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$$

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.359	II	35		36.9	31.0		67.9	0.359	19.0	C
2	0.502	II	30		60.2	20.6		80.8	0.502	22.4	C
3	0.42	II	38		39.8	36.2		76.0	0.42	19.9	C
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Grand Sum of Time = $\boxed{224.7}$

Grand Sum of Length = $\boxed{1.281}$ $\boxed{\text{C}}$

$$\frac{3{,}600 \times \boxed{1.281}}{\boxed{224.7}} = \boxed{20.5}\ \text{MPH}$$

[a] Use Table 11-4 and multiply by segment length

[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet

[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

Figure 4-27. Computation of Arterial Level of Service

5.0 OPTIMIZATION

After running a simulation to evaluate existing conditions and checking the output for accuracy, the next step toward retiming a freeway corridor network is to develop a signal timing plan that optimizes operations in the network. The objective of additional computer runs is to find cycle lengths, green splits, phase sequences, and offsets which will minimize delay, stops, and fuel consumption, increase capacity, and/or maximize bandwidth. The "best solution" depends on what the traffic signal analyst wants to accomplish. To quantify improvements, one should compare the best solution for the optimized runs and the evaluation run for the existing conditions.

Analysts can make optimization runs using PASSER III, PASSER IV, or TRANSYT-7F. Most of the data needed for the optimization runs has already been coded for the evaluation of existing conditions. The parameters that differ can be changed by editing the data. One edits the data base in PASSER III and PASSER IV by accessing the EDIT screen and in TRANSYT-7F by accessing the Data Input Manager.

The following sections discuss procedures and guidelines for optimizing a freeway corridor network signal timing using PASSER III, PASSER IV, or TRANSYT-7F presented by Wallace and Courage (25). As already mentioned, the user will have previously entered most of the data required for optimization, checked it for accuracy and calibrated for local conditions. The analyst will edit this data depending on the type of optimization to be performed.

Before proceeding, it should be stressed that the volume to capacity ratios and delays estimated by the computer models should prove consistent with existing conditions in the field (i.e., movements estimated to operate at oversaturated conditions should correspond to movements that fail to clear the entire queue during the green phase). If the model estimates are not consistent with existing conditions, the resultant signal timing design will prove less than optimum.

5.1 Signal Timing Design

Signal timing design ultimately depends on the quality of the traffic data and the parameters that represent the nature of traffic flow. The most significant of these inputs to traffic signal optimization models are clearly the traffic volumes and the saturation flow rates. These two classes of data prove so important to the process of traffic signal timing, they deserve special attention. The following are some general guidelines in selecting data for use in evaluation of existing conditions and developing a good signal timing design.

1. For evaluation, use actual **demand** volumes, as measured in the field.

2. For traffic signal timing purposes, use adjusted volumes, which consider lane distribution and peak hour factors. While adjusted volumes are proper for design, artificial vehicles suffer "artificial" stops, delay, and fuel consumption, which is inappropriate for evaluation.

3. Always use measured saturation flow rates if possible. If these must be estimated, use the *Highway Capacity Manual* (12) saturation flow adjustment method.

4. Use measured cruise speeds for evaluating (and calibrating) existing conditions, but use desired progression speeds for traffic signal design.

5.2 Optimization Strategies

PASSER III, PASSER IV, and TRANSYT-7F use their input data to decide how a signal system should operate. Their recommendations are expressed in terms of several control parameters, including phase sequence, cycle length, phase lengths and offsets. One should consider several important points about the design process. The following subsections give brief overviews of the considerations that go into determining signal design parameters.

Determining Phase Sequences. Selection of the best phase sequence (including how many phases are needed) hinges on the answers to (at least) the following questions:

1. What left-turn movements (if any) need to be protected?

2. Should left-turn protection (if required) be protected only or both permitted and protected?

The phasing used at a diamond interchange depends on the width of the interchange and the level and distribution of traffic volumes. PASSER III can analyze the phase sequence combinations, shown earlier in Figure 4-3. To optimize the phase sequence using PASSER III, the user accesses the Signal Phasing Data Screen and enters 'Y' beside the phase sequences to be considered. This data entry will also cause the internal offset for that sequence and the specified cycle length to be optimized. Normally, one uses the optimum cycle length during optimization of the phase sequence; i.e., cycle lengths and phase sequence/offsets are generally not optimized in a single run of the program. The sum of minimum green times should be less than the specified cycle length.

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

The optimum sequence of the interior left-turn phase depends on the origin of these interior movements, whether they come from the frontage road or the cross-street. For example, if the left side of the interchange has heavy left turns from the frontage road, a lead-lag phasing would be preferable to accommodate the heavy turns from the frontage roads. A lead-lead or lag-lag phase sequence would probably be used for heavy interior turns originating from the arterial.

While several factors are involved in the decision whether or not to use protected phasing (including local preference), TRANSYT-7F provides some guidance by showing the effect of protected turning intervals on delay, stops, fuel consumption and left-turn conflicts. The program's output also may be used directly to provide insight into all of the other questions. PASSER IV does not directly estimate the MOEs of the network. The program only estimates the progression parameters, like bandwidth, efficiency, and attainability.

PASSER IV evaluates network operations from the point of view of progression, and thus, one must take care not to overlook the potential impact of residual queues blocking the through platoons. When PASSER IV is allowed to consider both multiple phase sequences and overlap phasing, it generally recommends a mixture of phasing sequences. This mixture tends to give the best progression solution, and thus superior operation to say, the popular leading lefts. The point is, how willing are you to trade off improved operation for potentially confusing signal operation? Many locations concern themselves with motorist behavior being too unpredictable and are unwilling to allow phasing variations. At the other extreme, other locations will allow complete flexibility, some even to the extent of allowing the sequence at individual intersections to change during different times of the day. When coding a series of diamond interchanges in PASSER IV, the frontage roads are treated as one way streets, and each side of the interchange is treated as a separate node. The phasing sequence at each side of the interchange in most cases is fixed and the network optimized.

A consistent phasing sequence at all intersections, or allowing flexible phasing sequences at some locations, is a matter of local preference. It is almost always true, however, that varying the phase sequences among intersections will produce better progression along the arterials in the network.

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

The analyst should set the lower cycle length equal to the smallest permissible cycle based on the sum of the minimum conflicting greens as determined using Poisson or Webster's technique. Each side of the interchange may require a different cycle length. One should use the larger of the two cycle lengths as the lower limit. The upper cycle length will be constrained by the lower cycle length. Because a 15 to 20 second range is generally long enough to find a suitable cycle length, the upper cycle limit should not measure greater than 20 seconds longer than the lower cycle limit. PASSER III, however, will execute, even if the range exceeds 20 seconds. The maximum allowable value for the upper cycle length is 150 seconds for an optimization run. This report recommends an increment of 5 seconds, although other increments may be used.

PASSER IV bases cycle length selection on maximizing bandwidth efficiency, whereas TRANSYT-7F bases cycle length selection on minimizing some linear combination of stops and delay. The key issues about determining cycle lengths with PASSER IV are:

1. When evaluating a range of cycles, PASSER IV uses a less-than-full scale optimization process (i.e., optimization is based solely on maximizing bandwidth efficiency); thus, this report suggests that no decision be made on a single run with a wide range of cycle lengths.

2. One should carefully coordinate the cycle length with the phase sequences. A more flexible set of phase sequences may result in a different cycle length than a more constrained phasing plan.

Optimization of Offsets. The progression of movements on the interior approaches of the diamond interchange prove essential to minimizing vehicular delays. The internal offset may be evaluated and optimized by PASSER III based on the phase configuration, volumes, and cycle length. The internal offset is defined as the time from the beginning of the arterial or cross-street phase on the left side of the interchange (Phase A or Phase 2) to the end of the frontage road phase on the right side of the interchange (Phase B or Phase 8). The offset which produces minimum delay and adequate interior storage ratios is desirable. Based on the travel time, appropriate offsets can also be provided to frontage road phases (Phase B) for a series of diamond interchanges to obtain progression along the frontage roads. Such progression can minimize delay and improve mobility along the freeway corridor during extremely congested conditions, as well as in case of any unexpected diversions off the freeway. Traffic can also be diverted to other parallel arterials along the freeway. Thus, the analyst can effectively utilize any unused capacity along the corridor.

PASSER IV bases its design of offsets on maximizing bandwidth efficiency to provide progression on the major arterials without real regard to traffic performance. PASSER IV also maximizes progression on the major cross-streets depending on the weight given to the cross-street arterial compared to the major arterial(s). If the front of the through green is not in the progression band at the upstream intersection, if the left or right turn volumes from the upstream intersection exceed 10 percent of the total volume, or if the downstream volume exceeds the upstream volume by more than 10 percent, stopped queues may block the platoon. TRANSYT-7F can prove very useful in this determination, particularly by inspection of its Platoon Progression Diagrams.

PASSER IV will produce its best results on sections when the intersection spacing is such that most intersections are critical to the progression scheme and when the potential exists for significant progression bands. It also excels when most of the signals use multi-phase operation, provided that you are prepared to implement a mixture of phasing schemes on the same arterial. If an arterial street in the network system measures quite long and the through bands measure very narrow, consider splitting the system into two (or more) subsystems, where better operation may result within each of the shorter arterial subsystems.

5.3 Optimization Methodology

This section presents the development of an optimum freeway corridor network signal timing plan using PASSER III, PASSER IV, and TRANSYT-7F. The purpose of this section is to lead the traffic signal analyst through the steps they might normally use in a signal timing (or, more often) retiming project. The objectives of the study are as follows:

1. To simulate the existing timing plan, which would in reality be used to calibrate the models and to provide a basis for evaluating the new plans to be developed.

2. To develop optimal timing plans for pretimed control. Phasing changes will be permitted only if strongly recommended by the process, since they would require wiring changes.

All control periods (a.m., p.m., and off-peak) require the same degree of attention in reality, but to avoid redundancy, discussions of **the process** for multiple periods will not be repeated. When it is significant to mention the other time periods, appropriate mention will be made. In an actual study, the analysis and evaluation of differences, particularly in phasing, must be repeated for each control period.

Evaluating the Existing Conditions. One way to check the data is to print the Data Report, developed in the next step--evaluating the existing conditions. There are two major purposes of this step:

1. To check and verify the correctness of data; and
2. To calibrate the model parameters.

This step is perhaps the most important step in the design process. The final designs can only be as "good" as the data and parameters that control the models -- saturation flow rates, lost times, platoon dispersion factors, etc. Here, only the mechanics of the process will be reviewed.

The present version of PASSER IV does not predict any MOEs, like v/c ratios, stops, delays, and fuel consumption. One can make the most thorough evaluation of an existing timing plan using TRANSYT-7F to simulate existing conditions. To run this model in the simulation mode, select **Edit ... Run Instructions** to set up the run. At a minimum, the following should be done:

1. The initial run number ('1') is okay for this run. Enter a "Run Title," such as "T7F on Existing" for TRANSYT-7F simulation/evaluation of existing conditions.

2. Enter the actual cycle length in both the "Min" and "Max Cycle." The "Incr(ement)" may be ignored for a simulation run.

3. Most important at this point, use "unadjusted" volumes, so that a comparison with the known data can be made.

To help double check all coded data, the TRANSYT-7F Input Data Reports should be printed and reviewed carefully against the raw data sources. After checking the data, which should appear correct at least from visual observation, the process of model calibration should begin. This step involves examining the TRANSYT-7F <u>results</u> to insure that degrees of saturation (volume to capacity ratios), delays, queue back ups and platoon progression all conform to the actual operation; i.e., TRANSYT-7F's output should be reasonable and match observations made in the field.

One should note that traffic signal analysts should never assume they have finished reviewing the base data at the conclusion of this step. Often corrections need to be made to the data, even after making an optimization run. Thus, there may arise a need to make final evaluation runs for both Before and After conditions as a last step in the design process after the optimization.

After completing the evaluation of existing conditions, the next step is to optimize the freeway corridor network signal timings. PASSER III, PASSER IV, and TRANSYT-7F are the programs most useful for optimizing freeway corridor networks. The optimization process using these programs is briefly described below.

Optimization. As mentioned earlier, one can use PASSER III, PASSER IV, and TRANSYT-7F to optimize signal timings. Each of these computer programs has advantages and disadvantages.

PASSER III is the best program to evaluate and optimize a series of diamond interchanges. The program analyzes diamond interchanges in great detail and provides progression along frontage roads. PASSER III cannot, however, evaluate/optimize progression along cross-streets or parallel arterial streets.

PASSER IV can evaluate and optimize a large network. Frontage roads and diamond interchanges can be coded in as two one-way streets and as two nodes respectively. PASSER IV can select any phasing sequence and provide progression along any link to the degree the user desires. PASSER IV, however, tends to optimize a network in order to provide progression along arterials. Minimizing system-wide stops and delay is given a lower priority. Also, the present version of PASSER IV does not calculate any MOEs. A simulation program, such as TRAF-NETSIM, is to be used to evaluate the signal timings developed by PASSER IV.

Like PASSER IV, TRANSYT-7F can evaluate and optimize a large network. Unlike PASSER IV, TRANSYT-7F tends to optimize a network in order to minimize system-wide performance measures, like stops and delay. The program does not provide a good progression solution. Also, TRANSYT-7F cannot select a phasing sequence for the intersections. The user has to specify the phasing sequence for the intersections in the network. Hence, one can select the phasing sequences from the solution obtained by PASSER IV and code into TRANSYT-7F.

One can analyze a freeway corridor in three separate ways. In the first case, PASSER III analyzes the series of diamond interchanges and develops the optimal signal timings. The timings for the diamond interchange are fixed. The optimum cycle length, offsets, splits, and phasing sequence obtained from PASSER III are then input into PASSER IV, and the optimum signal timing plans for the entire freeway corridor network are obtained. In this case, the optimum cycle length obtained for diamond interchanges will equal the governing cycle length for the entire freeway corridor network.

In the second case, the signal timings obtained from PASSER III in the first case are input into TRANSYT-7F and the network is analyzed without considering bandwidth constraints. One obtains the phasing sequence for the intersections from PASSER IV. Note that the timings for the diamond interchanges remain fixed. TRANSYT-7F will select its own timing plans for the other intersections in the freeway corridor network. The cycle length selected is the same as in the first case and may not be the best cycle length for the entire freeway corridor network.

In the third case, one uses only PASSER IV to optimize the entire freeway corridor network. The user enters data into PASSER IV by treating each diamond interchange as two nodes and the frontage roads as two one-way streets. The best cycle length selected by PASSER IV will have maximum overall network bandwidth efficiency. The data can be analyzed such that arterials close to the freeway corridor will have more priority for progression. PASSER IV optimizes offsets, splits, and phasing sequences and selects the best cycle length for the entire freeway corridor network system.

Since, the present version of PASSER IV does not predict MOEs, the three cases can be compared using a common simulation program, such as TRAF-NETSIM. If the cycle length selected by PASSER IV differs from the previous cycle length, then a cycle length close to these two cycle lengths may be used, depending on the field conditions. This process enables the traffic signal analyst to select the best suitable cycle length for the entire freeway corridor network system. After selecting the suitable cycle length, the above process should be repeated using PASSER III, PASSER IV, and TRANSYT-7F.

Visual Inspection of the Timing Plans. The above procedure has illustrated the basic mechanics of using PASSER III, PASSER IV, and TRANSYT-7F to optimize a freeway corridor network. Before implementing the timing plans, the output should be studied to get an idea of how good the timings are. One can perform this examination in the following ways:

1. Print the outputs to get visual evidence of likely operations:

    a. Time-space diagrams (TSDs) to study utilization of leading and/or lagging phases, particularly lead-lag subsequences;

    b. Flow profile diagrams (FPDs) to get a better view of probable traffic performance at the intersections; and, most useful,

    c. Platoon progression diagrams (PPDs) to evaluate overall progression and the effect of queues.

2. Compare the timings generated by the various computer programs (PASSER IV and TRANSYT-7F) to determine if the timings produced by one program prove superior than the other.

3. Whatever the final decision, fine-tune the plan, both in the office and in the field.

The last point is the most important-- the traffic signal analyst should make the final decision on which is the best timing plan to implement, not the computer programs.

Fine-Tuning the Splits and Offsets Using TRANSYT-7F. After selecting a cycle length, the timings should be fine-tuned to minimize delays, stops, and improve progression. Fine-tuning is done by reducing the step size and selecting one cycle length and results in further reducing the delays and stops. A suggested range for closer scrutiny is as follows:

1. The minimum cycle length should measure no less than 0.85 times the maximin cycle length; and the maximum cycle length should measure no higher than 1.25 times the maximin cycle length.

2. These minimum and maximum values may need adjustment to the nearest five seconds to accommodate the cycle length resolution offered by some systems.

3. Although one may use this range as a general guide, one should really let the quality of progression and the resulting MOEs dictate which cycle length to use.

If at any step in the process, the "best" cycle length recommended falls on the minimum or maximum values, the analysis should be repeated while expanding the range to encompass the threshold value, unless the threshold value equals the absolute minimum.

Final Design. After obtaining fine-tuned signal timings, increase the stop penalty in Card 1 and run TRANSYT-7F for the fine-tuned timings in order to further fine-tune the splits and offsets and to improve the progression by reducing stops.

This step is an appropriate point in the design process to evaluate the suitability of a bandwidth solution. The measure of interest here is "attainability," previously defined as the bandwidth divided by the shortest green time in the system. The rationale behind this measure is that it is not possible to obtain a through band which exceeds the critical green time. Theoretically, an average attainability of approximately 50 percent can be achieved by arbitrarily assigning all progression to one of the two directions (100 percent in one direction and 0 percent in the other direction). So, a bi-directional attainability less than 50 percent indicates that the compromise required to achieve an equitable bandwidth distribution of progression opportunities resulted in a generally poor design. Such a result often indicates that a solution based on stops and delay optimization by TRANSYT-7F would be more appropriate.

An alternative solution for a freeway corridor network where an acceptable progression solution is not attainable would entail partitioning the network into two or more subsystems, with each free to operate on its own cycle length. The rationale behind this alternative is that the improvements resulting from suboptimization may prove great enough to offset the problem of lost progression between them. The analyst should optimize each of the two sections, of course, for phasing as well as timing and offsets.

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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 determining a "best solution" either from PASSER III, PASSER IV, or TRANSYT-7F, the results should be transferred to a controller worksheet for use in the field. Not all entries to the controller sheet are addressed in this text, only the entries directly related to the computer output are discussed. Different controller data sheets exist for electromechanical pretimed and actuated control; worksheets may vary with the brand of controller, or one may use a self-made worksheet. These guidelines address the current TxDOT controller standard specifications as much as possible.

The timing design from PASSER III, PASSER IV, and TRANSYT-7F is generally **not** the design "installed in the field." It is impossible for computer models to consider everything in the real world. Thus, the fourth component, "the traffic engineer" comes in.

Regardless of the type of control -- pretimed or actuated -- the first step in timing implementation should be to fine-tune the model's design. The analyst should examine the timing plan closely to adjust offsets and splits. A few seconds one way or the other may change the throughput of an intersection by hundreds of vehicles per hour.

Once satisfied with the design "on paper," the next step is implementation of the timing plan in the field, a function of the type of control.

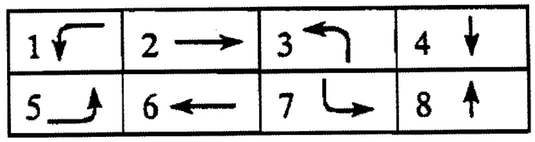
6.1 Terminology

Before discussing the implementation of the timing plans, one must define some terms commonly used in traffic signal timing (26):

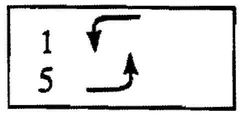
Split - The portion of the cycle length allocated to each of the various phases, expressed in seconds or as a percent of the entire cycle length.

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

Phase - Individual movement as seen by the controller. 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:

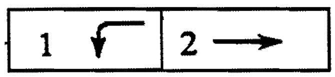


Figure 6-1 illustrates phases for a diamond interchange, their controller settings, and the relationship between the diamond controller and PASSER III phases. The figure illustrates four types of phases. One can define the internal offset as illustrated in Figure 6-1 as the difference in time from the start of the arterial phase (A) on the left side of the interchange to the end of the frontage road phase (B) on the right side of the interchange.

6.2 Output Interpretation

The starting point is the format of the typical signal timing outputs. Each program produces a typical table summarizing the signal timing plan and contains the following information:

1. Cycle length;
2. Movements serviced on each phase;
3. Time allotted to each phase; and
4. Offset with respect to a specified phase or interval.

Different programs use different methods to describe timing plans. Figure 6-2, 6-3, and 6-4 illustrate the typical signal timing design plans presented by PASSER III, PASSER IV, and TRANSYT-7F respectively. A comparison of the timing design plans indicates some interesting similarities and differences.

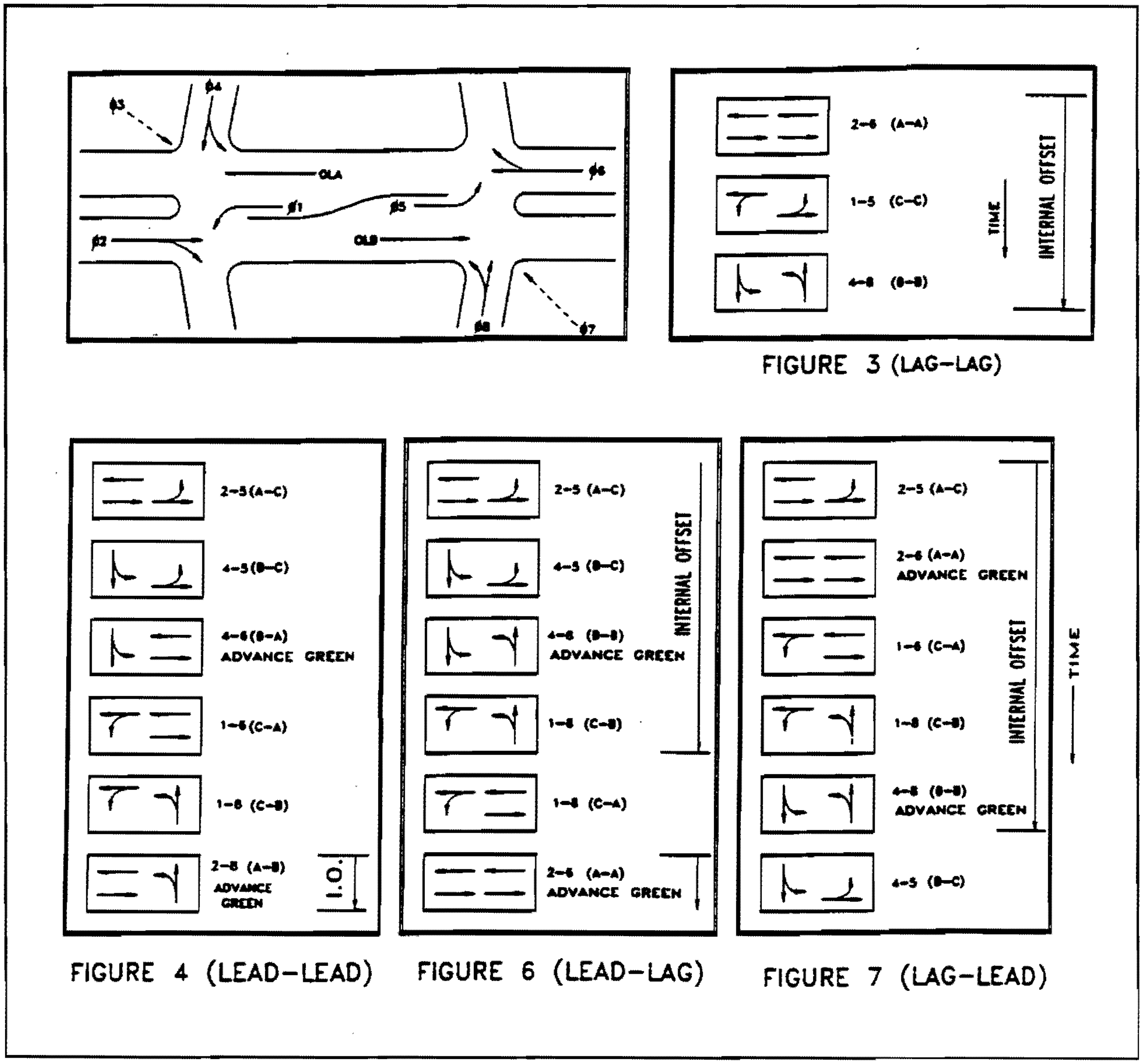


Figure 6-1. Relationship between Diamond Controller and PASSER III Phases

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<SP101>
* * * INTERCHANGE  1 bingle                                          RUN 01 PAGE  68

                     ***   SIGNAL PHASING INFORMATION   ***

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

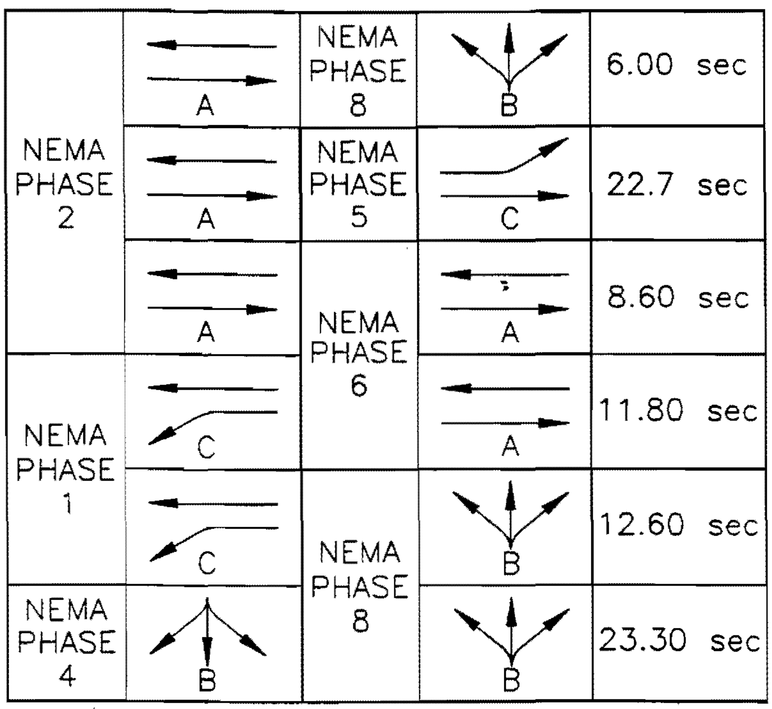


Figure 6-2. PASSER III Timing Design Table

**** SUMMARY OF PASSER-IV BEST SIGNAL TIMING SOLUTION ****

*-[MASTER INTERSECTION] = NODE NO. 25 CYCLE LENGTH = 100.0 SEC

NODE NO. 25

		MADRONA (W-E)			CARSON (N-S)		
		ARTERY 1 -- SIGNAL 1			ARTERY 4 -- SIGNAL 3		
PHASE NUMBER			1				2
NEMA MOVEMENTS		-	2+6	-	-	-	4+8
PHASE (SEC)		-	69.2	-	-	-	30.8
PHASE (%)		-	69.2	-	-	-	30.8
PIN SET (%)	100 / 0.0		.0	-	-	-	69.2

OFFSET POINT = .0 SEC (.0%)
SYSTEM REFERENCE: START OF PHASE NO. 1 OF THE MASTER SIGNAL.

NODE NO. 27

		MADRONA (W-E)			TORRENCE (N-S)		
		ARTERY 1 -- SIGNAL 2			ARTERY 3 -- SIGNAL 3		
PHASE NUMBER			1		2		3
NEMA MOVEMENTS		-	2+6	-	3+7	-	4+8
PHASE (SEC)		-	60.0	-	10.0	-	30.0
PHASE (%)		-	60.0	-	10.0	-	30.0
PIN SET (%)	100 / 0.0		.0	-	60.0	-	70.0

OFFSET POINT = 56.7 SEC (56.7%)
SYSTEM REFERENCE: START OF PHASE NO. 1 OF THE MASTER SIGNAL.

Figure 6-3. PASSER IV Timing Design Table

INTERSECTION 1 PRETIMED - SPLITS ARE FIXED										
INTERVAL NUMBER :	1	2	3	4	5	6	7	8	9	10
INTVL LENGTH(SEC):	8M	4	13	1	15	4	15	4	20	4
INTVL LENGTH (%):	9	5	15	1	17	5	17	5	21	5
PIN SETTINGS (%):	100/0	9	14	29	30	47	52	69	74	95
PHASE START (NO.):	1		2		3		4		5	
INTERVAL TYPE :	V	Y	V	Y	V	Y	V	Y	V	Y
SPLITS (SEC):	12		14		19		19		24	
SPLITS (%):	14		16		22		22		26	
LINKS MOVING :	103		101		101		105		107	
	104		103		102		106		108	
									112	

Figure 6-4. TRANSYT-7F Controller Setting Table

PASSER III Signal Timing Tables. The phase lengths reported by PASSER III include the green plus yellow plus any red clearance time for that phase. PASSER III reports the phase lengths in two places: 1) General Signalization Information, and 2) Signal Phasing Information. Figure 6-2 shows the General Signalization Information output with the Signal Phasing Information below it.

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

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

PASSER IV Signal Timing Tables. The PASSER IV format attempts to represent dual-ring operation, even though the program itself models a single-ring sequence. This representation is achieved by assigning a phase number to each of the movements in the NEMA compatible scheme. The assignment is based on PASSER IV's internal definition and may not reflect actual assignments in the field.

Figure 6-3 presents PASSER IV's recommended signal timing plan design in two columns. The left column shows the phase splits information for the arterial, while the right column shows the phase splits information for the cross-street. The PASSER IV signal design plan indicates the NEMA movements involved, their duration in seconds, and percent of the cycle of each phase. The order of the phases presented in these two columns indicates the order in which the phases will operate in the field. The phase duration as well as the "cycle count," which indicates the cumulative cycle time prior to the beginning of each phase, are also indicated. The offset point with respect to the system reference point is shown in seconds as well as in percentage of the cycle.

TRANSYT-7F Signal Timing Tables. The program presents a table with one column for each interval in the single-ring sequence. For each interval, the program provides the interval length and the pin settings. The interval which begins each phase is indicated along with the type of interval in each phase. The interval types are as follows:

F	Fixed	The duration of the phase is not subject to change by TRANSYT-7F during the optimization process.
V	Variable	The optimizer can modify the duration of this interval. Only one variable interval is allowed per phase.
Y	Yellow	This interval represents the yellow portion of the change period.
R	Red	This interval represents the red portion of the change period.

For each phase, the program shows the splits after the interval types. The link list is presented next. TRANSYT-7F uses a link node concept to represent a network of traffic signals. A node represents each intersection, and a link generally represents each movement. TRANSYT-7F User's manual (22) suggests two such link numbering schemes. The first two digits of the four digit link number represent the intersection number. The last two digits represent the movement number. Figure 6-5 illustrates these two schemes.

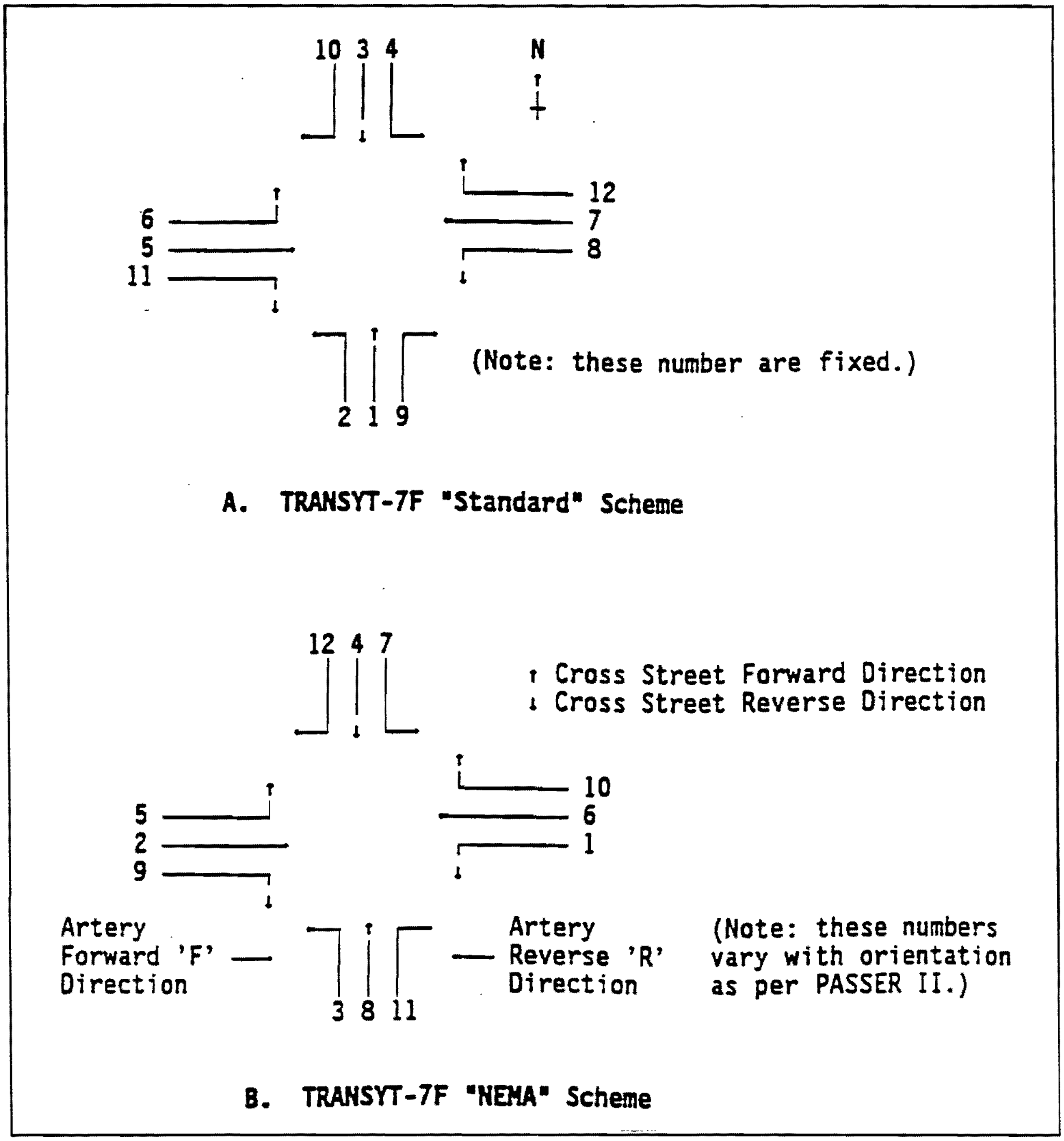


Figure 6-5. TRANSYT-7F Movement Numbering Schemes

6.3 Typical Traffic Control System Timing Inputs

Each traffic control system has its own unique requirements for timing plan data. Thus, it is essential that the analyst thoroughly understands the specific equipment operating the control system before making any timing design changes in the field. The following represent some categories of the type of control equipment.

1. Pretimed controllers which may operate either isolated or coordinated with other pretimed controllers in the same system.

2. Large scale computerized systems such as the, Urban Traffic Control System (UTCS), which takes direct control of the field equipment.

3. Traffic responsive systems in which standard NEMA dual-ring traffic actuated controllers are supervised by external hardware to impose some aspects of a coordinated operation. Analysts generally refer to this category as "externally coordinated systems."

4. Traffic responsive systems in which the developers have gone a step beyond the NEMA standards and incorporated the coordination hardware and software internally within the controller. The timing plan resides in a system master controller and not in the local traffic actuated controllers themselves. Analysts generally refer to this category as "internally coordinated systems."

6.4 Implementation in Pretimed Controllers

When implementing timing plans for pretimed control, both the splits and pin settings can be determined from the PASSER IV output. The splits, also referred to as *GREEN TIME*, and the *PIN.SET* for concurrent and individual phases are given in the output table titled "Summary of PASSER IV Best Signal Timing Solution," as indicated in Figure 6-3. One can read the pin settings directly under the phase times in percentages in Figure 6-3. The *PIN.SET* equals zero for the first phase; for every subsequent phase the *PIN.SET* equals the *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 PASSER IV output include the yellow and the red clearance for each phase.

The information about each artery in the network is illustrated in the PASSER IV output titled Artery-Wide Information, shown in Figure 6-6. Figure 6-6 illustrates the bandwidth for both directions and the phasing pattern for all the intersections along each arterial. The phase settings in fractions of a cycle and in seconds for the entire arterial at all the intersections and the progression times and speeds along each link are also illustrated. Figure 6-7 illustrates a typical time-space diagram for the arterial.

*** ARTERY 2 ***

NAME OF ARTERY: HAWTHORNE NUMBER OF SIGNALS: 5
A - DIRECTION : EASTBOUND

*** ARTERY WIDE INFORMATION ***

ARTERY DIRECTION	BAND (% of Cycle)	BAND (Seconds)
EASTBOUND	.3409	34.09
WESTBOUND	.3409	34.09

EFFICIENCY(%): 34.09 ATTAINABILITY(%): 84.05

*** INTERSECTION INFORMATION ***

NODE NO.	CROSS STREET NAME	LEFT TURN PATTERN SELECTED	SIGNAL NO.
6		LEAD- LAG	1
7	CARSON	LEAD-LEAD	2
8		LAG -LEAD	3
9		N/A	4
10	TORRENCE	LEAD- LAG	5

ALL PHASE STARTING TIMES ARE RELATIVE TO THE START OF GREEN IN THE EASTBOUND DIRECTION AT SIGNAL 1 IN ARTERY 1.

** PHASE SETTINGS -- FRACTIONS OF CYCLE **

		GREEN			LEFT			BAND			
NODE NO.	DIR	BEGIN	END	LENGTH	BEGIN	END	LENGTH	BEGIN	END	WIDTH	QUEUE TIME
6	EB	.553	.956	.403	.553	.725	.171	.587	.928	.341	.000
	WB	.725	.133	.408	.956	.133	.177	.792	.133	.341	.000
7	EB	.611	.108	.497	.511	.611	.100	.758	.099	.341	.000
	WB	.611	.108	.497	.511	.611	.100	.611	.951	.341	.000
8	EB	.584	.234	.650	.073	.234	.160	.893	.234	.341	.000
	WB	.484	.073	.590	.484	.584	.100	.484	.825	.341	.000
9	EB	.984	.734	.750			.000	.984	.325	.341	.000
	WB	.984	.734	.750			.000	.393	.734	.341	.000
10	EB	.052	.498	.446	.052	.183	.131	.112	.453	.341	.000
	WB	.183	.598	.414	.498	.598	.100	.257	.598	.341	.000

Figure 6-6. PASSER IV Artery Wide Information

```
                    *** PASSER-IV TIME-SPACE DIAGRAM ***

     NAME OF ARTERY: HAWTHORNE                  TIME  SCALE =   3 SEC/CHAR
     CYCLE LENGTH  : 100.00 SECONDS             DIST. SCALE =  150 FT/LINE

                1         2         3         4         5         6
NODE   1234567890123456789012345678901234567890123456789012345678901234  DIST
       ________________________________________________________________  (FT)
     6 !WWWW==============EEEEEE        WWWWW================EEEEE           0
       !                    .   .      .!  .
       !                                !
       !                     ..         !..
       !                                !
       !                     . .        . .
       !                                !
     7 !    ================                 ==================           1120
       !                                !
       !                 .        .  .  !     .
       !                                !
       !                ..          ..  !      .
       !                                !
     8 !  EEEEEE=========WWW                EEEEE=========WWWW            1950
       !                                !
       !             .          .      .!         .
     9 !                        =========                         ======  2510
       !           .           .        !.          .
       !                                !
       !          .          .          ! .           .
       !                                !
       !        .          .            !   .          .
    10 !==EEEE           WWW================EEEE          WWW===========  3350
       !________________________________________________________________
NODE   1234567890123456789012345678901234567890123456789012345678901234  DIST

NODE    6 ---> NODE   10:  EASTBOUND: DIRECTION-DOWNWARDS : GREEN -  EEEEEE
NODE   10 ---> NODE    6:  WESTBOUND: DIRECTION-UPWARDS   : GREEN -  WWWWWW
 EASTBOUND BAND =  34.1 SECS AT 43.5 MPH             GREEN IN BOTH DIRECTIONS
 WESTBOUND BAND =  34.1 SECS AT 42.0 MPH      ====== RED   IN BOTH DIRECTIONS
```

Figure 6-7. PASSER IV Time-Space Diagram

6.5. Implementation in Externally Coordinated Systems

Traffic actuated controllers do not recognize the concept of cycle length, phase splits, or offsets used by signal timing design programs. Without coordination, these programs would constantly adjust to fluctuations in demand and generate their own timing plans on each cycle. Thus, analysts need some means of imposing a constant cycle length with predictable time relationships if a group of traffic actuated controllers is to function as a coordinated system. Two external control functions, which are commonly used to impose a background cycle and offset on an isolated traffic controller are:

Hold Function - Hold function causes the controller to hold at one point in the cycle until it is released, regardless of the detector demand.

Force-off Function - This function causes the controller to terminate a phase and move on to the next phase, again regardless of detector demand.

Elements of Coordination. The external commands discussed earlier modify the operation of a traffic actuated controller to impose a specific cycle length, splits and offsets. In a freeway corridor network, progression is normally provided by coordinating the through movements. This is because the proportion of the through traffic is far more than the turning traffic. Thus, frontage road traffic, phases 2 and 6 on the major streets, and phases 4 and 8 on the minor streets (NEMA convention) of arterials are coordinated. One can find a detailed discussion of the control concepts in the *Traffic Control Systems Handbook* (3). The typical elements in coordination are:

Non-Actuated Phase - It is generally the "main-street" or arterial phase, displayed each cycle without the need for detector demand.

Subsequent Phase - It is referred to as "non-actuated+1," or "non-actuated+2," etc. A four-phase operation would generally consist of the non-actuated phase plus three actuated phases.

Yield Point - The point in the cycle at which the hold function is released is called the yield point. If the non-actuated phase has pedestrian provisions, the yield point would normally occur at the beginning of the Flashing Don't Walk. If no pedestrian provisions exist, it would occur at the end of the green. In a typical coordinator, all times are referenced to the yield point.

Offset -	Offset is the elapsed time from an arbitrary system zero reference point to the beginning of the non-actuated phase. This point is the value normally reported by PASSER IV and TRANSYT-7F, although for an "actuated" controller, TRANSYT-7F refers to this time as a yield point.

Figure 6-8 illustrates the calculation of the yield points, force-off points, and the offsets to the yield points on the left side (O_l) and the right side (O_r) of a diamond interchange having a cycle length of 60 seconds.

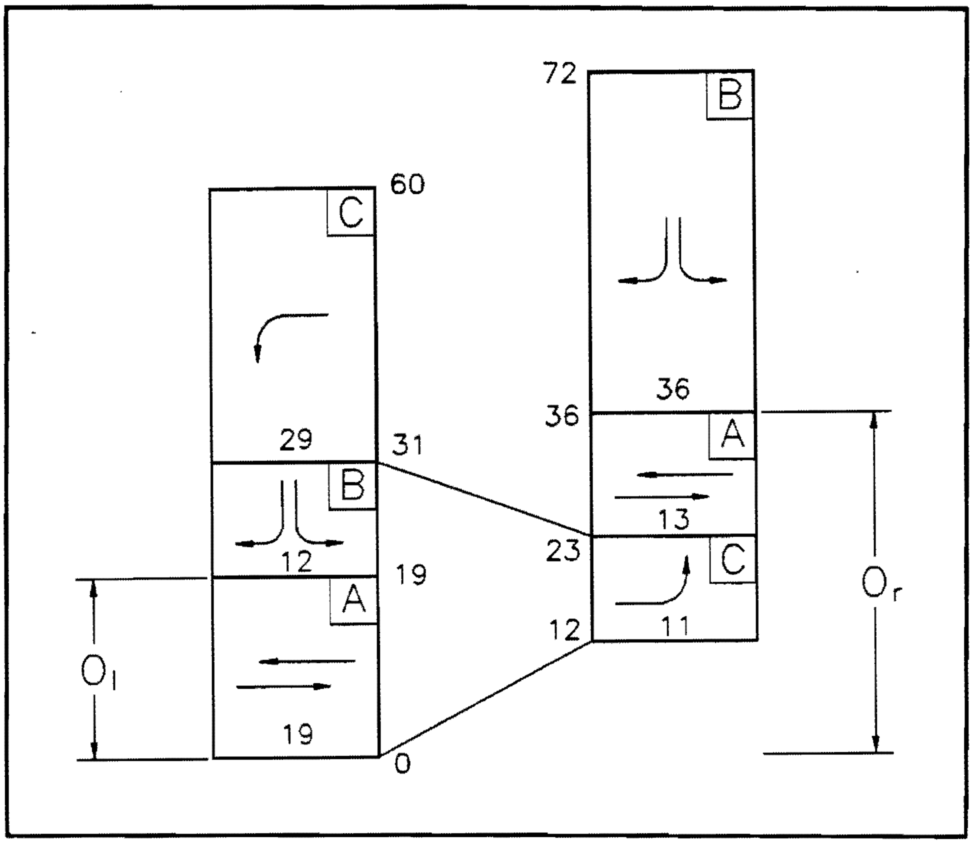
Figure 6-9 illustrates the computation of the yield point for a typical timing plan for arterials. A yield point can be calculated as:

Yield point	=	OFFSET	+	SPLIT	-	(FDW	+	CLEARANCE)
	=	27	+	35	-	(15	+	5)
	=	42						

The force-off point for any of the actuated phases may be calculated as:

Force-off	=	(phase) Begin Time	+	Splits	-	Clearance
	=	(non-act + 1) 20	+	20	-	5
	=	35				

(non-act + 1) 20 + 25 - 5 = 35
(non-act + 2) 40 + 25 - 5 = 60
(non-act + 3) 65 + 20 - 5 = 80



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

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

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

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

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

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

Figure 6-8. PASSER III Interchange Yield Point Calculation

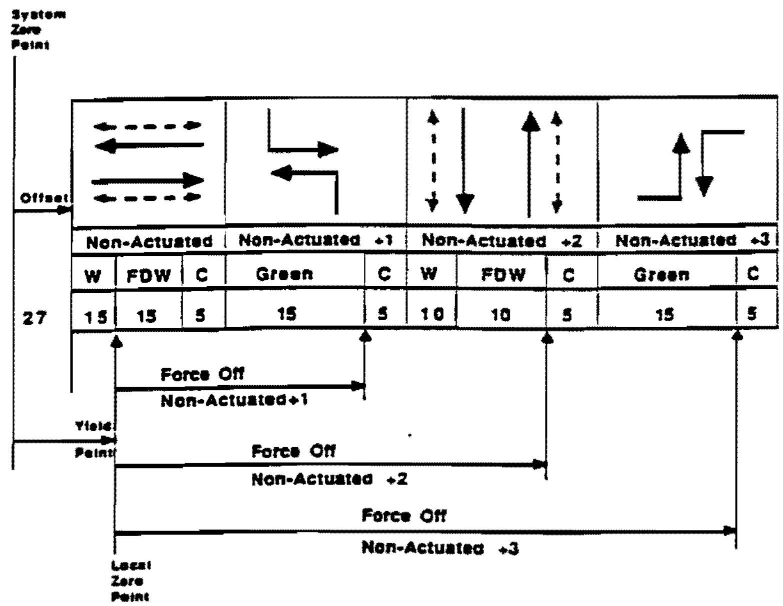


Figure 6-9. Sample Calculation of Force-offs and Yield Point

Permissive Periods. If the operation as described earlier was implemented as-is, there would be just one instant (yield point) when the actuated phase (cross-street) can be initiated if a vehicle places a call on the detector. During peak periods, a constant demand of traffic on the cross-street exists; however, during off-peak periods, if a cross-street vehicle arrives just after the yield point, it will have to wait the whole cycle to be serviced. Thus, the concept of permissive periods gains importance. Permissive periods service the cross-street traffic even after the onset of the yield point as long as the subsequent phases, remain unaffected. This action will reduce the delay to the cross-street without compromising arterial coordination. Figure 6-10 illustrates the calculation of the yield point, force-offs, and permissive periods for intersections.

Note that the term "permissive period" does not relate in any way to the left-turn treatment. The use of the word "permissive" or "permitted" in connection with left-turn protection is a totally different subject.

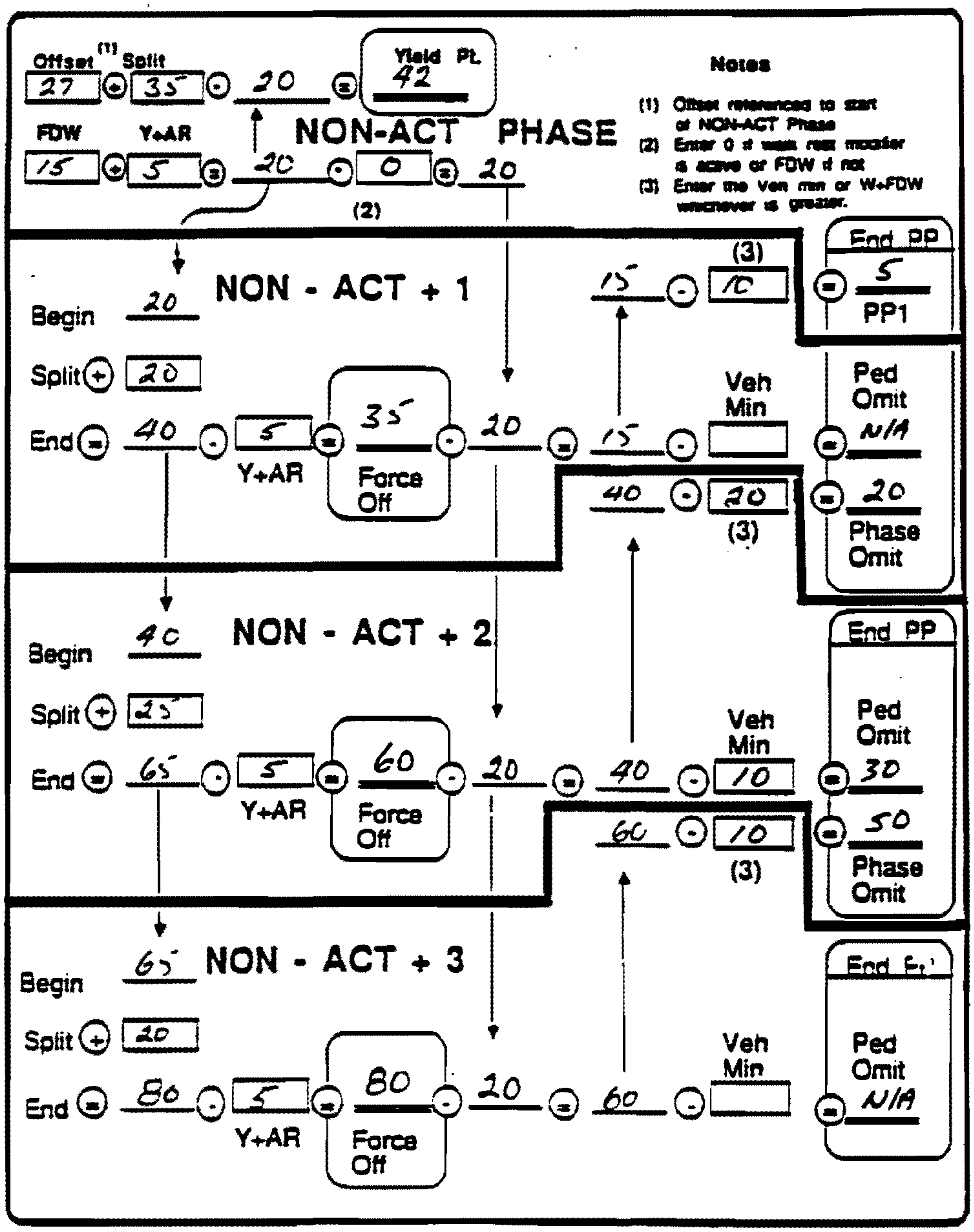


Figure 6-10. Sample Calculation of Yield Point, Force-Offs, and Permissive Periods for Intersections

6.6 Implementation in Internally Coordinated Systems

Due to the complications involved with external coordination, more and more traffic control systems have incorporated internal coordination features. The following discussion contains timing plan implementation guidelines suggested by Wallace and Courage (25). Internal coordination systems accept cycle length, splits, and offsets and perform computations. Computerized timing designs are, therefore, much easier to implement with internally coordinated systems. Some differences exist in the way offsets and reference points are referenced when using software control as opposed to using hardware control. When using software control, the offset is usually referenced to the start of the main-street green. When using hardware control, the offset is usually referenced in three ways:

1. To simultaneous display of coordinated phases;
2. To the first coordinated phase; and
3. To the barrier following coordinated phases.

While NEMA intersection controllers are well standardized, the coordination software and hardware vary considerably. Thus, the data transfer routines must be carefully tailored to each system.

6.7 Multiple Period Considerations

For a given control period, the development of a timing plan involves more than simply running the computer programs once. The steps presented in Section 5.0 should be followed to develop a recommended timing plan for each control period. Then the analyst must resolve the differences, as explained below.

Selection of a Timing Plan for a One-Dial System. If the signal system is capable of handling only one timing plan, one may follow the procedure discussed below to determine an acceptable timing plan; however, no one signal timing plan will prove optimum for all periods of the day; thus, the selection of the "best" signal timing plan necessarily requires engineering judgment.

To accomplish the objective of having good signal timing plans throughout the day, it is recommended the analyst develop more than one timing plan. One should evaluate each timing plan using the different traffic demand levels which exist during various periods of the day. Signal timing plans should be developed for the a.m. peak period traffic demands, the p.m. peak period traffic demands, and representative off-peak period traffic demands. Each timing plan should be evaluated using the traffic demand data for the AM peak, PM peak and off-peak periods. One should select the most appropriate signal timing plan based on engineering judgment, attempting to accommodate the majority of the traffic demand as well as possible.

One should also examine each plan for major problems, such as oversaturation or queue spillback. In short, plan selection should be a thoughtful and objective process, not a purely quantitative assessment.

Selection of Timing Plans for Multiple Dials. If a system can deal in multiple control periods, the analyst is less constrained as to the control parameters; what is likely to be constrained is the phase sequences. An analysis similar to the example above should be performed but the "best" phase sequences from each control period should be used to evaluate the others traffic conditions. As above, the best overall phase sequences should be picked, and those phase sequences should be applied all day.

6.8 Fine-Tuning the Timing Plan

The following discussion follows suggestions and guidelines presented by Yauch and Gray (27). The final step in the implementation phase of retiming signals is fine-tuning the signal timing plan. Fine-tuning involves observing signal timing plan in operation after its installation in the controller and determining if the new plan is operating effectively. Based on observations, one may need to make minor adjustments to improve the performance of the timing plan in the real world setting. Most adjustments will be made to the phase lengths or offsets.

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

Fine-Tuning In-house. Before actual field observation, the analyst 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 these steps are taken before field implementation, adjustments in the field will be minor.

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

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

Cycle Length and Phase Splits. After the controller settings have been modified, field observations should be made to ensure that the proper settings have been implemented. Cycle lengths and phase lengths can be verified in the field with a stopwatch. For most full actuated controllers, the maximum green for each phase may be observed by locking 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 needed or not, and most benefits from actuated control will be lost.

Implementation of new timing plans, especially those involving new phase sequences, may make drivers hesitate or may refer back to the old phase sequence out of habit. This should be taken 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 III, PASSER IV, or TRANSYT-7F, one should check the input data and controller settings again.

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

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7.0 PROJECT DOCUMENTATION

Assessing the benefits of the new signal timing plan is an important final step in the signal retiming process. 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 expenditures are allocated. Verifying estimated improvements (or better operations along the arterial) assists the analysts in future fund allocations for projects. Second, the analysts are interested in documenting any decisions pertinent to the signal timing process for future reference.

7.1 Estimation of Benefits

To evaluate the results 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 v/c ratios as a basis of comparison. The objectives or goals of the project should first be established before undertaking it. Some objectives may include:

1. Improved safety along the arterial network;
2. Reduced system delay on the arterial network;
3. Improved air quality;
4. Reduced fuel consumption; and
5. Increased arterial operational efficiency.

From these goals, the analyst chooses measures of effectiveness for use in the Before and After analysis. One can use either TRANSYT-7F or TRAF-NETSIM to estimate chosen measures for both the Before (existing) and After (optimized) conditions. The same program, however, should be used for MOE estimation for both conditions; i.e., one should not use TRANSYT-7F in the Before condition and TRAF-NETSIM in the After condition. The differences in the Before and After conditions represent the benefits of the new signal timing plan. Because both arterial street network program's analysis periods equal one hour, the analyst should multiply the benefits by unit costs and then convert them to daily and annual totals for the life of the project.

It is important to remember that, when estimating benefits, one should use actual traffic volumes rather than the adjusted traffic volumes to determine optimum signal timings during the peak hour; i.e., benefits should only be attributed to 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 one can use TRANSYT-7F and TRAF-NETSIM to estimate benefits for both pretimed and actuated control. In both cases, the benefits attributable to signal retiming represent the difference between the Before and After conditions. Actuated control, however, will result in better operation than that predicted by TRANSYT-7F, as long as the volume to capacity ratios at the intersection remain less than 0.95. The improvement due to actuated control equals an approximate 15 percent reduction in individual MOEs, when volume to capacity ratios equal less than 0.85 and a correspondingly lesser reduction as volume to capacity ratios approach 1.0.

Typically, benefits for retiming signals range from reductions in delay, stops, and fuel consumption of 5 to 20 percent, depending on the type of retiming strategy used (28). Generally, optimization of green splits or cycle length optimization will produce improvements of around 5 percent, while geometric and signal hardware improvements may show as much as a 20 percent overall improvement. The percentage of improvement also depends on how bad the signal timing plan was before it was optimized.

Some cities have published information regarding the benefits of signal retiming to motorists. This information allows local citizens and public officials to recognize the benefits gained through traffic signal retiming projects. A previous study conducted on signal retiming in 44 Texas cities (2,243 signals retimed) resulted in annual reductions in fuel consumption, delay, and stops of 9.1 percent (30 million gallons), 24.6 percent (43 million hours), and 14.2 percent (1.7 billion stops) respectively (29). One should note that signal retiming benefits citizens directly by reducing fuel consumption, delay time, and the number of stops at signalized intersection.

Example Calculations. In an example problem that represents a signal retiming project, analysts used PASSER II-87 to evaluate existing conditions at 25 signalized intersections in a freeway corridor network, shown in Figure 7-1, and then to produce an optimized timing plan. PASSER II reports the following measurements of effectiveness: stops, delay, and fuel consumption.

To estimate the total benefits of an optimized signal system, one should multiply the delay reduction (or other improvements reported by an analysis tool, such as PASSER II) by the number of hours a timing plan operates. If three timing plans are used in a day, typically one will use the a.m. and p.m. peak timing plans for one to two hours each, and the off-peak timing plan will be used for seven to ten hours for benefit analysis, i.e. ten to fifteen hours of the day will be used for benefit analysis. The following steps show how one may calculate benefits 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 stops, delay, 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.

Figure 7-1. Example Freeway Corridor Network System

2. **Compute Benefits for Each Timing Plan.** For each timing plan, the savings (stops, delay, or fuel consumption) should be multiplied by the number of hours that the timing plan operates. As discussed previously, the a.m. peak reduction may be multiplied by 2.5 hours, the p.m. reduction by 2.75 hours, the midday reduction by 5.25 hours, and the off-peak delay reduction by, 7.5 hours.

3. **Compute Daily Benefits.** Next, the reductions (stops, delay, and fuel consumption) for each timing plan should be summed; (a.m. reduction * 2.5) + (p.m. reduction * 2.75) + (midday reduction * 5.25) + (off-peak reduction * 7.5). 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, these reductions per day should be multiplied 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 timing plan equals three to five years. To estimate the benefit of reductions over the life of a project, the yearly reductions (stops, delay, and fuel consumption) should be multiplied by the life of the project say five years. 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 (30) per stop, per vehicle-hour of delay, and per gallon of fuel.

Example calculations show the Before and After conditions along a freeway corridor, as seen in Table 7-1. The difference in the Before (existing) and the After (optimized) conditions is:

Stops = 10,522 stops/day (8.0 percent reduction)
Delay = 1,763 veh-hrs/day (37.6 percent reduction)
Fuel = 1,273 gallons/day (16.6 percent reduction)

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	23594	22141	514.84	228.71	766.94	569.96
	OFF	12759	11711	106.03	99.37	323.89	312.18
	MID	18926	19521	260.0	201.9	514.84	487.58
	PM	30832	31102	606.69	370.15	927.21	751.33
HRS/DAY	AM	2.50	2.50	2.50	2.50	2.50	2.50
	OFF	7.50	7.50	7.50	7.50	7.50	7.50
	MID	5.25	5.50	5.25	5.50	5.25	5.50
	PM	2.75	2.50	2.75	2.50	2.75	2.50
DAILY	AM	58985	55353	1287	572	1917	1425
	OFF	95693	87833	795	745	2429	2341
	MID	99362	107366	1365	1110	2703	2682
	PM	84788	77755	1668	925	2550	1878
DIFFERENCES	AM	3633		715		492	
	OFF	7860		50		88	
	MID	-8004		255		21	
	PM	7033		743		672	
	TOTAL	10522		1763		1273	
UNIT VALUES		$0.014		$10.00		$1.00	
ANNUAL SAVINGS		$44,190		$5,288,543		$381,899	
PROJECT COST :		$14,621		TOTAL SAVINGS (FIVE YEARS)		$5,714,632	

7.2. Benefit-Cost Analysis

When considering the question of how much to budget for signal retiming projects, one should consider total costs and potential benefits. For example, say a district has 450 signals and a total budget for the signal section of $1,387,000, as shown in Figure 7-2. 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. One may estimate costs by person-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 is for 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 (31). Another estimate figured one person-week for retiming a signal, which corresponds to one person timing 50 signals in a year; of course, several persons work on one project at a time. These estimates include data collection and development of timing plans. Costs will be higher for geometric improvements or major signal hardware replacement.

After 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 the signal retiming projects. One should note that from the previous example (Table 7-1), motorists saved $735 for every dollar spent in signal retiming project.

After the new signal timing plan is implemented, fine-tuned and documented, 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 analyst should install the most versatile controller equipment and signal hardware to accommodate future growth and fluctuations. As demonstrated by this example, retiming signals can prove a cost-effective means of 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 7-2. Example District's Budget for a Signal Section

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

COST ITEM	LEVEL/TYPE	TIME	COST	COMMENTS
Personal	Engineer	20 hrs	$715.60	$37.78 per hour
	Oprtns. Supt	32 hrs	$743.68	$23.24 per hour
		40 hrs	$990.00	$24.75 per hour
	Traffic Tech.	32 hrs	$427.52	$13.36 per hour
		115 hrs	$1,656.00	$14.40 per hour
	Total		$4532.80	Hourly rates include salary plus 30 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 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. One should document the final signal timing plan agreed upon for implementation. This documentation includes recording 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. Any unusual design procedures or engineering judgment should be recorded and explained.

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

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