EVALUATION OF SIGNAL TIMING AND COORDINATION PROCEDURES

Volume I: Technical Report

by

E. D. Arnold, Jr.
Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

Based on a review of available literature, recommended procedures for timing the various types of signals are provided. Specifically, procedures are included for both pretimed and vehicle-actuated controllers located at isolated intersections and at intersections in a signal system. Simplicity and ease of use are emphasized as the targeted users are field technicians and those responsible for signals in small cities and towns. A separate Field Manual has been prepared which is intended to provide a concise and easily applied set of procedures. Detailed theory and logic behind the procedures are provided in the Technical Report, as are brief descriptions of current computer programs which provide timing information.

The Technical Report also presents the results of a questionnaire survey which had the objective of determining the types of signal equipment used in Virginia.

SI CONVERSION FACTORS

To Convert From	To	Multiply By
Length:		
in	cm	2.54
in	m	0.025 4
ft	m	0.304 8
yd	m	0.914 4
mi	km	1 . 609 344
Area:		
in^2	cm^2	6.451 600 E+00
ft^2	m^2	9.290 304 E-02
yd^2	m^2	8.361 274 E-01
mi^2	Hectares	2.589 988 E+02
acre (s)	Hectares	4.046 856 E-01
Volume:		
oz	m^3	2.957 353 E-05
pt	m^3	4.731 765 E-04
qt	m^3	9.463 529 E-04
gal	m^3	3.785 412 E-03
in^3	m^3	1.638 706 E-05
ft^3	m^3	2.831 685 E-02
yd^3	m^3	7.645 549 E-01

NOTE: 1m^3 = 1,000 L

To Convert From	To	Multiply By
Volume per Unit Time:		
ft^3/min	m^3/sec	4.719 474 E-04
ft^3/s	m^3/sec	2.831 685 E-02
in^3/min	m^3/sec	2.731 177 E-07
yd^3/min	m^3/sec	1.274 258 E-02
gal/min	m^3/sec	6.309 020 E-05
Mass:		
oz	kg	2.834 952 E-02
dwt	kg	1.555 174 E-03
lb	kg	4.535 924 E-01
ton (2000 lb)	kg	9.071 847 E+02
Mass per Unit Volume:		
lb/yd^2	kg/m^2	4.394 185 E+01
lb/in^3	kg/m^3	2.767 990 E+04
lb/ft^3	kg/m^3	1.601 846 E+01
lb/yd^3	kg/m^3	5.932 764 E-01
Velocity: (Includes Speed)		
ft/s	m/s	3.048 000 E-01
mi/h	m/s	4.470 400 E-01
knot	m/s	5.144 444 E-01
mi/h	km/h	1.609 344 E+00
Force Per Unit Area:		
lbf/in^2	Pa	6.894 757 E+03
lbf/ft^2	Pa	4.788 026 E+01
Viscosity:		
cS$_t$	m^2/s	1.000 000 E-06
P	Pa·s	1.000 000 E-01

Temperature: °F-32)^5/9 = °C

AN EVALUATION OF SIGNAL TIMING AND COORDINATION PROCEDURES

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INTRODUCTION

The Manual on Uniform Traffic Control Devices (MUTCD) defines a traffic signal as a power-operated traffic control device by which traffic is alternately directed to stop and permitted to proceed. Signals are most commonly used at street intersections to control the assignment of vehicular or pedestrian right-of-way; thus, they exert a significant influence on traffic flow. Signals that are warranted and are properly designed, installed, and operated provide for the orderly, efficient movement of traffic. They also increase the traffic handling capability of the intersection and reduce the frequency of certain types of accidents.

One of the most important elements of signal operation is signal timing, which can be defined as the proper assignment of time to the various vehicular or pedestrian movements at a particular intersection. As compared to a correctly timed signal, a signal timed improperly can result in increases in delay, in gasoline consumption and air pollution, and in certain types of accidents. Signal timing has received a great deal of attention in recent years as the importance of utilizing the existing transportation system in the most efficient manner has been recognized.

There are many signal timing procedures and strategies, and they vary according to the type and capability of the controller and the traffic requirements at the intersection. Pretimed and vehicle-actuated controllers are timed differently, as are the signals located at isolated intersections, at intersections along an arterial, and at intersections in a system network. Further, the timing procedures range from simple, manual techniques to comprehensive techniques applicable to mainframe or minicomputers. Techniques are available or are being developed for the microcomputer.

PURPOSE AND SCOPE

Accordingly, the main purpose of the study was to compile in a single document recommended procedures for timing the various types of signals. Specifically, procedures are included for both pretimed and vehicle-actuated controllers located at isolated intersections and at intersections in a signal system. Simplicity and ease of use are emphasized in the procedures as the targeted users are field technicians and those responsible for maintaining signals in small cities and towns. The procedures are based on a synthesis of the pertinent literature. Finally, brief descriptions of popular computer programs which calculate timing are included.

A secondary purpose was to survey jurisdictions responsible for signals in Virginia to determine the types of equipment being utilized. The survey was conducted through a mail-back questionnaire to all cities and towns, the two counties that maintain signals, and the Department of Highways and Transportation's field offices.

FORMAT AND USE OF REPORT

A separate Field Manual sets forth the recommended procedures in a simplified, step-by-step manner exclusive of detailed, background discussion. Designed for the most part to stand by itself, the Field Manual is intended to provide the user with concise and easily applied procedures for timing the various types of signals. The Technical Report provides the user with the theory and logic underlying the summarized procedures in the Field Manual and should be reviewed to obtain a thorough understanding of timing.

Also, some of the definitions and timing procedures are applicable to more than one category of signals. In these cases, the information is often duplicated for the convenience of the user.

INVENTORY OF EQUIPMENT

A survey was conducted to determine the types of control equipment in use in the state of Virginia. The questionnaire in Appendix A was sent to 65 cities and towns, the 2 counties that maintain signals, and the 9 construction districts of the Virginia Department of Highways and Transportation. Responses were received from 26 cities, 16 towns, 2 counties, and 9 construction districts. Following is a summary of the survey results.

Manufacturers

Table 1 shows the responding jurisdictions and the manufacturers of the signal controllers that each has. The most commonly used controllers in Virginia were manufactured by Crouse Hinds, Automatic Signal, or Eagle. It is interesting to note that the Department utilizes the largest variety of manufacturers, partly because of the large number it maintains and partly because it continually purchases equipment on a low bid basis.

Isolated Intersections

Based on the responses received, essentially all of the Department's signalized intersections are actuated -- approximately 20% operate semi-actuated, 50% operate fully actuated, and 30% operate fully actuated with volume-density timing. On the other hand, local jurisdictions, especially small cities and towns, maintain a significant number of pretimed signals. Seventeen percent of the isolated intersections are under pretimed control, whereas 26% are semi-actuated, 48% are fully actuated, and 9% are fully actuated with volume-density timing.

Signal Systems

The Department maintains 46 systems, all of which are arterial systems. Approximately 200 intersections are included in these systems, and the majority of these operate semi-actuated. Twenty-six of the systems utilize time-based coordinators, whereas the other 20 are hard-wired through a street master controller.

Local jurisdictions reported a total of 82 systems with 919 intersections. Sixty-three are arterial systems. Fifty-nine percent of the intersections in the systems operate pretimed, 25% operate semi-actuated, 13% operate fully actuated, and 3% operate fully actuated with volume-density timing. Only 13% are coordinated through time-based coordination; the remaining 87% have hard wire interconnection. Eighty-two percent of the interconnected systems are controlled by a street master, 14% by a central computer, and 2% by a time clock.

Table 1

Manufacturers of Signal Controllers Used in Virginia

State & Counties	Crouse-Hinds (Traffic Control Technologies)	Automatic Signal	Eagle	Marbelite/ G.E.	Econolite	Multisonics	Safetran/ Singer	EMCO	TRACONEX	Transyt	Honeywell
VDH&T	X	X	X	X			X	X		X	X
Henrico	X	X	X		X						
Arlington	X	X	X								
Cities											
Charlottesville	X	X					X				
Chesapeake	X	X	X				X				
Clifton Forge	X			X							
Colonial Heights	X	X	X								
Covington	X										
Danville	X	X							X		
Emporia	X										
Fredericksburg	X										
Hopewell	X	X	X				X				
Lexington	X	X	X	X							
Lynchburg	X	X				X					
Martinsville	X	X									
Newport News	X	X		X							
Norfolk	X	X	X								
Poquoson	X										
Radford	X		X	X							
Richmond	X	X	X	X							
Roanoke	X	X	X		X						
Salem	X	X	X								
South Boston	X	X									
Staunton	X	X		X							
Suffolk		X				X					
Va. Beach	X	X				X					
Waynesboro	X	X		X		X					
Williamsburg	X	X					X				
Winchester	X	X	X				X				
Towns											
Abingdon		X									
Ashland		X									
Big Stone Gap	X				X						
Blackstone	X										
Bluefield		X						X			
Christiansburg		X	X	X							
Front Royal	X	X									
Herndon	X	X	X								
Marion			X								
Pulaski	X						X				
Richlands	X			X							
South Hill		X									
Vienna	X										
Warrenton		X									
Wise		X									

Auxiliary Equipment

In recent years the functions performed by auxiliary, stand-alone equipment have been incorporated into the signal controller itself. The Virginia Department of Highways and Transportation, as well as several local jurisdictions, reported that the stand-alone equipment is being phased out as quickly as possible. Of the stand-alone equipment remaining, the most common types by far are minor movement controllers and coordination units.

Detectors

The most common type of detector in use in Virginia is the inductive loop detector. Six of the Department's construction districts reported actual numbers of detectors, and 80% are loop detectors and 20% are magnetic detectors. For the districts providing estimated percentages of each kind of detector, the average percentages are 65% loops, 30% magnetics, 2% magnetometers, and 3% radar.

For those local jurisdictions which reported actual numbers of detectors, it was found that 77% of the detectors are loops, 13% are magnetics, 8% are magnetometers, and 2% are pressure sensitive. For those 7 jurisdictions reporting a percentage breakdown by type of detector, the average percentages are 80% loops and 20% magnetics.

Availability of Computers

Computers are becoming increasingly available at the local level. Within the Department, four of the districts reported the availability of a microcomputer.

At the local level, 61% of the respondents indicated the availability of a computer, with 30% of those having access to a mainframe, 7% to a mini, and 63% to a microcomputer. The various models of IBM computer are the most common. Of particular note is the fact that 10 of the 17 microcomputers are IBM.

Conclusions

Based on the results of the questionnaire survey regarding types of signals utilized in the state, the following general conclusions can be made.

1. Controllers manufactured by Crouse-Hinds (now called Traffic Control Technologies), Automatic Signal, and Eagle are the most common in

Virginia. This is the same finding reported in an inventory obtained in 1976.

2. The Department maintains only 4 pretimed signals at isolated intersections; the remainder are actuated. On the other hand, approximately 17% of the signals at isolated intersections reported by local jurisdictions are pretimed. Old, pretimed equipment is quite common in the small cities and towns. Therefore, it is still important to discuss timing procedures for pretimed signals.

3. The Department maintains 46 signal systems, whereas 44 of the 67 local jurisdictions maintain 82 systems. All of the Department's systems are arterial systems, whereas 63 of the local systems are arterial systems. The remaining 19 are grid systems.

   Over 1,100 intersections are known to be in a system. The majority of the local jurisdictional intersections operate pretimed, whereas the majority of the Department's intersections in a system operate semi-actuated. This is explained by the fact that the 19 grid systems reported by local jurisdictions contain predominantly pretimed intersections.

   Of the Department's 46 systems, 26 use the new time-based coordination. Only 11 of the local jurisdictional systems use time-based coordination. This is explained by the fact that the Department is continually upgrading and expanding its signal systems. Due primarily to budget constraints, cities and towns cannot do this on a routine basis.

4. Auxiliary, stand-alone equipment is generally being phased out through modernization programs as the functions performed by this equipment are built into the new replacement controllers. Auxiliary equipment that is still commonly found includes minor movement controllers and coordination units.

5. Inductive loop detectors are the most commonly used type. For those respondents reporting actual numbers of detectors, it was found that there are approximately 5,700 loops, or 78% of the total number of detectors. The next most common, at 1,200 and 17%, are magnetic detectors. There are only a few magnetometers, radar, and pressure detectors.

6. Computers are available to a limited extent at the local level. Seventeen of the responding jurisdictions have microcomputers, whereas another 10 have access to a mini or mainframe computer. Only 4 of the Department's construction districts reported the availability of a microcomputer; however, the other 5 should be receiving micros shortly. Thus, the use of signal timing computer

programs is feasible for many of the agencies maintaining signals in Virginia.

TIMING FOR PRETIMED SIGNALS AT ISOLATED INTERSECTIONS

Background

A pretimed controller operates according to a predetermined schedule; that is, it has a fixed cycle length which is subdivided into discrete, preset phases to accommodate required individual traffic movements. This type of equipment is best suited when traffic patterns and volumes are predictable and do not vary significantly. There is some flexibility in timing as most controllers allow for at least three independent timing plans, which are generally based on time of day or day of week variations in the traffic patterns.

Definitions

The following definitions are applicable to timing pretimed signals. See Figure 1.

1. Timing plan - a unique combination of cycle length and split.

2. Cycle - the time required for one complete sequence of signal indications.

3. Phase - that part of a signal cycle allocated to any combination of one or more traffic movements simultaneously receiving the right-of-way during one or more intervals.

4. Interval - a discrete portion of the signal cycle during which the signal indications remain unchanged.

5. Split - the percentage of a cycle length allocated to each of the phases.

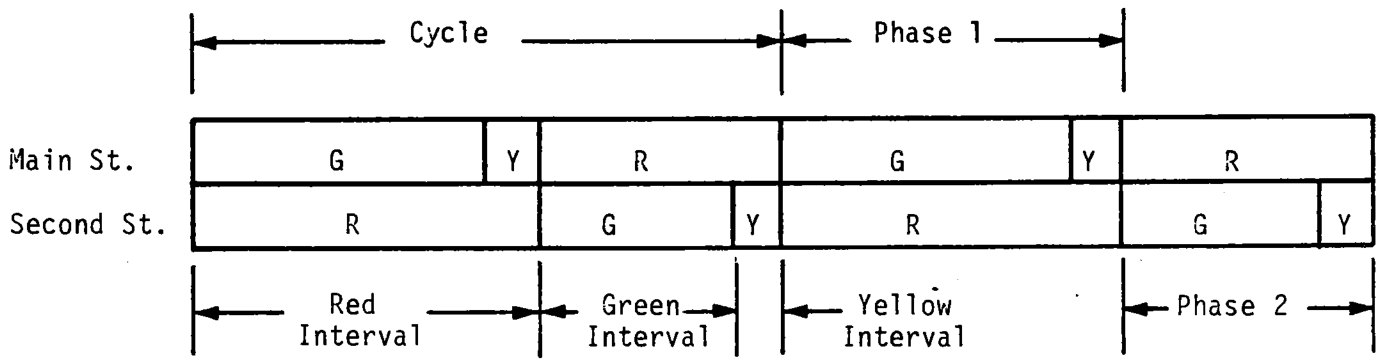

Figure 1. Timing sequence for simple two-phase controller.

Objective

The major objective of signal timing is to assign the right-of-way to alternate traffic movements so that all vehicles are accommodated with a minimum amount of delay to any single group of vehicles. Short cycle lengths minimize average delay, or delay to single groups of vehicles, provided the capacity of the cycle to pass vehicles is not exceeded. If there is a constant demand, however, long cycles will accommodate more vehicles over a given period of time because there is a lower frequency of starting delays and clearance intervals between phases. Satisfying the objective of signal timing, therefore, results in conflicting requirements for the cycle length. Thus, the objective should be restated to that of determining the shortest cycle length which will accommodate the traffic demand, within certain limits.

Timing Procedures

A summary of the recommended procedures for timing a pretimed signal is listed below. The basic concepts for each step along with appropriate examples of using the procedures are described in the remainder of this section. Because of the relationships among physical data, type of equipment, timing plans, and phasing, it may be necessary to undertake steps 1 through 3 simultaneously if a new signal is being installed.

1. Determine the number of timing plans needed.

2. Collect necessary information at the intersection.

3. Determine number of phases needed.

4. Calculate passenger car equivalents.

5. Find critical lane volumes.

6. Calculate optimum cycle length.

7. Calculate cycle splits.

8. Calculate phase change interval.

9. Check for minimum phase time.

10. Check for minimum pedestrian requirements.

11. Verify or adjust timing after actual field observation.

Setting of the timing values is dependent on the controller. The settings may be in percent of cycle, to the nearest whole second, or to the nearest tenth of a second.

Determine Number of Timing Plans

The maximum number of timing plans is determined by the type of controller. The typical three-dial electromechanical controller can provide for three independent timing plans, one per dial. The modern microprocessor-based controller is generally capable of a total of 12 plans, a combination of at least four cycles and three splits. The variation or pattern of traffic demand at an intersection determines the number of plans. Traffic demand patterns are typically categorized as a.m. or p.m. peak period, average or midday period, late night or low volume period, weekend period, shopping period, evening period, or special function period. Within the capabilities of the controller, each of these well-defined periods would normally receive a separate timing plan. It can generally be assumed that a minimum of two plans are needed -- one for peak conditions and one for off-peak conditions. Two plans are often needed for peak condition -- inbound peak and outbound peak.

Extensive traffic counting may be undertaken to evaluate the daily or weekly variations in traffic demand in order to determine the number of timing plans needed. However, this determination is most often based on local knowledge of the traffic conditions coupled with the limitations of the controller at the intersection.

With the exception of information on existing controller equipment and physical data, the remaining procedures apply to each timing plan needed.

Collect Necessary Intersection Information

Basic information concerning the intersection must be obtained in order to apply the actual timing procedures described later. Following is a description of the minimum data needed to calculate signal timing. Effective timing is dependent upon the accuracy of the input data.

Control Equipment

Knowledge of the control equipment already at the intersection or equipment to be installed is mandatory. The controller's timing functions and their characteristics and limitations must be known. In the case of equipment already at the intersection, information on its timing, especially the number of timing plans and phases, is important to know.

Physical Data

The following information concerning the physical dimensions and geometrics at the intersection should be obtained.

1. Number of approaches

2. Number of lanes and type of flow (through, right turn, left turn, or combination) per lane for each approach

3. Width of lanes and medians

4. Percent grade on approaches, if severe

5. Speed limits

6. Location of parking, crosswalks, stop bars, bus stops, loading zones, etc.

Traffic and Pedestrian Data

In order to apply the timing procedures described later, hourly traffic volumes and pedestrian counts are needed on every approach to the intersection. Further, the approach traffic should be categorized into the number of vehicles turning left, going straight through, and turning right. It is also necessary to count and record the number of buses and large trucks per hour on each approach. Finally, the average speed of traffic approaching the intersection on each leg should be obtained.

This hourly information is needed for each timing plan determined earlier. For example, a three-dial controller may have a timing plan for the morning peak period, a lunch or midday period, and an afternoon peak period. In order to calculate the timing for each plan, the above described hourly information representative of these three periods must be obtained. Likewise, traffic and pedestrian data for weekends, nights, and special functions must be obtained in order to calculate the timing for these periods. It is noted that data collected on Tuesdays, Wednesdays, and Thursdays are more representative of average weekday conditions than those collected on Mondays and Fridays.

A typical data collection form is provided in Figure 2. It is noted that the volumes are tabulated by one-half hour intervals during the normal morning and afternoon rush hours. This enables a more accurate determination of peak-hour statistics than would be possible with one-hour summaries. Volume counts by 15-minute intervals would be the most accurate. Figures 3 and 4 show other common forms used to summarize the data for 3-legged and 4-legged intersections, respectively.

Although undesirable, it is possible to derive an estimate of the peak-hour volume based on general relationships. Generally, the 12-hour volume between 7:00 a.m. and 7:00 p.m. is from 70% to 75% of the 24-hour volume and the peak-hour volume is from 10% to 12% of the 24-hour volume. Thus, if either a 12- or 24-hour count is conducted or known, then the peak-hour volume can be estimated. Further, approximately 60% of the traffic volume during the peak hour is in the heavier direction in suburban areas. In central areas the approximate percentage in the heavier direction of flow is 55%. As an example of the usage of these relationships, a 12-hour count at an intersection in a suburban setting shows a volume of 700 vehicles. Thus, the 24-hour volume can be estimated at 1,000 vehicles and the peak-hour volume, which generally occurs in the afternoon, can be estimated at 100 vehicles. Finally, the two approach volumes can be estimated at 60 vehicles and 40 vehicles. It is emphasized that actual traffic counts provide much better timing than counts estimated from these relationships.

Determine Number of Phases

As a general rule, the number of phases should be kept to a minimum. Cycle lengths that are long result in delays to individual groups of vehicles awaiting the green indication; therefore, there are practical limits to cycle lengths in order to avoid these intolerable delays. Accordingly, additional phases tend to decrease the available green time for other phases since they must be accommodated within the practical maximum cycle length. Also, there is additional lost time through start-up delays and phase change or clearance intervals over the course of a cycle as the number of phases increases.

DIRECTIONAL TRAFFIC MOVEMENT – INTERSECTION OF ROUTES ____________

COUNTY ____________ LOCATION ____________

DATE ____ / ____ / ____ WEATHER ____________

	APPROACHING INTERSECTION																
	ON ROUTE						FROM THE		ON ROUTE						FROM THE		
	EAST				WEST				NORTH				SOUTH				
HOURS	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	TOTAL
6:00 - 7:00																	
7:00 - 7:30																	
7:30 - 8:00																	
8:00 - 8:30																	
8:30 - 9:00																	
9:00 - 10:00																	
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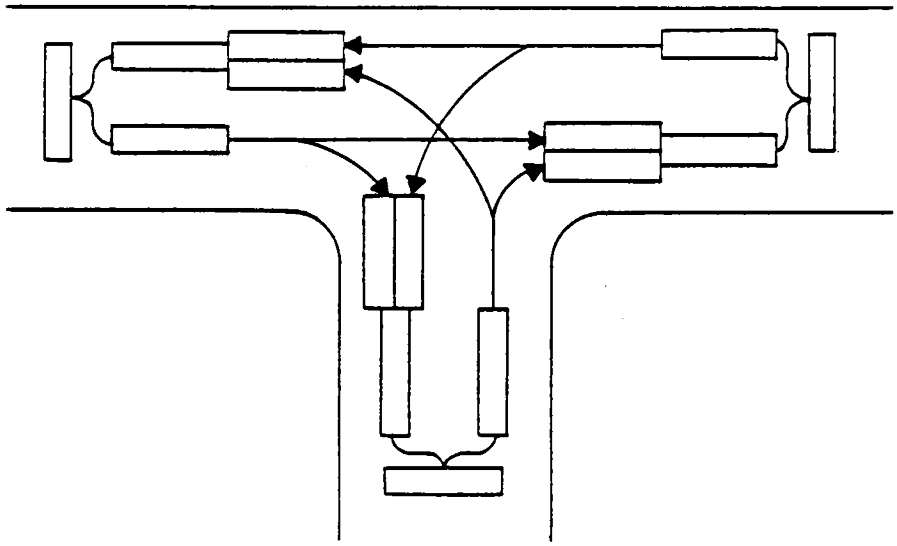
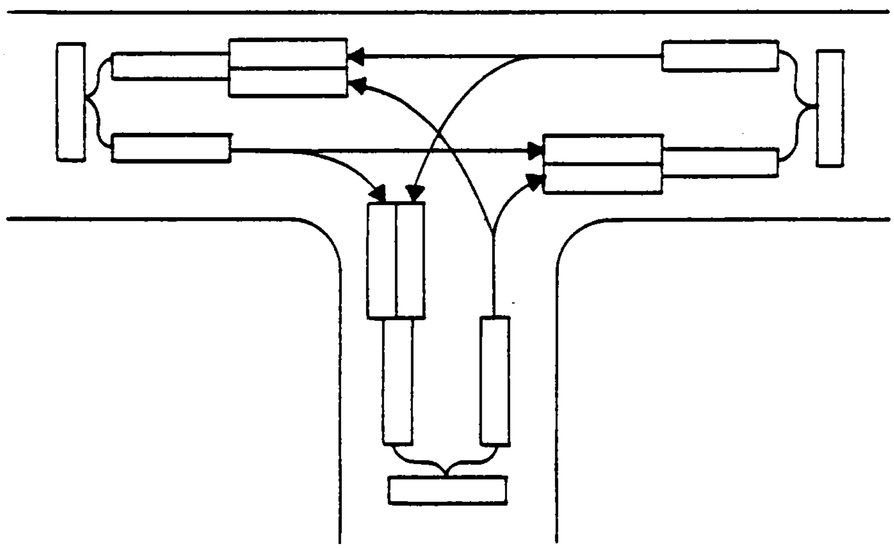
3:00 - 4:00																	
4:00 - 4:30																	
4:30 - 5:00																	
5:00 - 5:30																	
5:30 - 6:00																	
12 - HOUR TOTAL																	
24 - HOUR TOTAL																	

RECORDED BY ____________ ____________

SUPERVISOR ____________

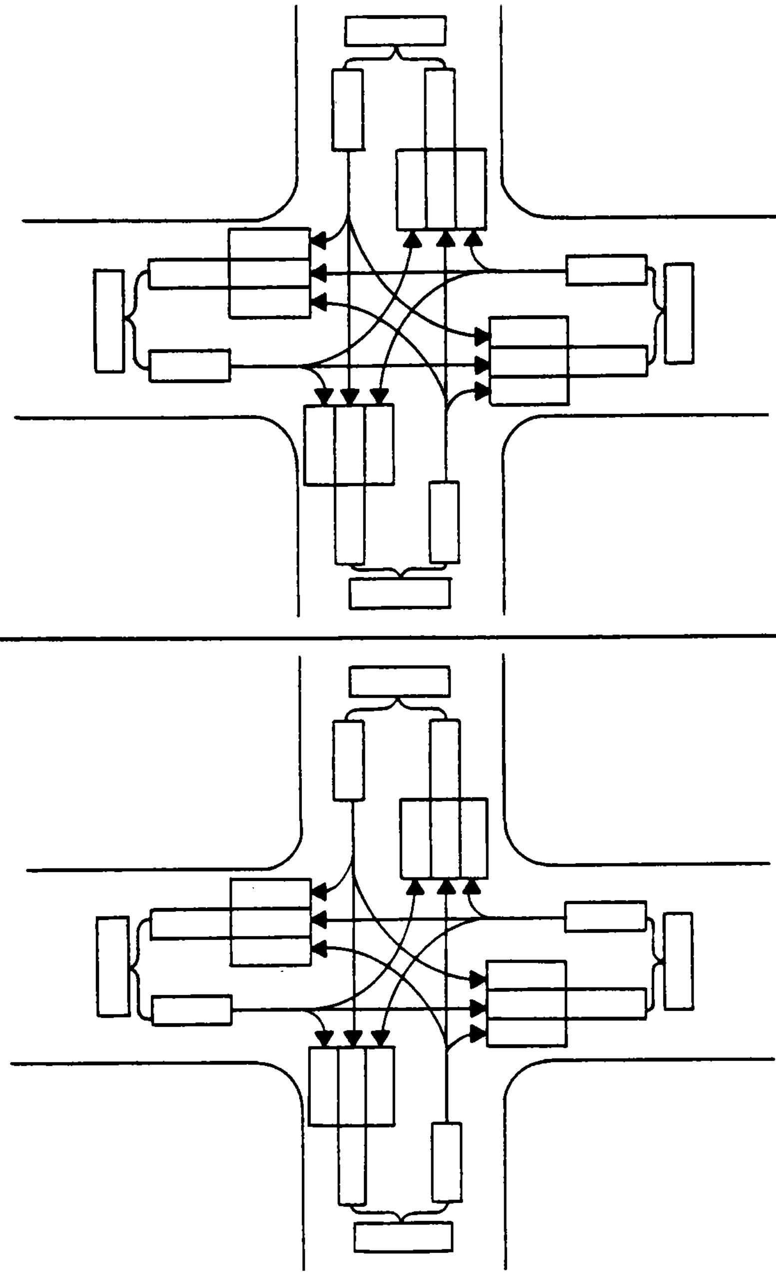
Figure 2. Typical data collection form.





EIGHT MAXIMUM HOUR VOLUMES OF APPROACH VEHICLES									
TIME		RTE.		RTE.		RTE.		TOTAL	
FROM	TO	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.
TOTAL									

Figure 3. Typical data summary form for three-legged intersection.



EIGHT MAXIMUM HOUR VOLUMES OF APPROACH VEHICLES											
TIME		RTE.		RTE.		RTE.		RTE.		TOTAL	
FROM	TO	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.
TOTAL											

Figure 4. Typical data summary form for four-legged intersection.

The number of phases required at an intersection is most often a left-turn issue. As the volumes of left-turn and opposing traffic increase, it becomes more difficult for the traffic turning left to find adequate gaps. A separate left-turn lane can alleviate the problem to some degree by providing storage for vehicles awaiting an adequate gap; however, at a certain point a separate phase for movement from that left-turn lane is needed. The following guidelines applicable to intersections having separate left-turn lanes may be used when considering the addition of separate left-turn phases. These are contained in a recent report entitled Guidelines for Exclusive/Permissive Left-Turn Signal Phasing, by B. H. Cottrell, Jr.(1)

1. Volumes -- consider left-turn phasing on an approach when the product of the left-turn volume and opposing volume divided by the number of lanes during the peak hour exceeds 50,000, provided that the left-turn volume is greater than two vehicles per cycle on average.

2. Delay -- consider left-turn phasing if a left-turn delay of 2.0 vehicle-hours or more occurs in the peak hour, provided that the left-turn volume is greater than two vehicles per cycle on average. Also, the average delay per left-turning vehicle must be at least 35 sec. See Appendix B for a procedure for determining intersection delay.

3. Accident experience -- consider left-turn phasing if the critical number and resulting rate of left-turn accidents have been exceeded. For one approach the critical number is five left-turn accidents in one year. The accident rate, as defined by the annual number of left-turn accidents per 100 million left-turn plus opposing vehicles, must exceed the critical rate determined by the equation

   $Rc = 32.6 + 1.645\sqrt{32.6/M} - 0.5\ M$, where M is the annual left-turn plus opposing volume in 100 million vehicles.

4. Site conditions -- consider left-turn phasing if there is inadequate sight distance, if there are three or more lanes of opposing through traffic, if intersection geometrics promote hazardous conditions, or if there are access management problems.

It is emphasized that the above are guidelines and should be coupled with engineering judgement. More detailed information on these guidelines plus guidelines for using protected/permissive phasing can be found in the above referenced report.

Calculate Passenger Car Equivalents

The timing procedures described later require that volumes be known in terms of passenger car equivalents (PCE) per hour. The use of PCEs accounts for the negative impacts of trucks, buses, and turning vehicles on the traffic handling capability of an intersection. Trucks and buses not only occupy more space than an automobile, but they also require more start-up time due to their acceleration characteristics. Trucks having 6 or more tires and intercity buses should be considered the equivalent of 1.75 passenger cars. Local buses stopping in the vicinity of the intersection have even greater negative impacts than do intercity buses, and should be estimated to be the equivalent of 5.0 passenger cars.

Turning vehicles also have an adverse impact on intersection operation. Left-turning vehicles which must yield to oncoming vehicles should be considered the equivalent of 1.75 passenger cars, and right-turning vehicles yielding to pedestrians on the cross street should be estimated at 1.25 passenger cars if the number of right turns is more than 10% or the number of pedestrians is significant. It is noted that other equivalency factors are sometimes used; however, the above factors are recommended for use with the timing procedures described later. See Table 2 for a summary of the PCE factors.

Table 2

Passenger Car Equivalents (PCE) Factors

Type of Vehicle or Movement	PCE Factor
Trucks (6 or more tires)	1.75
Intercity Buses (e.g., Trailways/Greyhound)	1.75
Local Buses	5.00
Left-turns with Opposing Traffic	1.75
Right-turns Conflicting with Pedestrians (more than 10% right turns)	1.25

As an example, consider the case of an intersection having significant pedestrian flow and a total approach volume on one leg of 1,000 vehicles, of which 10% are intercity buses and trucks, 2% are local transit buses, 15% are left turns, and 12% are right turns. The following steps illustrate the calculation of PCEs for the approach.

Adjust for vehicle types

1.	Number of intercity buses and trucks		= 10% x 1,000 =	100
		PCEs	= 1.75 x 100 =	175
2.	Number of local buses		= 2% x 1,000 =	20
		PCEs	= 5.0 x 20 =	100
3.	Number of passenger cars		= 1,000-100-20 =	880
		PCEs	= 1.0 x 880 =	880
		Total PCEs	= 175+100+880 =	1,155

Adjust for turning movements

4.	Number of left-turning vehicles		= 15% x 1,155 =	174
		PCEs	= 1.75 x 174 =	305
5.	Number or right-turning vehicles		= 12% x 1,155 =	139
		PCEs	= 1.25 x 139 =	174
6.	Number of through vehicles		= 1,155-174-139=	842
		PCEs	= 1.0 x 842 =	842
		Total Approach PCEs	= 305+174+842 =	1,321

Find Critical Lane Volumes

A critical lane volume (CLV) is the highest lane volume in vehicles per hour (vph) for a particular phase. In this step the CLV for all phases must be determined and then summed over the entire intersection. If enough green time is provided to handle the lane having the highest volume during a phase, then there automatically is sufficient green to accommodate other lanes of traffic moving during that phase. The following general rules apply to calculating the CLV.

1. CLVs are calculated in PCEs.

2. Right-turn and left-turn movements are considered part of the through movement unless there are exclusive turn lanes.

3. Exclusive left-turn and right-turn lanes without separate phasing should be assigned the appropriate number of PCEs as determined by the 1.75 factor for opposing traffic to left-turns or the 1.25 factor when right turns are more than 10% or pedestrian flow is significant. The turning volumes should then be compared directly with the through volumes to determine the CLV.

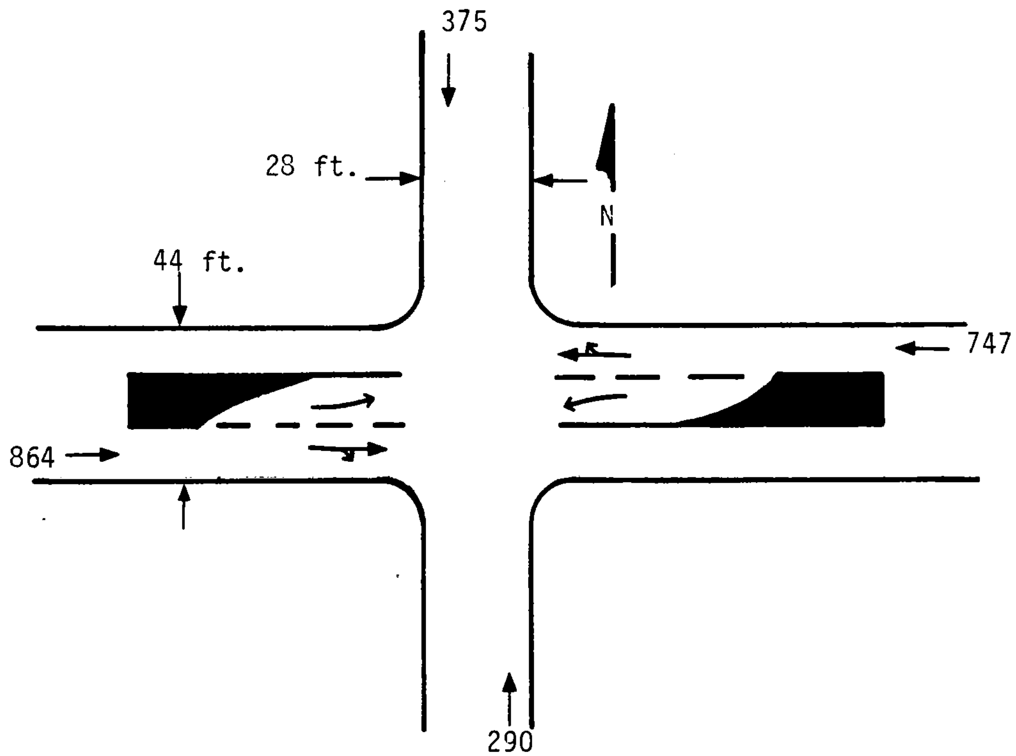
4. If the left-turn movement is protected from conflict with separate phasing, the adjustment factor of 1.75 is not applied.

5. When 2 approach lanes handle through traffic, it should be assumed that the critical lane carries 55% of the volume. Likewise, for 3 approach lanes the critical lane is assumed to carry 37% of the volume.

As an example, consider the intersection shown in Figure 5. Following are the steps necessary to obtain the CLV for each phase.

Characteristics

Pedestrians - minimal
Buses - none
Approach speeds - 25 mi/h
Control - 2-phase, pretimed



Approach	Total Volume (vph)	Passenger Cars (vph)	Trucks (vph)	Left Turns (%)
Northbound	290	255	35	10
Southbound	375	322	53	12
Eastbound	864	786	78	20
Westbound	747	695	52	25

Figure 5. Example intersection A.
Source: Reference 2.

Phase 1 - North/South Movement

	Northbound	Southbound
Trucks as PCEs	1.75 x 35 = 61	1.75 x 53 = 93
Passenger Cars	255	322
Total	316	415
Left-turn traffic	10% x 316 = 32	12% x 415 = 50
Left-turns as PCEs	1.75 x 32 = 56	1.75 x 50 = 87
Through and right-turn traffic	90% x 316 = 284	88% x 415 = 365
Total approach PCEs	340	452

CLV Phase 1 = 452 PCE/hr

Phase 2 - East/West Movement

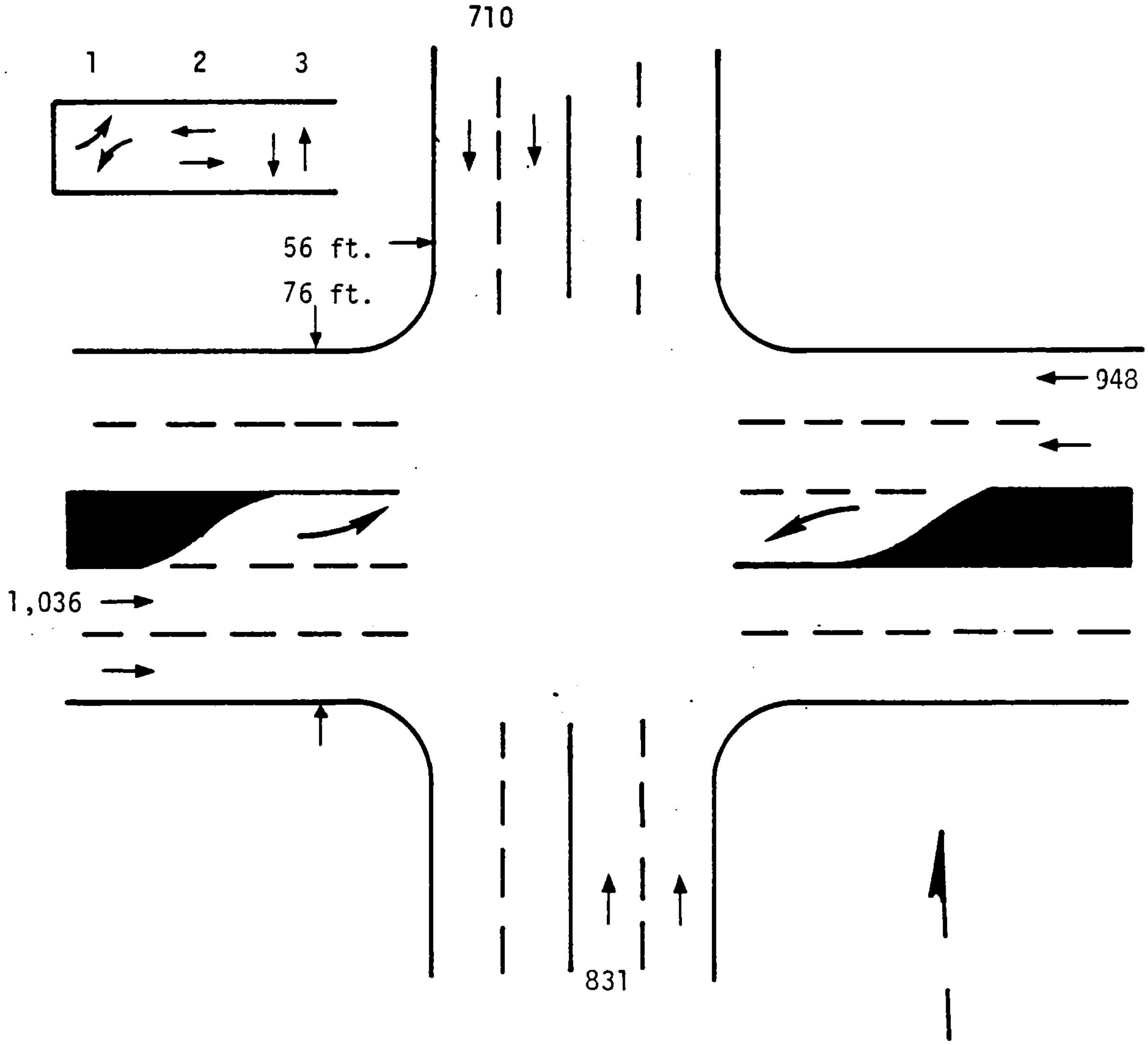
	Eastbound	Westbound
Trucks as PCEs	1.75 x 78 = 137	1.75 x 52 = 91
Passenger Cars	786	695
Total	923	786
Left-turn traffic	20% x 923 = 185	25% x 786 = 197
Left-turns as PCEs	1.75 x 185 = 324	1.75 x 197 = 345
Through and right-turn traffic	80% x 923 = 738	75% x 786 = 590

CLV Phase 2 = 738 PCE/hr (largest of 738, 324, 345, 590)

CLV Total = 452 + 738 = 1,190 PCE/hr

As a second example, consider the intersection shown in Figure 6. Following are the steps necessary to obtain the CLV for each phase.

Pedestrians - minimal
Buses - none
Control - 3-phase, pretimed
Approach speed - 45 mi/h north/south
Approach speed - 55 mi/h east/west



Approach	Total Volume (vph)	Passenger Cars (vph)	Trucks (vph)	Left Turns (%)
Northbound	831	748	83	12
Southbound	710	625	85	14
Eastbound	1,036	995	41	19
Westbound	948	891	57	24

Figure 6. Example intersection B.
Source: Reference 2.

Phase 1 - East/West Left-Turns

	Eastbound	Westbound
Trucks as PCEs	1.75 x 41 = 72	1.75 x 57 = 100
Passenger cars	995	891
Total	1,067	991
Left-turn traffic	19% x 1,067 = 203	24% x 991 = 238

CLV Phase 1 = 238 PCE/hr (factor not applied since left turns are unopposed)

Phase 2 - East/West Through and Right Movements

	Eastbound	Westbound
Trucks as PCEs	1.75 x 41 = 72	1.75 x 57 = 100
Passenger cars	995	891
Total	1,067	991
Through and right-turn traffic	81% x 1,067 = 864	76% x 991 = 753
Citical lane traffic	55% x 864 = 475	55% x 753 = 414

CLV Phase 2 = 475 PCE/hr

Phase 3 - North/South Movement

Trucks as PCEs	1.75 x 83 =	145	1.75 x 85 =	149
Passenger cars		748		625
Total		893		774
Left-turn traffic	12% x 893 =	107	14% x 774 =	108
Left-turns as PCEs	1.75 x 107 =	187	1.75 x 108 =	189
Through and right-turn traffic	88% x 893 =	786	86% x 774 =	666
Total approach PCEs		973		855
Critical lane traffic	55% x 973 =	535	55% x 855 =	470

CLV Phase 3 = 535 PCE/hr
CLV Total = 238 + 475 + 535 = 1,248

Calculate Optimum Cycle Length

As stated earlier, the specific objective of timing a pretimed signal is to determine the cycle which minimizes average delay and will also accommodate the traffic demand. One such technique, Webster's Method, accomplishes this through the equation

$$C = \frac{1.5L + 5}{1-Y}, \qquad (1)$$

where

C = cycle length in seconds which minimizes delay at the intersection,

L = total lost time per cycle in seconds, typically 4.0 to 5.0 seconds/phase, and

Y = total of the ratios of the actual volume to the saturation volume for the critical approaches, with saturation volume typically in the range 1,700 to 1,800 vph.

The delay at the intersection is reasonably constant in the range of 0.75 C to 1.50 C; therefore, a good estimate of C can still be obtained even when simplifying assumptions are made for the above equation. If the lost time per phase is assumed to be 4.0 seconds and the saturation volume is assumed to be 1,800 vph, then equation 1 is modiified as follows.

$$C = \frac{6N + 5}{1 - \frac{CLV_T}{1,800}}, \qquad (2)$$

where

C = as before,

N = number of phases, and

CLV_T = sum of CLVs per phase in PCEs/hr for the intersection.

A graphical solution to equation 2 is presented in Figure 7, and in most cases the optimum cycle can be determined directly from the graph. As an example of the use of Figure 7, consider the previous example intersections A and B. Intersection A has 2-phase control and a CLV_T of 1,190 PCEs/hr. If Figure 7 is entered on the horizontal axis at 1,190, an optimum cycle of 50 seconds can be read from the vertical axis across from the 2-phase curve. Similarly, the 1,248 PCEs/hr at the 3-phase intersection in example B has an optimum cycle of 75 seconds.

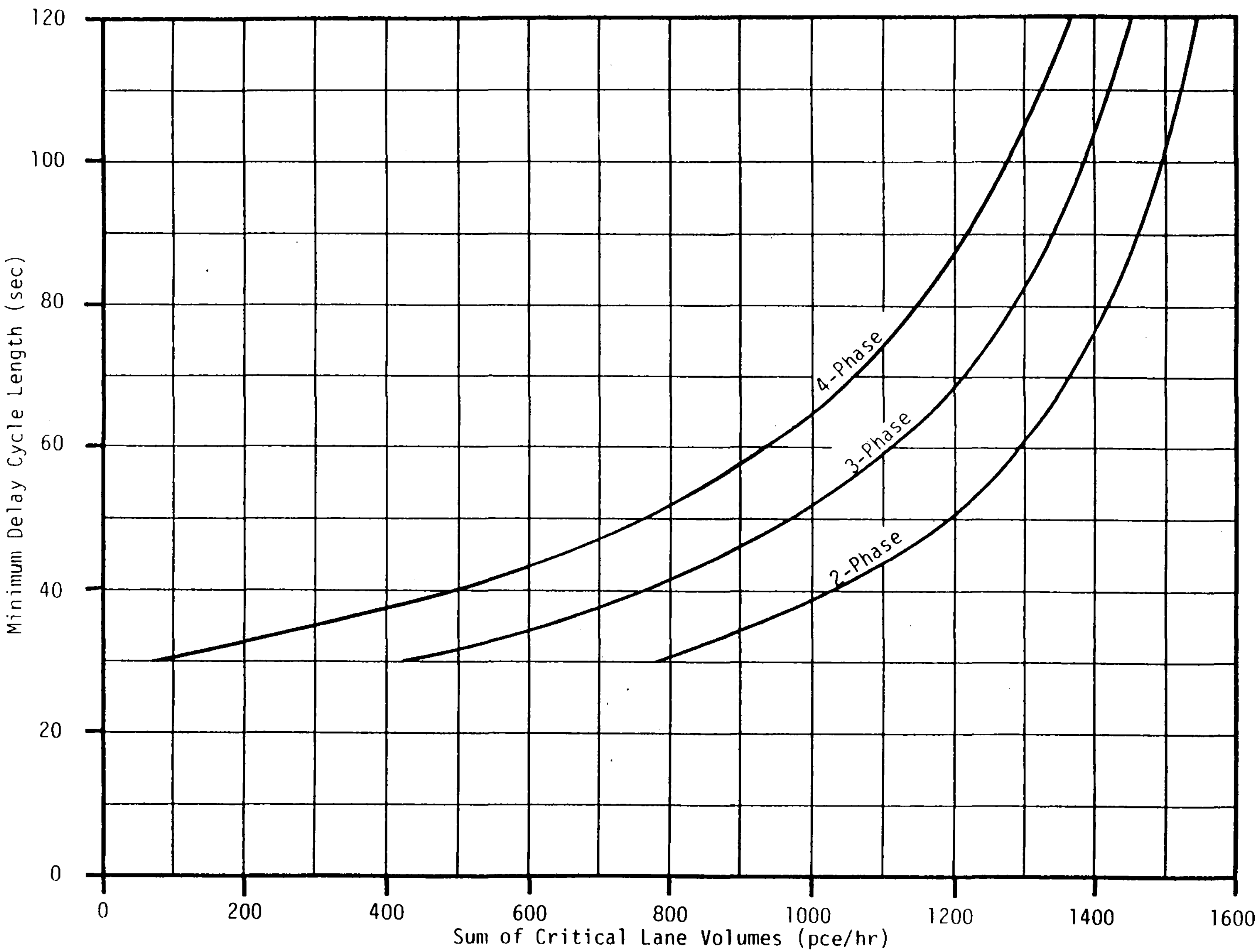


Figure 7. Optimum cycle length for pretimed control.
Source: Reference 2.

It is noted that, in practice, cycle lengths should be no less than 40 seconds and no greater than 120 seconds. In recent years the tendency has been to use longer cycles, even more than 120 seconds. Timing above and below these limits will cause excessive delay and motorists impatience. If a cycle greater than 120 seconds is required, consideration should be given to alternative solutions, such as intersection modifications.

Calculate Cycle Splits

Cycle splits expressed in seconds for each phase can be calculated by the following equation from Webster's Method.

$$(G+A) = \frac{y}{Y}(C-L) + l, \qquad (3)$$

where for the phase being considered

G = green time in seconds,

A = phase change or clearance interval in seconds,

y = ratio of the actual volume to the saturation volume for the critical approach for the phase,

Y = total of the ratios of the actual volume to the saturation volume for the critical approaches,

C = cycle length in seconds,

L = total lost time per cycle in seconds, typically 4.0 to 5.0 seconds per phase, and

l = lost time for the phase.

As before, the above equation can be modiified if an average lost time of 4.0 seconds and a constant saturation volume is assumed for all phases.

$$(G+A) = \frac{CLV(C-4N)}{CLV_T} + 4, \qquad (4)$$

where

$(G+A)$ = phase time in seconds as defined before,

CLV = CLV for phase being considered in PCEs/hr,

CLV_T = sum of CLVs per phase in PCEs/hr for the intersection,

C = cycle length in seconds, and

N = number of phases.

Graphical solutions to equation 4 are presented in Figures 8 through 10 for 2-, 3-, and 4-phase control, respectively. In most cases splits can be obtained directly from the graphs. The total of the phase times should equal the known cycle length. Again, the previously described intersections A and B can be used to exemplify the use of the graphs.

Intersection A: 2-phase, $C = 50$ sec, $CLV_1 = 452$ PCE/hr,

$CLV_2 = 738$ PCE/hr, $CLV_T = 1{,}190$ PCE/hr

$CLV_1/CLV_T = 452/1{,}190 = 0.38$ and $CLV_2/CLV_T = 738/1{,}190 = 0.62$

Therefore, from Figure 8,

Entering 0.38 to the 50 sec line, $(G+A)_1 = 20$ sec,

Entering 0.62 to the 50 sec line, $(G+A)_2 = 30$ sec, and

Total = 50 sec.

Intersection B: 3-phase, $C = 75$ sec, $CLV_1 = 238$ PCE/hr,

$CLV_2 = 475$ PCE/hr, $CLV_3 = 535$ PCE/hr,

$CLV_T = 1{,}248$ PCE/hr

$CLV_1/CLV_T = 238/1{,}248 = 0.19$, $CLV_2/CLV_T = 475/1{,}248 = 0.38$, and

$CLV_3/CLV_T = 535/1{,}248 = 0.43$

Therefore, from Figure 9,

Entering 0.19 to the 75 sec line, $(G+A)_1 = 16$ sec,

Entering 0.38 to the 75 sec line, $(G+A)_2 = 28$ sec,

Entering 0.43 to the 75 sec line, $(G+A)_3 = 31$ sec, and

Total = 75 sec.

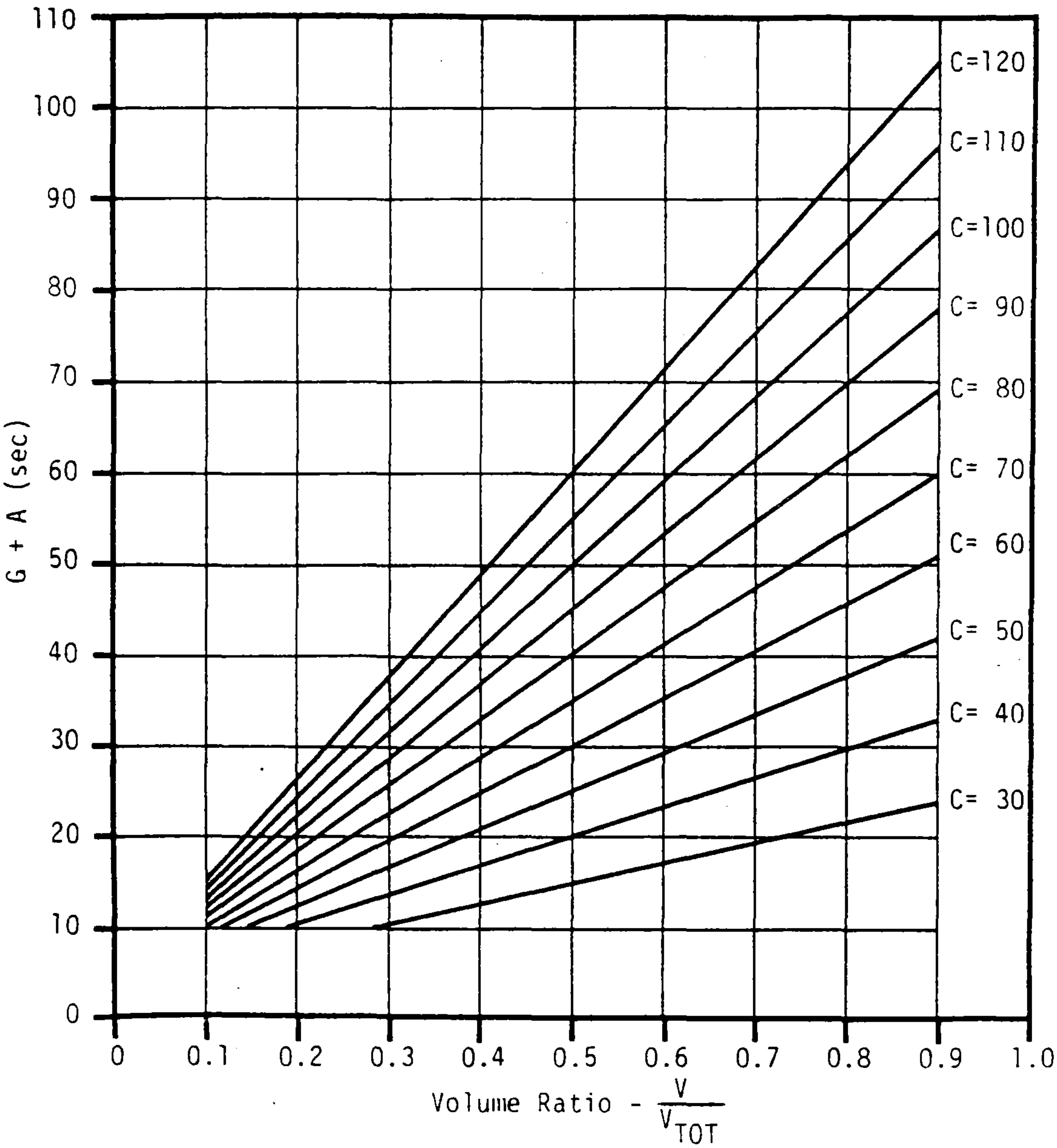


Figure 8. Cycle splits for 2-phase pretimed control.
Source: Reference 2.

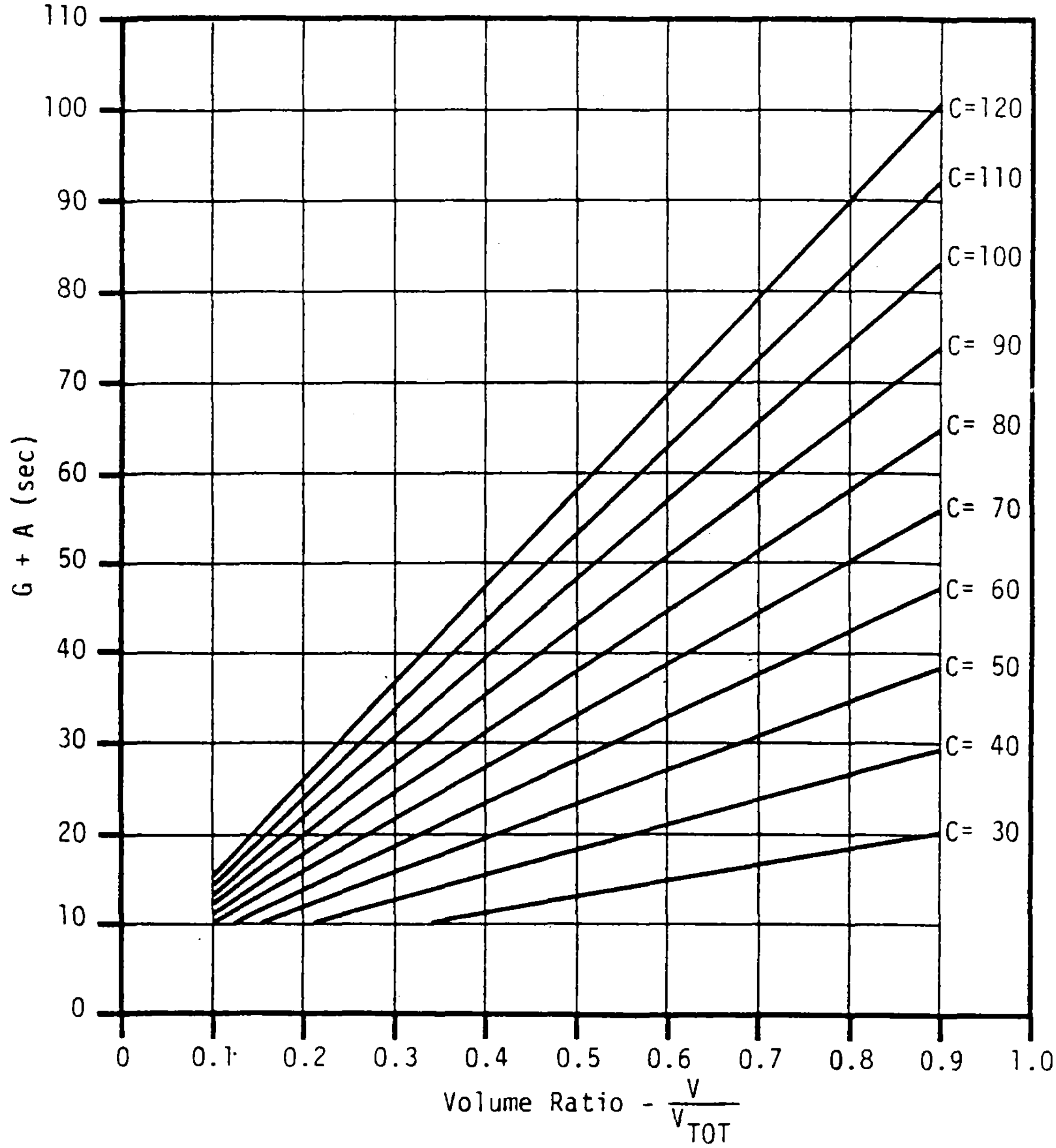


Figure 9. Cycle splits for 3-phase pretimed control.
Source: Reference 2.

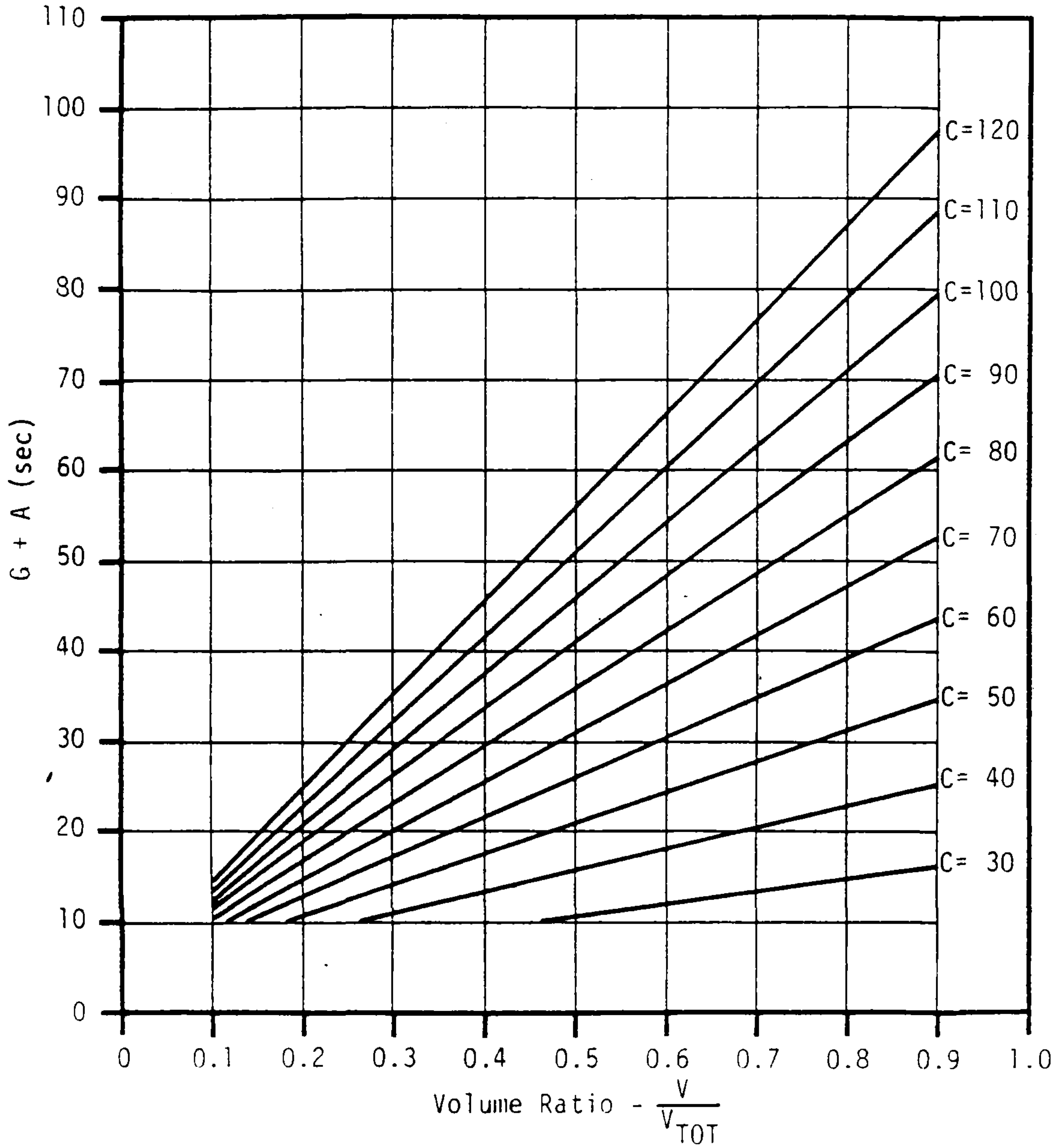


Figure 10. Cycle splits for 4-phase pretimed control.
Source: Reference 2.

It is noted that the phase times include the phase change or clearance interval. Actual green time for each phase is found by subtracting that time as obtained in the next step.

Calculate Phase Change Interval

The purpose of the phase change or clearance interval, which consists of the yellow interval and, possibly, an all-red interval, is to advise motorists of an impending change in the right-of-way assignment, that is; the commencement of a red interval on their approach. Upon commencement of the change interval, a motorist should have sufficient time to either stop his vehicle or clear the intersection. At a given approach speed, a certain amount of time is needed to decelerate to a safe stop at the intersection or proceed through the intersection prior to commencement of the green interval on the cross street. The following equation is used to calculate the phase change interval.

$$CP = t + \frac{V}{2a \pm 64.4g} + \frac{W+L}{V}, \tag{5}$$

where

CP = change period in seconds,

t = perception/reaction time, usually 1.0 second,

V = approach speed in feet/second, typically the 85th percentile speed or prevailing speed limit,

a = deceleration rate in feet/second^2, usually 10 feet/second^2,

W = width of intersection in feet,

L = length of vehicle in feet, usually 20 feet, and

g = percent of grade divided by 100, with upgrade being positive and downgrade being negative.

It is important that motorists have a reasonable expectation of the length of the yellow interval; therefore, the yellow interval should be set in the range of from 3.0 to 5.0 seconds. Within these limits, the yellow interval is often set according to the time it takes to decelerate to a stop; that is, the first two terms in the above equation. Yellow intervals that are longer than necessary decrease capacity and encourage motorists to try to "beat the light."

The time needed to clear the intersection as calculated by the last term in the above equation should be included in an all-red interval where all approaches receive a red indication. Required stopping time above 5.0 seconds should also be included in the all-red interval.

Exclusive turn phases do not typically have an all-red interval. Normally, a through-movement phase follows the exclusive turn movement; therefore, motorists receiving the green directly face straggling left-turners and can safely yield the right-of-way. An all-red interval may be needed, however, at a high-speed intersection or at an intersection with a wide median.

Equation 5, minus the grade factor, coupled with the aforementioned rules regarding the phase change interval have been used to develop the information in Table 3. For a given approach speed, the yellow change interval plus the total phase change interval for various intersection widths are presented. The all-red interval is the difference between the two given intervals. It is sometimes the practice to round up the intervals to the nearest 0.5 second.

As an example of the use of Table 3, again consider the previously described intersections A and B.

Intersection A: 25 mi/h approach speed, all approaches

north/south street - 28 ft, $(G+A)_1 = 20$ sec

east/west street - 44 ft, $(G+A)_2 = 30$ sec

Therefore, from Table 3,

north/south st. - yellow = 3.0 sec, all-red = 1.6 sec, green = 15.4 sec

east/west st. - yellow = 3.0 sec, all-red = 1.2 sec, green = 25.8 sec

Intersection B: 45 mi/h approach speed, north/south

55 mi/h approach speed, east/west

north/south through - 56 ft, $(G+A)_3 = 31$ sec

east/west through - 76 ft, $(G+A)_2 = 28$ sec

east/west left - $(G+A)_1 = 16$ sec

Therefore, from Table 3,

north/south through - yellow = 4.3 sec, all-red = 1.5 sec,
green = 25.2 sec

east/west through - yellow = 5.0 sec, all-red = 1.0 sec,
green = 22.0 sec

east/west left - yellow = 5.0 sec, green = 11.0 sec
(note the absence of an all-red interval)

Table 3

Phase Change Intervals

Approach Speed (mi/h)	Yellow Change Interval (sec)	Total Clearance Interval (Yellow Plus All-Red Clearance) for Crossing Street Widths (ft)				
		30	50	70	90	110
20	3.0	4.2	4.9	5.5	6.2	6.9
25	3.0	4.2	4.7	5.3	5.8	6.4
30	3.2	4.3	4.8	5.2	5.7	6.2
35	3.6	4.5	4.9	5.3	5.7	6.1
40	3.9	4.8	5.1	5.5	5.8	6.1
45	4.3	5.1	5.4	5.7	6.0	6.3
50	4.7	5.3	5.6	5.9	6.2	6.4
55	5.0	5.7	5.9	6.2	6.4	6.7

Source: Reference 3.

Check for Minimum Phase Time

For safety reasons, due primarily to motorists' expectations, there are minimum values on the timing of the phases at an intersection operating under pretimed control. These minimums, including the clearance interval, are 15 seconds for through movements and 12 seconds for turning movements. A quick review of the timing derived for the example intersections will show no violation of these minimums.

Should these minimums be violated, the phase timing should be increased to the minimum and the time added to the total cycle length.

Check for Minimum Pedestrian Requirements

Pedestrian movements at a signalized intersection are typically accommodated by one of the following methods:

1. Pedestrians cross the street with the parallel vehicular green indication with no pedestrian signals.

2. Pedestrians cross the street with the parallel vehicular green indication as instructed by special pedestrian signals.

3. Pedestrians cross the street on an exclusive phase when all vehicular traffic is stopped.

For any of the above methods, sufficient time must be provided for pedestrians to enter the intersection, called the walk interval, and to safely cross the street, called the pedestrian clearance interval. In the first two cases above, the time needed for pedestrians occurs while the parallel vehicular traffic, or traffic on the street <u>not</u> being crossed, is receiving a green and clearance interval. Therefore, the sum of the green and clearance interval for an approach should be long enough to accommodate any pedestrian flow on the cross street.

In many cases the combination of pedestrian and vehicular volumes may not create enough conflicts to warrant a check for the minimum time needed by pedestrians. At locations where there are significant pedestrian volumes or pedestrians require special attention, such as near elderly housing, it is necessary, however, to calculate the needed crossing time and compare it with the time allocated to the movement of parallel vehicular traffic. The walk interval, or time needed by a pedestrian to perceive the signal change and move into the intersection, is generally assumed to be from 4.0 to 7.0 seconds. The higher values are used when pedestrian volumes are high. The pedestrian clearance interval is dependent upon the width of the street being crossed and the walking speed of the pedestrian, which is generally assumed to be 3.5 to 4.0 feet/second. The slower speeds are used when pedestrian volumes are high or in special cases such as in the vicinity of elderly housing.

Except for the special situations mentioned, the following general equation is applicable. Field measurement of walking speeds at the intersection would provide the best data.

$$(G+A)_{min.} = 5 + W/4, \tag{6}$$

where

(G+A) = minimum green plus phase change interval in seconds on approach not being crossed, and

W = width in feet of the street being crossed.

In the case of very wide streets with a median, it may be judged acceptable to allow only enough time for pedestrians to safely reach the median. The entire crossing would then require the timing of two cycles.

If the phase in question does not meet the minimum pedestrian requirements, the timing should be increased to the minimum and the cycle length adjusted accordingly.

The previous example intersections can be used to illustrate the use of equation 6.

<u>Intersection A</u>:	north/south street - 28 ft, $(G+A)_1$ = 20 sec
	east/west street - 44 ft, $(G+A)_2$ = 30 sec
Check:	north/south - 5 + 28/4 = 12 sec less than 30 so o.k.
	east/west - 5 + 44/4 = 16 sec less than 20 so o.k.
<u>Intersection B</u>:	north/south through - 56 ft, $(G+A)_3$ = 31 sec
	east/west through - 76 ft, $(G+A)_2$ = 28 sec
	east/west left - $(G+A)_1$ = 16 sec
Check:	north/south - 5 + 56/4 = 19 sec less than 28 sec so o.k.
	east/west - 5 + 76/4 = 24 sec less than 31 sec so o.k.

Verify or Adjust Timing

The signal timing developed by the preceding procedures should be considered only as a starting point. The procedures are based on typical traffic performance, and factors at the intersection being timed may negate or modify some of the theory or assumptions used. Therefore, it is very important to observe the intersection in operation under the calculated timing in order to either verify the settings or adjust them if necessary.

TIMING FOR ACTUATED SIGNALS AT ISOLATED INTERSECTIONS

Background

A traffic-actuated controller operates in response to traffic demand. Detectors on the roadway "advise" the controller of the presence of vehicles, and that particular movement or phase receives a green indication. That phase retains the green as long as sufficient demand exists, or until a preset maximum time has been reached. Then the controller switches the green to another phase which has been called due to the detection of a vehicle. Thus, within the constraints of the preset maximum times, the controller provides continuously variable cycle lengths and phases in accordance with actual demand. This type of control is very efficient as it allocates the right-of-way based on real time demand, not on the basis of an assumed demand distribution as is the case with pretimed control. It is interesting to note that when the traffic flow is heavy for all movements, the actuated controller functions in pretimed operation with the cycle length and phase times being governed by the preset maximum times.

Definitions

The following general definitions are applicable to timing actuated signals.

1. Cycle - the time required for one complete sequence of signal indications.

2. Phase - that part of a signal cycle allocated to any combination of one or more traffic movements simultaneously receiving the right-of-way during one or more intervals.

3. Detector - a device which detects the passage or presence of a vehicle with the purpose of advising a controller of the need for a green indication. For purposes of this project, detectors will be categorized as either small area detectors or large area detectors. Small area detectors provide passage, point, motion, or unit detection. These detectors simply register the passage of a vehicle. It is noted that a 6 x 6-foot loop is often used as a point detector. Large area detectors provide presence or area detection. These detectors

register the presence of a vehicle in the zone of detection. As will be discussed later, the timing can vary with the type and location of the detectors.

4. Gap - distance between successive vehicles crossing a point on the roadway. For signal timing the "distance" is usually measured in seconds.

Types of Equipment

The three distinct types of actuated equipment are described in the following subsections.

Semi-actuated Controllers

The best use of a semi-actuated controller at an isolated intersection is where the major street volumes are high compared to the minor street volumes. The major street phase is not actuated; therefore, the right-of-way always returns to the major street when there are no vehicles present on the minor street or when the minor street's maximum green time has been reached. This type of operation is also used where the controller is incorporated into a signal system. The non-actuated phase is coordinated with adjacent intersections while the actuated phases are allowed to respond to detected demand within certain limitations. Following is a list of characteristics of semi-actuated control.

1. Detectors are located on only the minor street approaches to the intersection.

2. The major phase, or non-actuated phase, receives a preset minimum green interval.

3. The major phase green extends indefinitely until interrupted by a call from the minor street.

4. The minor phase receives a green indication after it is called if the major phase has completed its minimum green interval.

5. The minor phase receives a preset minimum green; however, the green will be extended by additional calls until a preset maximum green time is reached or until a preset gap in traffic occurs.

6. If the green time is terminated by the preset maximum, a memory feature automatically returns the right-of-way to the minor street once the major street receives its minimum green.

7. The yellow change and all-red clearance intervals are preset for each phase.

Full-actuated Controllers

Full-actuated control has traffic actuations for all phases. This type of control is used at isolated intersections where traffic volumes vary significantly throughout the day and where there is not a large difference between volumes on the major and minor streets. The operational characteristics were generally defined in the previous section on background. Following are specific characteristics of full-actuated operation.

1. Detectors are located on all approaches to the intersection.

2. Each phase receives a preset minimum green; however, the green will be extended by additional calls until a preset maximum green time is reached or until a preset gap in traffic occurs.

3. The yellow change and all-red clearance intervals are preset for each phase.

Volume-density Controllers

Volume-density control is also fully actuated; however, added features enable a more comprehensive evaluation of, and thus response to, traffic conditions than does the basic full-actuated operation. The preset minimum green can be extended so as to accommodate the actual number of vehicles awaiting the right-of-way. Likewise, the preset gap, which is measured in time, can be reduced so as to be more sensitive to traffic flow. The use of volume-density features offers particular advantages on high-speed approaches where detectors are located several hundred feet from the intersection. Specific characteristics are as follows:

1. Detectors are located on all approaches to the intersection.

2. Each phase receives a minimum green which can be extended as additional vehicles queue up at the red indication.

3. Once the minimum or extended minimum green is reached, the green is maintained by additional calls until a preset maximum is reached or a preset gap in traffic occurs. In the case of volume-density control, the preset gap can be reduced after a period of time so that the green is terminated at the occurrence of a smaller gap than necessary at first.

4. The yellow change and all-red clearance intervals are preset for each phase.

Phase Control Functions

Each phase on an actuated controller has several switches which control functions or modes of operation for that phase. Although these functions are not specifically related to timing, it is important to be aware of their operation. Following is a brief description of the modes.

Lock Detector

When a vehicle actuates a detector on a phase which is set in the lock detector mode, that call is "locked in" the memory of the controller until such time as that phase is serviced, or receives a green indication. Small area or point detectors require that the controller be set in this mode.

Non-lock Detector

A phase set in the non-lock detector mode sends a call to the controller only if a vehicle is present in the detection zone. Once the vehicle moves out of the zone, the call for service is cancelled. Large area or presence detectors require this setting. This mode of operation is appropriate for locations where right-turn-on-red occurs and for left-turn phases with exclusive-permissive control.

Non-actuated

A phase set in the non-actuated mode automatically operates under semi-actuated control, with that phase controlling the major street or non-actuated traffic flow.

Recall

When the recall switch on a phase is on, the controller automatically returns to that phase during each cycle. If all recall switches are activated, the controller automatically cycles through all phases. In this case the controller operates in a pretimed manner and all advantages of actuated control are lost. If no recall switches are activated, the controller stays in the last serviced phase indefinitely until a call is received from another phase.

There are several variations of this mode. If "minimum vehicle recall" is set when the detectors are functioning, the controller automatically returns to the phase to service the minimum green and then operates based on demand. A "vehicle recall to max" setting causes the phase's maximum green interval to time out. Finally, a "pedestrian recall" setting causes the pedestrian intervals to time out.

The "vehicle recall to max" switch should be activated to ensure service to a phase if the detectors are broken. It may be beneficial during periods of low volume to have the controller "resting" in green on the major street. In this case the "minimum vehicle recall" switch on the main line phase is activated.

Objective

The major objective of signal timing is to assign the right-of-way to alternate traffic movements so that all vehicles are accommodated with a minimum amount of delay to any single group. Actuated control is responsive within certain limitations to traffic demand, and thus can provide very efficient operation at an intersection. Unlike pretimed control, cycles and phases vary in timing and sequence. Thus, timing actuated controllers involves the understanding of and setting of the preset intervals, or timing parameters, alluded to in the previous discussion on types of controllers. These parameters must be set for each phase in the cycle.

Timing Procedures

As indicated earlier, the essential part of timing actuated intersections is the setting of values for the timing parameters. Several other steps are necessary, however, and following is a list of the recommended procedures. The basic concepts and, if applicable, suggested settings are described in the remainder of this section.

1. Collect necessary information at the intersection.

2. Determine number of phases needed.

3. Determine values for timing parameters.

4. Verify or adjust timing after field observation.

Setting of the timing values is dependent on the controller. The settings may be to the nearest whole second or to the nearest tenth of a second.

Collect Necessary Intersection Information

Basic information concerning the intersection must be obtained in order to apply the actual timing procedures described later. Following is a description of the minimum data needed to calculate signal timing. Effective timing is dependent upon the accuracy of the input data.

Control Equipment

Knowledge of the control equipment already at the intersection or equipment to be installed is mandatory. The controller's timing functions and their characteristics and limitations must be known. In the case of equipment already at the intersection, information on its timing is important to know.

Physical Data

The following information concerning the physical dimensions and geometrics at the intersection should be obtained.

1. Number of approaches
2. Number of lanes and type of flow (through, right turn, left turn, or combination) per lane for each approach
3. Width of lanes and medians
4. Percent grade on approaches, if severe
5. Speed limits
6. Location of parking, crosswalks, stop bars, bus stops, loading zones, etc.
7. Type, location, and, if applicable, size of detectors.

Traffic and Pedestrian Data

Hourly traffic volumes and pedestrian counts are needed on every approach to the intersection. Further, the approach traffic should be categorized into the number of vehicles turning left, going straight through, and turning right. It is also necessary to count and record the number of buses and large trucks per hour on each approach. Finally, the average speed of traffic approaching the intersection on each leg should be obtained.

The traffic and pedestrian data are needed for the peak-flow condition at the intersection. Typically, peak flow occurs during the afternoon rush period; however, side street peak flow may occur at another time during the day. Likewise, peak flow at the entrance to a shopping center may occur around 9:00 p.m. Accordingly, it is important to obtain the data over a period of time which will definitely include the peak-flow condition.

A typical data collection form was provided previously in Figure 2. It is noted that the volumes are tabulated by one-half hour intervals during the normal morning and afternoon rush hours. This enables a more accurate determination of peak-hour statistics than would be possible with one-hour summaries. Volume counts by 15-minute intervals would be the most accurate. Figures 3 and 4 showed other common forms used to summarize the data for 3-legged and 4-legged intersections, respectively. These figures have been reproduced in this section of the report for the convenience of the reader.

Although undesirable, it is possible to derive an estimate of the peak-hour volume based on general relationships. Generally, the 12-hour volume between 7:00 a.m. and 7:00 p.m. is from 70% to 75% of the 24-hour volume and the peak-hour volume is from 10% to 12% of the 24-hour volume. Thus, if either a 12- or 24-hour count is conducted or known, then the peak-hour volume can be estimated. Further, approximately 60% of the traffic volume during the peak hour is in the heavier direction in suburban areas. In central areas the approximate percentage in the heavier direction of flow is 55%. As an example of the usage of these relationships, a 12-hour count at an intersection in a suburban setting shows a volume of 700 vehicles. Thus, the 24-hour volume can be estimated at 1,000 vehicles and the peak-hour volume, which generally occurs in the afternoon, can be estimated at 100 vehicles. Finally, the two approach volumes can be estimated at 60 vehicles and 40 vehicles. It is emphasized that actual traffic counts provide much better timing than counts estimated from these relationships.

DIRECTIONAL TRAFFIC MOVEMENT – INTERSECTION OF ROUTES ____________________

COUNTY ____________________ LOCATION ____________________

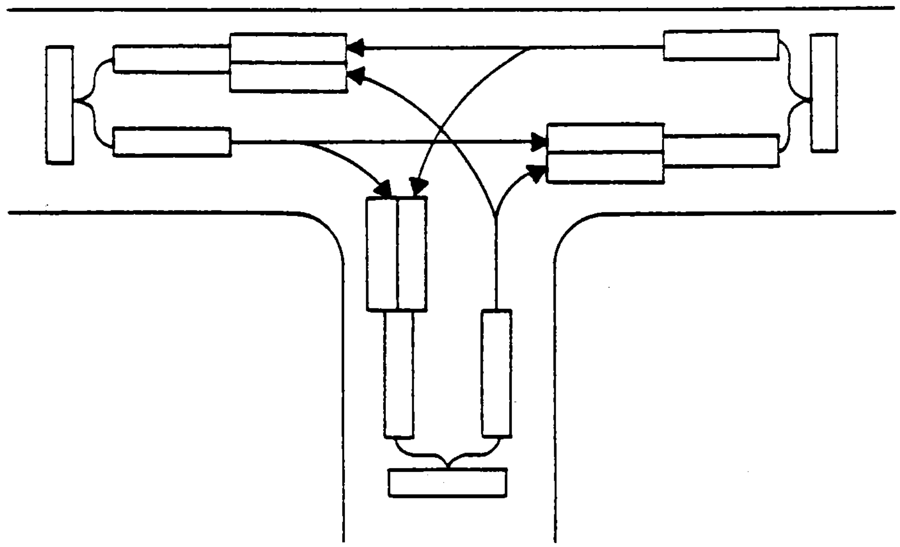
DATE ____ / ____ / ____ WEATHER ____________________

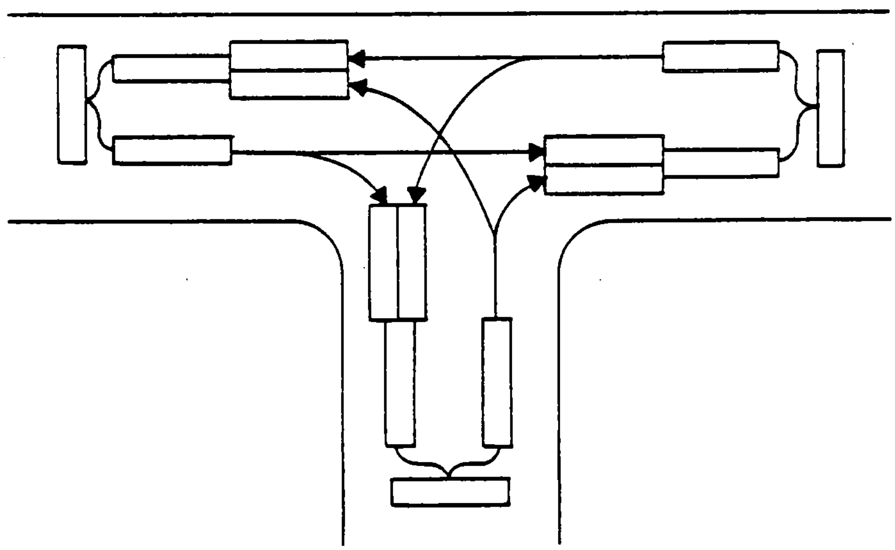
	APPROACHING INTERSECTION																
	ON ROUTE							FROM THE	ON ROUTE							FROM THE	
	EAST				WEST				NORTH				SOUTH				
HOURS	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	TOTAL
6:00 - 7:00																	
7:00 - 7:30																	
7:30 - 8:00																	
8:00 - 8:30																	
8:30 - 9:00																	
9:00 - 10:00																	
10:00 - 11:00																	
11:00 - 12:00																	
12:00 - 1:00																	
1:00 - 2:00																	
2:00 - 3:00																	
3:00 - 4:00																	
4:00 - 4:30																	
4:30 - 5:00																	
5:00 - 5:30																	
5:30 - 6:00																	
12 - HOUR TOTAL																	
24 - HOUR TOTAL																	

RECORDED BY ____________ ____________

SUPERVISOR ____________________

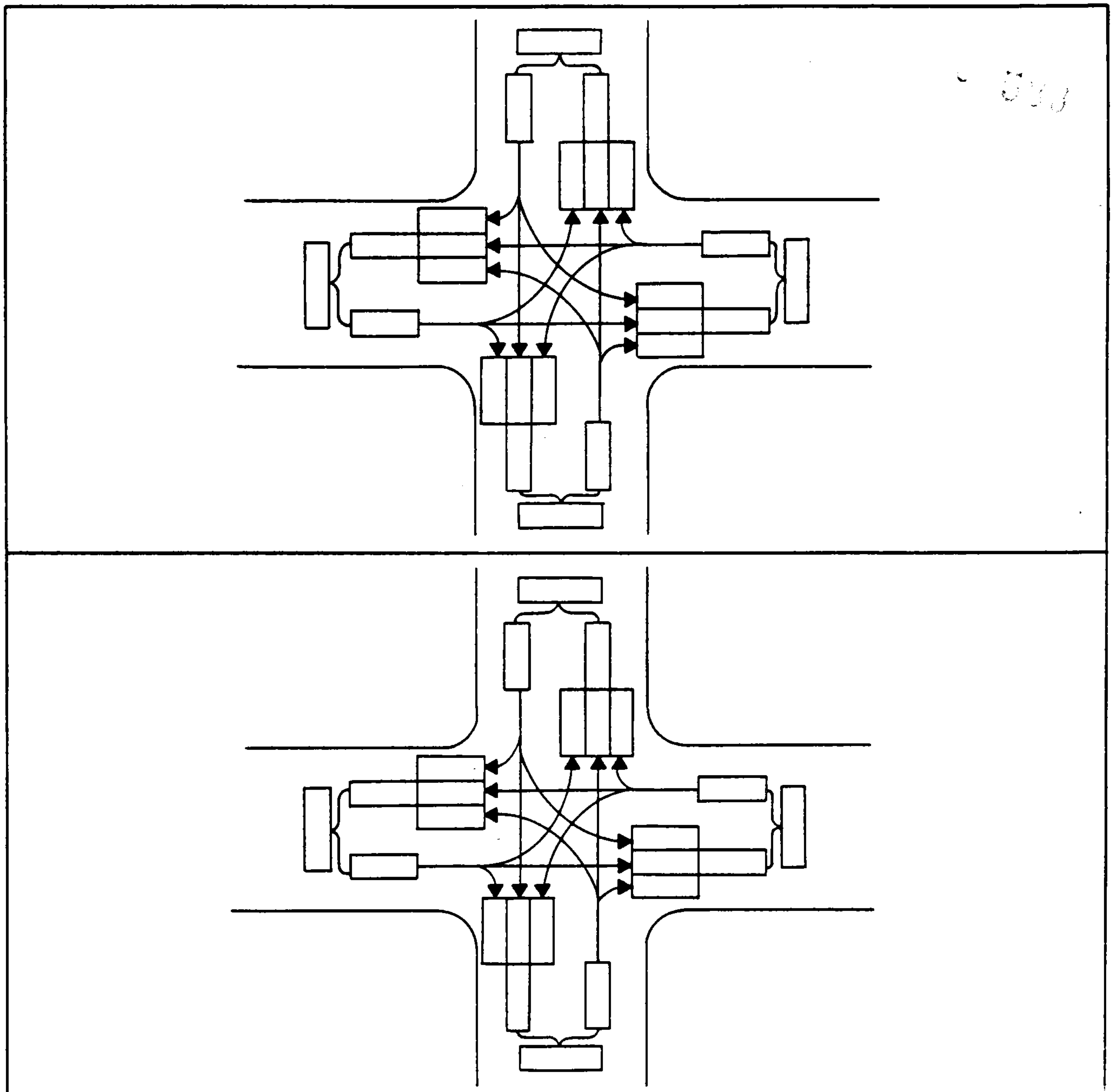
Figure 2. Typical data collection form.





EIGHT MAXIMUM HOUR VOLUMES OF APPROACH VEHICLES									
TIME		RTE.		RTE.		RTE.		TOTAL	
FROM	TO	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.
TOTAL									

Figure 3. Typical data summary form for three-legged intersection.



EIGHT MAXIMUM HOUR VOLUMES OF APPROACH VEHICLES											
TIME		RTE.		RTE.		RTE.		RTE.		TOTAL	
FROM	TO	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.
TOTAL											

Figure 4. Typical data summary form for four-legged intersection.

Determine Number of Phases

As a general rule, the number of phases should be kept to a minimum. Cycle lengths that are long result in delays to individual groups of vehicles awaiting the green indication; therefore, there are practical limits to cycle lengths in order to avoid these intolerable delays. Accordingly, additional phases tend to decrease the available green time for other phases since they must be accommodated within the practical maximum cycle length. Also, there is additional lost time through start-up delays and phase change or clearance intervals over the course of a cycle as the number of phases increases.

The number of phases required at an intersection is most often a left-turn issue. As the volumes of left-turn and opposing traffic increase, it becomes more difficult for the traffic turning left to find adequate gaps. A separate left-turn lane can alleviate the problem to some degree by providing storage for vehicles awaiting an adequate gap; however, at a certain point a separate phase for movement from that left-turn lane is needed. The following guidelines applicable to intersections having separate left-turn lanes may be used when considering the addition of separate left-turn phases. These are contained in a recent report entitled Guidelines for Exclusive/Permissive Left-Turn Signal Phasing, by B. H. Cottrell, Jr.(1)

1. Volumes - consider left-turn phasing on an approach when the product of the left-turn volume and opposing volume divided by the number of lanes during the peak hour exceeds 50,000, provided that the left-turn volume is greater than two vehicles per cycle on average.

2. Delay - consider left-turn phasing if a left-turn delay of 2.0 vehicle-hours or more occurs in the peak hour, provided that the left-turn volume is greater than two vehicles per cycle on average. Also, the average delay per left-turning vehicle must be at least 35 sec. See Appendix B for a procedure for determining intersection delay.

3. Accident experience - consider left-turn phasing if the critical number and resulting rate of left-turn accidents have been exceeded. For one approach the critical number is five left-turn accidents in one year. The accident rate, as defined by the annual number of left-turn accidents per 100 million left-turn plus opposing vehicles, must exceed the critical rate determined by the equation

   $Rc = 32.6 + 1.645\sqrt{32.6/M} - 0.5\,M$, where M is the annual left-turn plus opposing volume in 100 million vehicles.

4. Site conditions -- consider left-turn phasing if there is inadequate sight distance, if there are three or more lanes of opposing through traffic, if intersection geometrics promote hazardous conditions, or if there are access management problems.

It is emphasized that the above are guidelines and should be coupled with engineering judgement. More detailed information on these guidelines plus guidelines for using protected/permissive phasing can be found in the above referenced report.

Determine Values for the Timing Parameters

In recent years, traffic control equipment has become reasonably standardized by the National Electrical Manufacturers Association (NEMA). Thus, the models of equipment manufactured in recent years have basically the same dials and settings, and employ the same terminology. Accordingly, the following discussion on timing parameters will focus on the NEMA controllers; however, information on pre-NEMA equipment will also be presented where possible. It is very important to be familiar with the timing functions of the equipment being retimed or being considered in the case of a new installation.

The phase timing for a NEMA traffic-actuated controller is shown in Figure 11, and a typical phase timing for older equipment is shown in Figure 12. Both of these figures are referenced in the following discussion of timing parameters.

The following general rules concerning the timing of traffic-actuated controllers are often cited. These are contained in the International Municipal Signal Association's Traffic Signal Manual.(4)

1. Make all timing adjustments during heavy traffic. The controller will then automatically take care of the light traffic efficiently.

2. Set the dials at values considered correct after evaluating detector spacing, relative volumes, and desired results. Then, adjust or "tune" the controller to accommodate the heaviest traffic.

3. After the dials have been set, take steps to ensure that unauthorized persons cannot change them.

4. The tendency is to set the times too high. In general, lower settings produce snappier, more efficient intersection operation.

Timing parameters must be established for each phase.

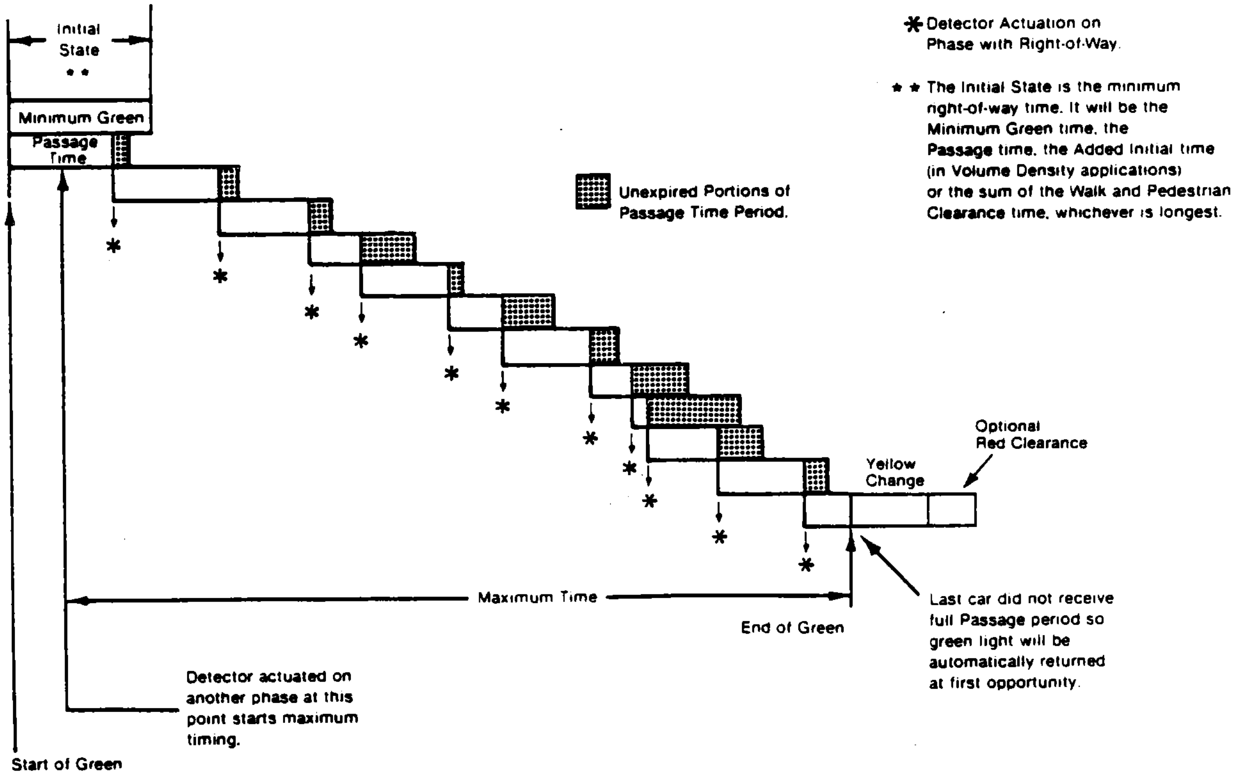


Figure 11. Typical timing diagram for a full-actuated NEMA controller. Source: Reference 5.

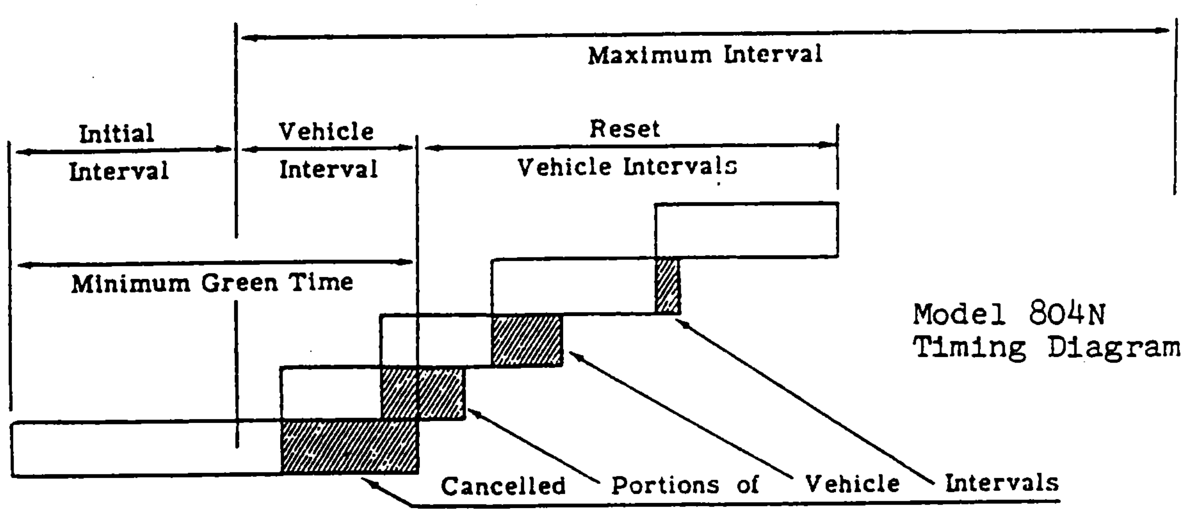


Figure 12. Typical timing diagram for a full-actuated non-NEMA controller. Source: Reference 2.

Passage Time

For the typical, low-speed intersection, the passage time, which has also been referred to as the vehicle interval or unit extension interval, is defined as the time needed for a vehicle moving at average speed to travel from the detector to and through the intersection. As shown in both Figures 11 and 12, this interval begins to retime itself when a vehicle actuates the detector. Retiming continues upon each actuation until such time as the maximum time setting is reached; then, the controller switches to another phase that has been called. This operation is referred to as "force off" or "max out." If a vehicle does not receive a full passage time interval because of being forced off or an actuation is received during the yellow change interval, the controller assumes the vehicle did not clear the intersection. Thus, the green is automatically returned to that phase at the earliest opportunity. On the other hand, if a gap in traffic is large enough such that the passage time is reached before being re-actuated, then the phase ends and the controller switches to another phase that has been called. This is often called a "gap out." Accordingly, the passage time has two functions -- it allows vehicles to travel from the detector through the intersection, and it establishes the gap, measured in seconds, at which green is terminated.

At intersections where speeds are 35 mi/h or less, and where small area or point detectors are used, the needed passage time is calculated by dividing the distance between the intersection and the detector by the average approach speed of traffic. However, the gap required to retain the green must also be considered. Bumper-to-bumper traffic produces gaps of from 2.0 to 3.0 seconds, with the former being indicative of fast-paced, urban areas and the latter being more likely in rural areas. Passage times set in this range make the operation too sensitive to gaps. On the other hand, gaps greater than about 5.0 seconds cater to the "stragglers" in the traffic stream and cause a reduction in efficiency. Detectors are typically located such that the passage time falls in this range. Therefore, the passage time interval at intersections where speeds are 35 mi/h or less and point detectors are used should be set to the nearest tenth of a second as calculated by dividing the detector spacing by the average approach speed; however, it should be no less than 3.0 seconds and no more than 5.0 seconds. Table 4 summarizes these rules.

Table 4

Passage Times for Various Point Detector Spacings and Speeds

Average Approach Speed (mi/h)	Distance Between Stop Bar and Detector (d) (ft)	Passage Time Interval (sec)
15	0-67	3.0
	68-108	d/22.0
	more than 108	5.0
20	0-89	3.0
	90-145	d/29.3
	more than 145	5.0
25	0-111	3.0
	112-181	d/36.7
	more than 181	5.0
30	0-134	3.0
	135-217	d/44.0
	more than 217	5.0
35	0-156	3.0
	157-253	d/51.3
	more than 253	5.0
40	0-179	3.0
	180-290	d/58.7
	more than 290	5.0
45	0-201	3.0
	202-326	d/66.0
	more than 326	5.0

In slow-paced, rural areas the passage time may have to be set higher than the recommended minimum of 3.0 seconds. Also, if the detectors are placed such that the needed passage time is greater than 5.0 seconds, it may be necessary to set the passage time interval above the recommended maximum for safety purposes. Doing this will decrease the efficiency of the intersection.

If large area or presence detectors are used at the stop bar, very little time is needed for a vehicle to clear the intersection. For example, a typical vehicle would be about 20 feet into the intersection when its presence would no longer be detected, and would need only enough passage time to traverse the remaining width of the intersection. Also, large area detectors have a "built-in" gap because a vehicle is detected for a finite period of time as it traverses the detector. This gap can be calculated by adding the length of the loop and an assumed 20 feet for the length of the vehicle and dividing by the average speed of vehicles on the approach. The built-in gaps for various lengths of detectors and approach speeds are given in Table 5. Accordingly, when large area detectors are used at intersections having speeds of 35 mi/h or less, the following procedures should be undertaken to determine the passage time interval.

1. Select the gap required to retain the green, which, as discussed above, should be between 3.0 and 5.0 sec. As a general guideline, a gap of from 3.0 to 4.0 sec is good for fast-paced, urban areas or where snappy operation is desired, and a gap of from 4.0 to 5.0 sec is good for slow-paced, rural areas.

2. From Table 5 determine the built-in gap for the size of the detector used and the average speed.

3. Calculate the setting for the passage time interval by subtracting the built-in gap from the gap required to retain the green selected in step 1. This setting is usually between 1.5 and 3.0 sec in most applications. In the case of very long detectors and slow speeds, the detector's built-in gap may be the same or even exceed the gap selected to retain the green; therefore, the setting on the passage time dial conceivably could be zero.

The passage time at high-speed intersections, defined generally as those having speeds greater than 35 mi/h, is treated somewhat differently. Since most high-speed intersections are controlled by the Virginia Department of Highways and Transportation, the two most common types of operations are described in this report. The first treatment considers the fact that at high speeds the decision to stop or continue at the onset of a yellow clearance interval becomes a much more critical issue than at slow speeds. The area in which this indecision occurs is called the dilemma zone. Motorists caught outside the dilemma zone, or away from the intersection, generally reach the decision to stop when a yellow indication is observed. Motorists inside the dilemma zone, or toward the intersection, generally decide to proceed through the intersection. The boundaries of the dilemma zone are given in Table 6.

Table 5

Built-In Gaps for Large Area Detectors
(Seconds)

Length of Detector (ft)	Average Approach Speed (mi/h) 15	20	25	30	35	40	45
20	1.8	1.4	1.1	0.9	0.8	0.7	0.6
30	2.3	1.7	1.4	1.1	1.0	0.9	0.8
40	2.7	2.0	1.6	1.4	1.2	1.0	0.9
50	3.2	2.4	1.9	1.6	1.4	1.2	1.1
60	3.6	2.7	2.2	1.8	1.6	1.4	1.2
70	4.1	3.1	2.5	2.0	1.8	1.5	1.4
80	4.5	3.4	2.7	2.3	1.9	1.7	1.5
90	5.0	3.8	3.0	2.5	2.1	1.9	1.7
100	5.5	4.1	3.3	2.7	2.3	2.0	1.8
110	5.9	4.4	3.5	3.0	2.5	2.2	2.0
120	6.4	4.8	3.8	3.2	2.7	2.4	2.1

Based on the formula [length of detector (ft) + 20]/speed (ft/s) and 1 mi/h = 1.47 ft/s.

Table 6

Dilemma Zone Boundaries

Approach Speed (mi/h)	Distance from Intersection (ft) for Probabilities of Stopping 10%	90%
35	102	254
40	122	284
45	152	327
50	172	353
55	234	386

Source: Reference 6.

Two small area or point detectors are used on each approach lane. The first is placed a distance corresponding to a travel time of from 2.0 to 4.0 seconds at the average speed outside an imaginary line located within the dilemma zone at which 90% of the motorists will decide to proceed through the intersection. This is the 10% probability of stopping distance in Table 6. The second detector is placed the same 2.0 to 4.0 seconds of travel time beyond the first detector. This configuration is shown in Figure 13. The passage time is then set at the same 2.0 to 4.0 seconds, and actuation of either detector will cause the interval to retime itself. A single vehicle traveling at the average speed will be ensured consecutive 2.0- to 4.0-second intervals which will place the vehicle at the aforementioned 90% line when a yellow clearance signal is received. Likewise, a single vehicle traveling above or below the average speed will find itself inside or outside the dilemma zone, respectively.

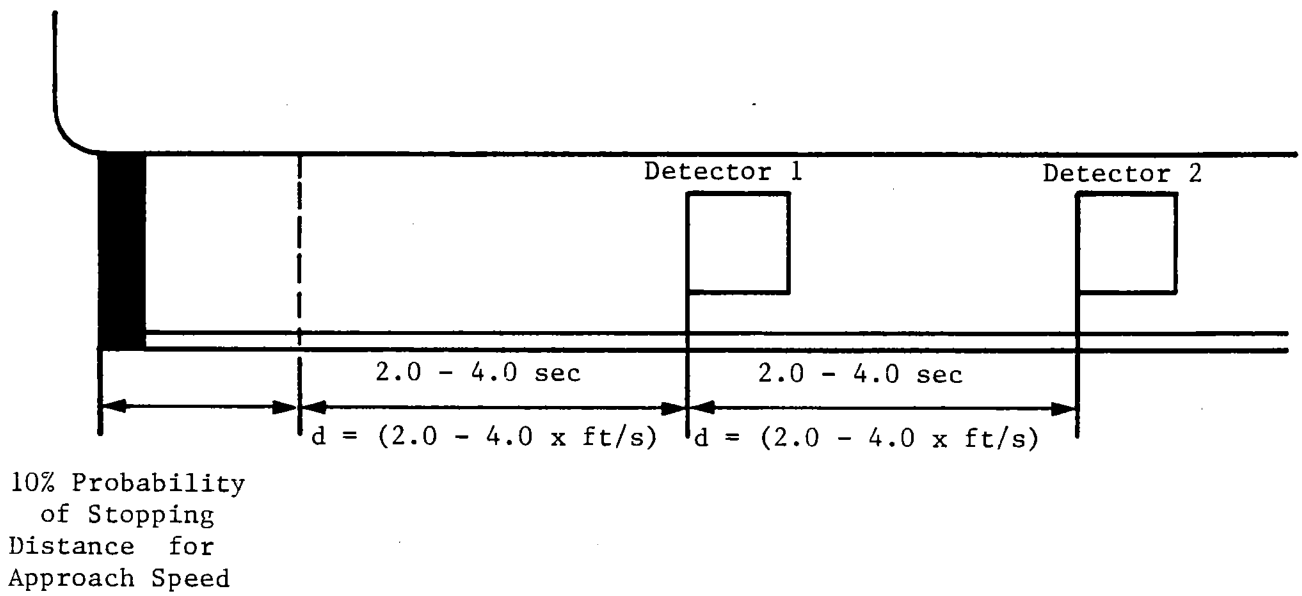


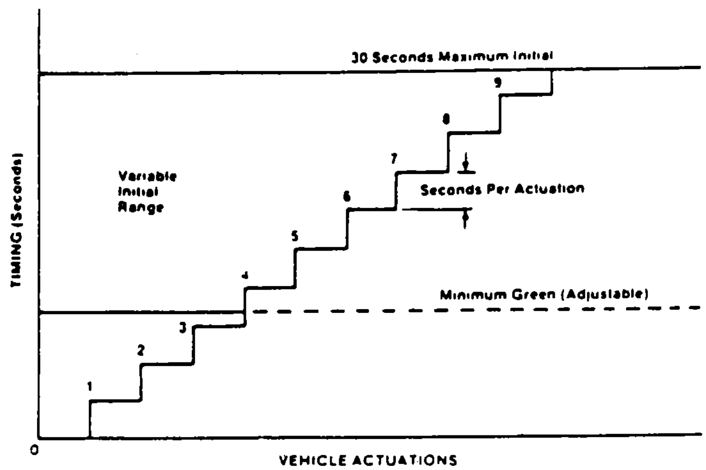
Figure 13. High-speed approach detection.

An advantage of dual detection is that there are more chances of extending the green for that phase. Since the phases having dual detection receive favored treatment, this type of design is most often used when the main line volumes and speeds are high as compared to those of the side street.

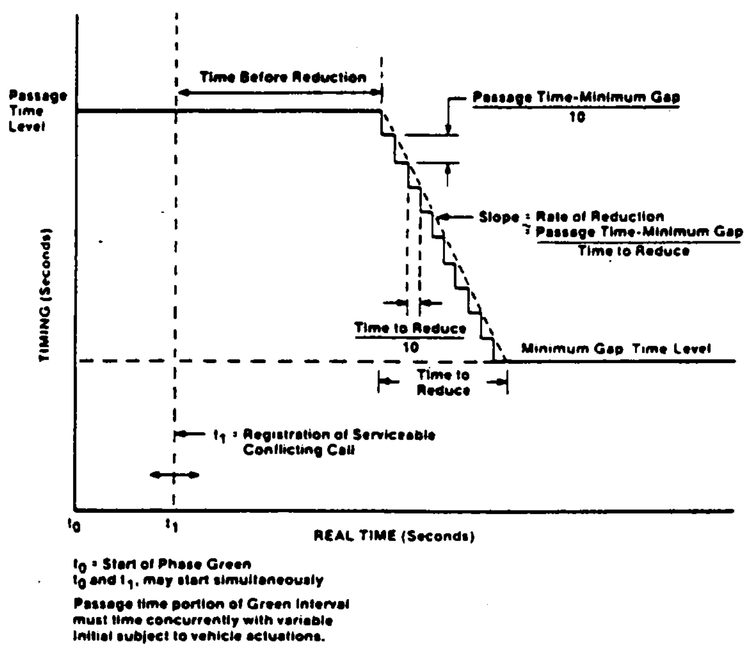
In summary, if this design is encountered in a retiming situation, the passage time should be set at 2.0 to 4.0 seconds. The original design must be reviewed to determine the proper setting. Another method of obtaining the setting is to divide the distance between the detectors by the average approach speed. If there is a tendency for vehicles to slow down after crossing the second detector, for example in the case of right-turning vehicles, the passage time may be set up to 1.0 second above the design time so that there is less chance of motorists being caught in the dilemma zone.

If traffic volumes at a high-speed intersection are approximately the same on all approaches, a volume-density operation is often employed. This operation was described earlier. The NEMA controllers have four or five timing parameters for volume-density operation, three of which involve the passage time. These are the time before reduction, the time to reduce, and the minimum gap. In operation, the passage time of from 3.0 to 5.0 seconds, determined as described previously for low-speed intersections with point detectors, controls the gap needed to retain the green during the "time before reduction" period. Once this latter period times out, the passage time incrementally decreases during the "time to reduce" period until the "minimum gap" is reached. This gap then controls the phase until gap out or force off is attained. The lower portion of Figure 14 shows gap reduction in a schematic form. Note that gap reduction is initiated when a call is received on a conflicting phase.

Since the gap reduction feature is intended to increase the controller's response to traffic demand, it is necessary that these settings be established based on field observations during the period of heaviest demand. Initial settings which should be adjusted on-site are generally made based on logical considerations rather than a specific methodology. The main purpose of gap reduction is to make the controller increasingly sensitive to the traffic flow on the phase being serviced in recognition of the waiting time a motorist on a conflicting phase is experiencing.



EXPLANATORY DIAGRAM
VARIABLE INITIAL



EXPLANATORY DIAGRAM
GAP REDUCTION

Figure 14. Volume-density timing functions (NEMA).
Source: Reference 2.

The controller is most sensitive to traffic flow when operating with the minimum gap, and should maintain green only during bumper-to-bumper traffic. Thus, a minimum gap setting of from 2.0 to 3.0 seconds will usually be effective. If the phase being considered is relatively minor, then the time to reduce can be rather short, e.g., 15 to 20 seconds. On the other hand, the time to reduce on a major phase may be set at 30 seconds or more. The time before reduction is useful in delaying gap reduction until slow-moving traffic such as large trucks can get in motion. Another general rule of thumb is that the minimum gap should be reached by the time the phase is at 80% of its maximum green time. The split between time before reduction and time to reduce can be determined according to the general logic just discussed.

Minimum Green -- Actuated Phase

The minimum green interval for an actuated phase is set to allow vehicles stopped between the detector and the intersection to get started and move into the intersection. If small area or point detectors are being used, there is a finite distance between the detector and the intersection in which vehicles can be stored while awaiting a green indication. When the green is received, a minimum amount of green time is needed to ensure that these stored vehicles can start up and move into the intersection in case there are no further actuations to initiate timing of the passage time interval. Studies have indicated that for a single line of vehicles, the time in seconds needed to accomplish this can be estimated by the formula $2.1\ n + 3.7$, where n is the number of vehicles. Assuming the average length of a vehicle is 20 feet, then the number of vehicles can be estimated as the distance between the intersection and the detector divided by 20 feet.

The minimum green timing parameter is shown schematically in Figures 11 and 12. It is important to note from Figure 12 that for pre-NEMA controllers, the minimum green is the sum of a setting called initial interval plus the vehicle (passage time) interval. In this case, the minimum green is calculated as described above, and the initial interval is set by subtracting the vehicle interval.

In summary, the procedure for determining minimum green when point detection is utilized is as follows.

1. Determine the maximum number of vehicles, n, that can be stored in a single lane between the stop bar and the point detector by dividing the distance in feet between the two by 20 and rounding up.

2. Determine the minimum green setting by applying the formula

2.1 n + 3.7. Application of this formula for various detector spacings is given in Table 7.

3. For pre-NEMA controllers, the initial interval should be set as the difference between the above minimum green and the passage time interval.

Table 7

Minimum Green Time versus Point Detector Spacing

Distance Between Stop Bar and Detector (ft)	Minimum Green (sec)
0-40	7.9
41-60	10.0
61-80	12.1
81-100	14.2
101-120	16.3
121-140	18.4

Source: Reference 3.

If detectors are located at the stop bar, there is no finite distance in which a vehicle can be stored without being detected. Accordingly, the minimum green conceivably could be set at zero. There are practical considerations, most related to motorists' expectations, however, which require that a minimum green time of between 4.0 and 7.0 seconds be set.

For high-speed intersections, the point detectors are typically located a considerable distance from the intersection to allow a passage time interval setting between 3.0 and 5.0 seconds. A minimum green based on the assumed storage of vehicles in that distance results in excess green being given to that phase when traffic is light and the storage space is not fully utilized. In this case, a volume-density controller is frequently used. In order to utilize the volume-density function, an additional timing parameter called "seconds per actuation" needs to be set. Some NEMA controllers also have a parameter called "maximum added initial". In operation, a minimum green time based on one vehicle is set, that is, 2.1 seconds plus 3.7 seconds, or 5.8 seconds. Each additional vehicle approaching the intersection during the nongreen time will actuate the detector and increase the minimum green by the preset seconds per actuation until such time as the maximum added initial green time is reached. This operation is depicted schematically in the upper portion of Figure 14. In theory, the seconds per actuation should be set at 2.1 seconds and the maximum added initial should be set to accommodate the maximum storage in a single lane, as given by the previously described formula, 2.1 n + 3.7. This procedure is satisfactory for a single-lane approach; however, in the case of a multi-lane approach, the seconds per actuation should be set at 1.0 second.

Again, volume-density settings should be based on field observations. Further, pre-NEMA controllers may have slightly different ways to account for additional vehicles approaching the intersection; therefore, it is most important to review the instructions for the specific controller being retimed.

Minimum Green -- Non-actuated Phase

The minimum green interval for a non-actuated phase is applicable to the major street at an intersection under semi-actuated control on which there are no detectors to call or advise the controller of the presence of vehicles. The major street is guaranteed a minimum green even if calls for service are incoming from the side street or from pedestrians. If the side street demand is occasional and occurs randomly throughout the day, then a relatively short setting of from 25 to 40 seconds should be used to prevent excessive delay on the side street. On the other hand, in a situation where the side street discharges large numbers of vehicles at certain times during the day, e.g. from a factory, with almost no demand at other times, a relatively long setting of from 40 to 60 seconds is appropriate. This ensures that the major street is not interrupted too frequently during the period of heavy side street demand.

Minimum green should be calculated based on the relative traffic volumes at the intersection. To accomplish this, it is assumed that pretimed control exists, and minimum green on the major street, or for the non-actuated phase, is compared with the maximum green for the side street, or for the actuated phase. Then, the procedures for timing pretimed controllers presented elsewhere in this report should be used. The green time calculated for the non-actuated phase should be set on the dials for both minimum green and maximum green on that phase. Other settings are determined based on the actuated signal procedures.

Maximum Green

The maximum green interval determines the longest time that continuously moving traffic can hold the green signal once a call has been received on a conflicting phase. In operation, when gaps in the traffic flow are sufficiently small to cause the passage time interval to continuously retime itself, the green is forced off because the maximum value is reached rather than because the gap required to retain the green is exceeded. If the passage time has been set correctly, the force-off condition is attained only during times of heaviest traffic flow at the intersection. As mentioned previously, the controller is operating essentially in a pretimed fashion during heavy flow conditions because all phases are being forced off at the preset maximum times. Accordingly, the maximum green per phase for actuated control should be determined in the same manner as the green time per phase is determined for pretimed control. The reader is referred to the section of the report on pretimed control.

Figures 11 and 12 depict schematically how the maximum green is timed. As shown in Figure 11, the maximum green for NEMA controllers begins timing upon receipt of a call on a conflicting phase. As shown in Figure 12, the maximum interval for older controllers begins after the initial interval has timed out. While it is not critical to take these differences into account when setting the maximum green, it is important to be aware of how the controller is timing when field checks are being made.

Finally, it should be noted that some controllers are capable of providing two maximum green intervals per phase. A time clock or other external control selects the interval to be used. The timing would be determined from two sets of volumes and pedestrian counts. An example would be the use of a longer maximum green during peak hours than during the remainder of the day.

Yellow Change and Red Clearance

The purpose of the phase change or clearance interval, which consists of the yellow interval and, possibly, an all-red interval, is to advise motorists of an impending change in the assignment of right-of-way; that is, the commencement of a red interval on their approach. Upon commencement of the change interval, a motorist should have sufficient time to either stop his vehicle or clear the intersection. At a given approach speed, a certain amount of time is needed to decelerate to a safe stop at the intersection or proceed through the intersection prior to commencement of the green interval on the cross street. The previously presented equation 5 is used to calculate the phase change interval.

$$CP = t + \frac{V}{2a \pm 64.4g} + \frac{W+L}{V}, \tag{5}$$

where

CP = change period in seconds,

t = perception/reaction time, usually 1 second,

V = approach speed in feet/second, typically the 85th percentile speed or prevailing speed limit,

a = deceleration rate in feet/second^2, usually 10 feet/second^2,

W = width of intersection in feet,

L = length of vehicle in feet, usually 20 feet, and

g = percent of grade divided by 100, with upgrade being positive and downgrade being negative.

It is important that motorists have a reasonable expectation of the length of the yellow interval; therefore, the yellow interval should be set in the range of from 3.0 to 5.0 seconds. Within these limits, the yellow interval is often set according to the time it takes to decelerate to a stop; that is, the first two terms in the above equation. Yellow intervals that are longer than necessary decrease capacity and encourage motorists to try to "beat the light."

The time needed to clear the intersection as calculated by the last term in the above equation should be included in an all-red interval where all approaches receive a red indication. Required stopping time above 5.0 seconds should also be included in the all-red interval.

Exclusive left-turn phases do not typically have an all-red interval. Normally, a through-movement phase follows the exclusive turn movement; therefore, motorists receiving the green directly face straggling left-turners and can safely yield the right-of-way. An all-red interval may be needed, however, at a high-speed intersection or at an intersection with a wide median.

Equation 5, minus the grade factor, coupled with the aforementioned rules regarding the phase change interval were used to develop the information in Table 3. This table has been reproduced in this section for the convenience of the reader. For a given approach speed, the yellow change interval plus the total phase change interval for various intersection widths are presented. The all-red interval is the difference between the two given intervals.

Table 3

Phase Change Intervals

Approach Speed (mi/h)	Yellow Change Interval (sec)	Total Clearance Interval (Yellow Plus All-Red Clearance) for Crossing Street Widths (ft) 30	50	70	90	110
20	3.0	4.2	4.9	5.5	6.2	6.9
25	3.0	4.2	4.7	5.3	5.8	6.4
30	3.2	4.3	4.8	5.2	5.7	6.2
35	3.6	4.5	4.9	5.3	5.7	6.1
40	3.9	4.8	5.1	5.5	5.8	6.1
45	4.3	5.1	5.4	5.7	6.0	6.3
50	4.7	5.3	5.6	5.9	6.2	6.4
55	5.0	5.7	5.9	6.2	6.4	6.7

Source: Reference 3

Walk and Pedestrian Clearance

Pedestrian movements at a signalized intersection are typically accommodated by one of the following methods:

1. Pedestrians cross the street with the parallel vehicular green indication with no pedestrian signals.

2. Pedestrians cross the street with the parallel vehicular green indication as instructed by special pedestrian signals.

3. Pedestrians cross the street on an exclusive phase when all vehicular traffic is stopped.

For any of the above methods, sufficient time must be provided for pedestrians to enter the intersection, called the walk interval, and to safely cross the street, called the pedestrian clearance interval. In the first two cases above, this time occurs while the parallel vehicular traffic, or traffic on the street <u>not</u> being crossed, is receiving a green and clearance interval. Therefore, the sum of the green and clearance interval for an approach should be long enough to accommodate any pedestrian flow on the cross street.

In many cases the combination of pedestrian and vehicular volumes may not create enough conflicts to warrant concern about the minimum time needed by pedestrians. At locations where there are significant pedestrian volumes or pedestrians require special attention, such as near elderly housing, it is necessary to provide pedestrian actuation and set the pedestrian intervals to ensure that the minimum crossing time is received. Upon receipt of a pedestrian call, the walk and pedestrian clearance intervals for the phase controlling traffic on the street not being crossed will begin timing and extend the green to the set values if necessary. In other words, if necessary, the minimum green is reestablished based on a pedestrian demand. This separate timing may or may not be in conjunction with separate pedestrian signals.

The walk interval, or time needed by a pedestrian to perceive the signal change and move into the intersection, is generally assumed to be from 4.0 to 7.0 seconds. The higher values are used when pedestrian volumes are high. The pedestrian clearance interval is dependent upon the width of the street being crossed and the walking speed of the pedestrian, which is generally assumed to be from 3.5 to 4.0 feet/second. The slower speeds are used when pedestrian volumes are high or in special cases such as in the vicinity of elderly housing. Thus, the pedestrian clearance interval can be determined by dividing the width in feet of the street being crossed by the assumed walking speed in feet/second. Field measurement of walking speeds at the intersection would provide the best data. The actual setting on the dial can be reduced by the yellow change and red clearance intervals as these intervals must time out before a conflict occurs. It is important to remember that the intervals are set for the phase controlling traffic on the street not being crossed.

Verify or Adjust Timing

The signal timing developed by the preceding procedures should be considered only as a starting point. The procedures are based on typical traffic performance, and factors at the intersection being timed may negate or modify some of the theory or assumptions used. Therefore, it is very important to observe the intersection in operation under the calculated timing in order to either verify the settings or adjust them if necessary.

TIMING FOR SIGNAL SYSTEMS

Background

A signal system consists of two or more signalized intersections operated in coordination; that is, that have a fixed time relationship to each other. This relationship is based on the fact that vehicles at a signal are released in platoons, or groups, upon receipt of a green indication and then travel in these platoons to the next signal. Thus, it becomes desirable to establish a fixed time relationship between the beginning of the green interval at the first intersection and the beginning of the green interval at the second intersection such that the platoon receives the green interval just as it arrives at the second intersection. This permits the continuous or progressive flow of traffic along the street. When the coordinated intersections are located along a single route, the term "arterial system" is applied. When two or more routes cross at a common intersection, the result is a "signal network". An open network has only one common intersection, whereas a closed network has two or more common intersections. This latter network is often referred to as a "grid system," and is commonly found in the centers of large cities. Determination of the fixed time relationship becomes increasingly difficult as the number of intersections in an arterial system and the number of common intersections in a network increase.

The effectiveness of coordinated control depends on whether traffic can be kept in platoons between intersections. The ability to maintain platoons depends on traffic characteristics, topography, condition of the roadway, and roadside friction. As the distance between intersections increases, the effects of these factors become more pronounced, and the probability of platoon dispersal increases. The Manual on Uniform Traffic Control Devices states that, "Traffic control signals within 0.5 mile of one another along a major route or in a network of interconnecting major routes should be operated in coordination...." Although this suggests a maximum spacing between signalized intersections of 0.5 mile for effective coordination, there are many examples of effective coordination where signals are spaced up to a mile apart, particularly where roadside friction is minimal, speeds are high, and signals are visible for some distance in advance of the intersection. Generally, it is best to attempt to coordinate intersections if at all possible to maintain traffic flow in platoons.

The following advantages of providing coordination among signals are listed in the Transportation and Traffic Engineering Handbook.(7)

1. A higher level of traffic service is provided in terms of higher overall speed and reduced number of stops.

2. Traffic should flow more smoothly, often with an improvement in capacity and decrease in energy consumption.

3. Vehicle speeds should be more uniform because there will be no incentive to travel at excessively high speed to reach a signalized intersection before the start of the green interval, yet slower drivers will be encouraged to speed up to avoid having to stop for a red light.

4. There should be fewer accidents because platoons of vehicles will arrive at each signal when it is green, thereby reducing the possibility of red-signal violations or rear-end collisions. Naturally, if there are fewer red intervals displayed to the majority of motorists, there is less likely to be trouble because of driver inattention, brake failure, slippery pavement, and so on.

5. Greater obedience to the signal commands should be obtained from both motorists and pedestrians because the motorist will try to keep within the green interval, and the pedestrian will stay at the curb because the vehicles will be more closely spaced.

6. Through traffic will tend to stay on the arterial street instead of on parallel minor streets.

Definitions

The following definitions are applicable to timing signal systems. Figure 1 depicted some of these, and it has been reproduced in this section for the convenience of the reader.

1. Timing plan - a unique combination of cycle length, split, and offsets.

2. Cycle - the time required for one complete sequence of signal indications. The term "background cycle" is often used to identify the common cycle length established for all intersections in a system.

3. Phase - that part of a signal cycle allocated to any combination of one or more traffic movements simultaneously receiving the right-of-way during one or more intervals.

4. Interval - a discrete portion of the signal cycle during which the signal indications remain unchanged.

5. Split - the percentage of a cycle length allocated to each of the phases.

6. Detector - a device which detects the passage or presence of a vehicle with the purpose of advising a controller of the need for a green indication. For purposes of this project, detectors will be categorized as either small area detectors or large area detectors. Small area detectors provide passage, point, motion, or unit detection. It is noted that a 6 x 6-foot loop is often used as a

point detector. These detectors simply register the passage of a vehicle. Large area detectors provide presence or area detection. These detectors register the presence of a vehicle in the zone of detection.

Sampling detectors are placed upstream of the intersection to count the vehicles and provide volume data to the controller or computer which is operating the system.

7. Offset - the time difference in seconds or percentage of cycle length between the start of the green interval at one intersection and the start of the green indication at another intersection, or from another system reference point. See Figure 15.

8. Yield point - associated with actuated controllers, a reference point in the cycle where the controller "yields" the right-of-way to an opposing phase. It marks the end of the non-actuated phase on the major street and establishes the background cycle for coordination.

9. Time-space diagram - a graphical representation of a signal system showing cycles, splits, offsets, and distance relationships of the intersections. It is also used to manually determine offsets and the progressive flow characteristics. See Figure 15.

10. Progression or band speed - the speed which a platoon needs to travel in order to progress or continue from intersection to intersection in the system without being stopped. It is the slope of the band lines in Figure 15.

11. Band, band width, or through band - the amount of time in seconds between the first and last vehicles traveling at the band speed which can progress through the system without stopping. The efficiency of the timing plan is often measured by the band width as a percentage of the cycle length. See Figure 15.

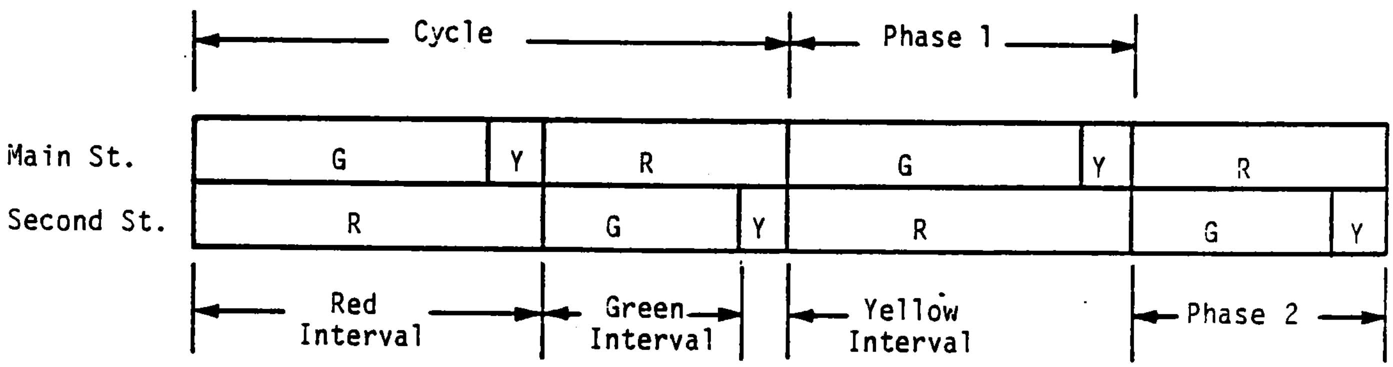


Figure 1. Timing sequence for simple two-phase controller.

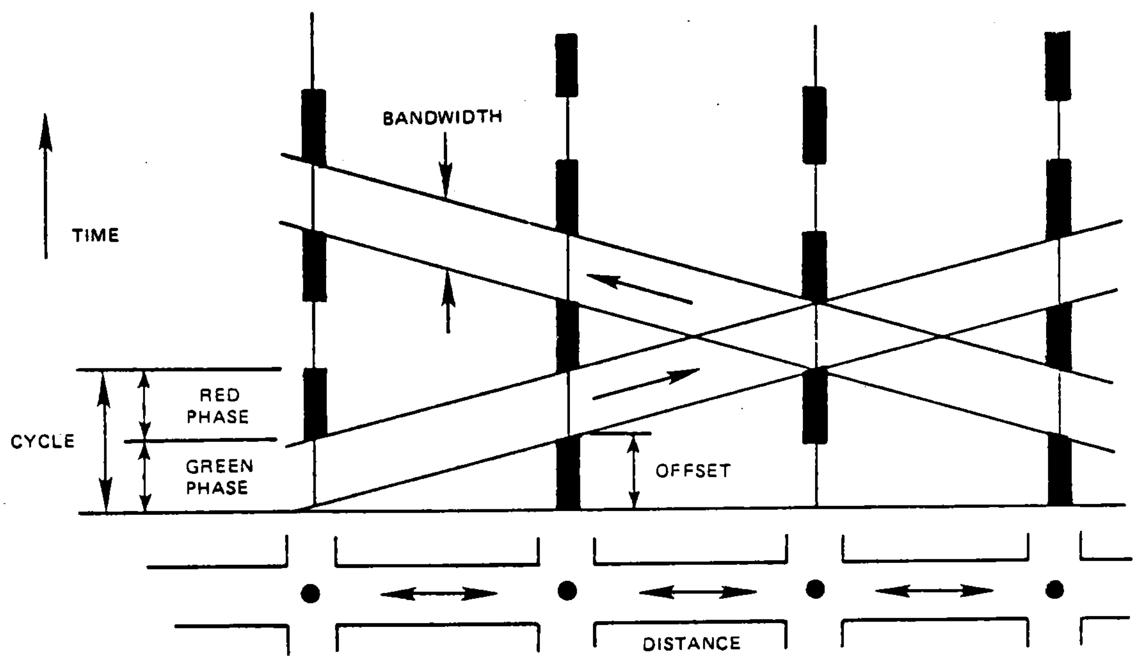


Figure 15. Time-space diagram.
Source: Reference 8.

Types of Systems

In addition to the broad categories of arterial and grid systems described earlier, the Traffic Control Devices Handbook(6) categorizes systems according to the type of hardware components. These categories are described in the following subsections.

Noninterconnected System

In a noninterconnected system the controllers or coordinating units are synchronized through the 60-hertz cycle of the area's power supply. This type of system is usually limited to a simple timing plan, and as such has no flexibility in adjusting to traffic conditions. The principal disadvantage, however, is the inability to hold the offset relationship due to fluctuations in the power supply. Whenever a controller is "out-of-step," it must be reset manually in the field. Because of this problem, systems of this type are not considered effective and are rarely seen in practice.

Time-based Coordinated System

The time-based coordinated system is relatively new and also noninterconnected. Synchronization is maintained through extremely accurate digital timing and control devices called time-based coordinators at every intersection. New controllers may have this function built in. Time-based coordinators can be programmed with a time-of-day and day-of-week schedule for implementing timing plans. This allows some flexibility to adjust to traffic patterns and demand conditions; however, each timing plan and schedule must be set manually at every intersection in the system. The main advantage is the potential savings in cost of not having to physically interconnect the controllers. Also, if one of the coordinators fails, the remaining signals in the system maintain coordination.

Interconnected, Master Controlled System

Intersection controllers are physically connected, generally through a hard wire buried in the ground or carried along overhead utility wires. A variety of "master controllers" can implement coordination by advising the "local" or "slave" controllers at the individual intersections when to change phases. The master controller may simply be one of the intersection controllers which acts as a master, and the number of timing plans is dependent on the capabilities of the individual controllers. There may be a separate, independent master controller located in the field or in some convenient office. This independent master can range from a simple electromechanical dial driven by a time clock to a highly sophisticated, programmable controller having the capability of scheduling a number of timing plans. Traffic patterns, however, should be constant over time since the timing plans are prescheduled. This type of system is relatively simple and has the capability of changing timing plans at one location; however, the interconnection may be costly, especially for systems where intersections are far apart.

Traffic Adjusted System

A traffic adjusted system is characterized by the fact that timing plans are adjusted according to changing traffic conditions by an analog computer receiving volume information from sampling detectors located on the roadway. Based on the traffic demand, the computer, within certain constraints, selects the best system cycle length, offsets, and splits. Additional expense is incurred because of the need for detectors.

Computerized System

Computerized systems are characterized by centralized control through a digital computer and two-way communication between the computer and the individual intersection controllers and detectors. The most common control approach is to let the computer handle all of the timing functions based on traffic demand and use the intersection controllers to merely change the signal display lamps. These systems offer practically unlimited flexibility in implementing signal timing plans. They also offer additional advantages, including the ability to monitor system performance and to detect system malfunction. The disadvantages of computerized systems are the high costs of installation and maintenance, and the complexity, which generally requires additional personnel expertise.

Types of Progression

There are four general ways in which continuous flow, or progression, through an arterial signal system is achieved. These are discussed below.

Simultaneous Progression

If simultaneous progression is used, all signals along the route which are in the system operate with the same cycle length and display the green indication at the same time. All traffic moves at one time, and a short time later all traffic stops at the nearest signalized intersection to allow cross street traffic to move. This type of progression is typically used in downtown areas where intersections are close together, 300 to 500 feet, and the spacing is reasonably uniform. Offsets at all intersections are zero. See Figure 16.

Alternate Progression

With alternate progression, there is a common cycle length; however, each successive signal or group of signals along the route which are in the system shows opposite indications. If each signal alternates with those immediately adjacent, the progression is called single alternate. If pairs of signals alternate with adjacent pairs, the progression is called double alternate, and so on. Again, this type of progression is associated with uniform spacing of the intersections. Ideal spacing for single alternate progression is 0.25-mile, or 1,320 feet; however, spacing in the range of from 1,000 to 2,000 feet is satisfactory. Double alternate spacing is best suited with spacings ranging from 500 to 1,000 feet. Offsets are either zero or 50% of the cycle length. See Figures 17 and 18.

Limited or Simple Progression

Limited or simple progression also employs a common cycle length; however, the relationships of the indications between the intersections vary because the spacing of the intersections is nonuniform. Simple progression is used where the pattern of traffic flow is relatively uniform throughout the day. Offsets are different at each intersection. See Figure 19.

Flexible Progression

Flexible progression is identical to simple progression, except that the common cycle length can be changed during the day to reflect changing traffic patterns. Offsets are different at each intersection and for each cycle length being used.

CYCLE: 90 SECONDS

SPLIT: 70/30 PERCENT
63/27 SECONDS

SPACING: 300 FEET

PROGRESSION SPEED:
1500/36 = 42 ft/sec. = 28 MPH

BAND WIDTH:
27 sec. = 30%

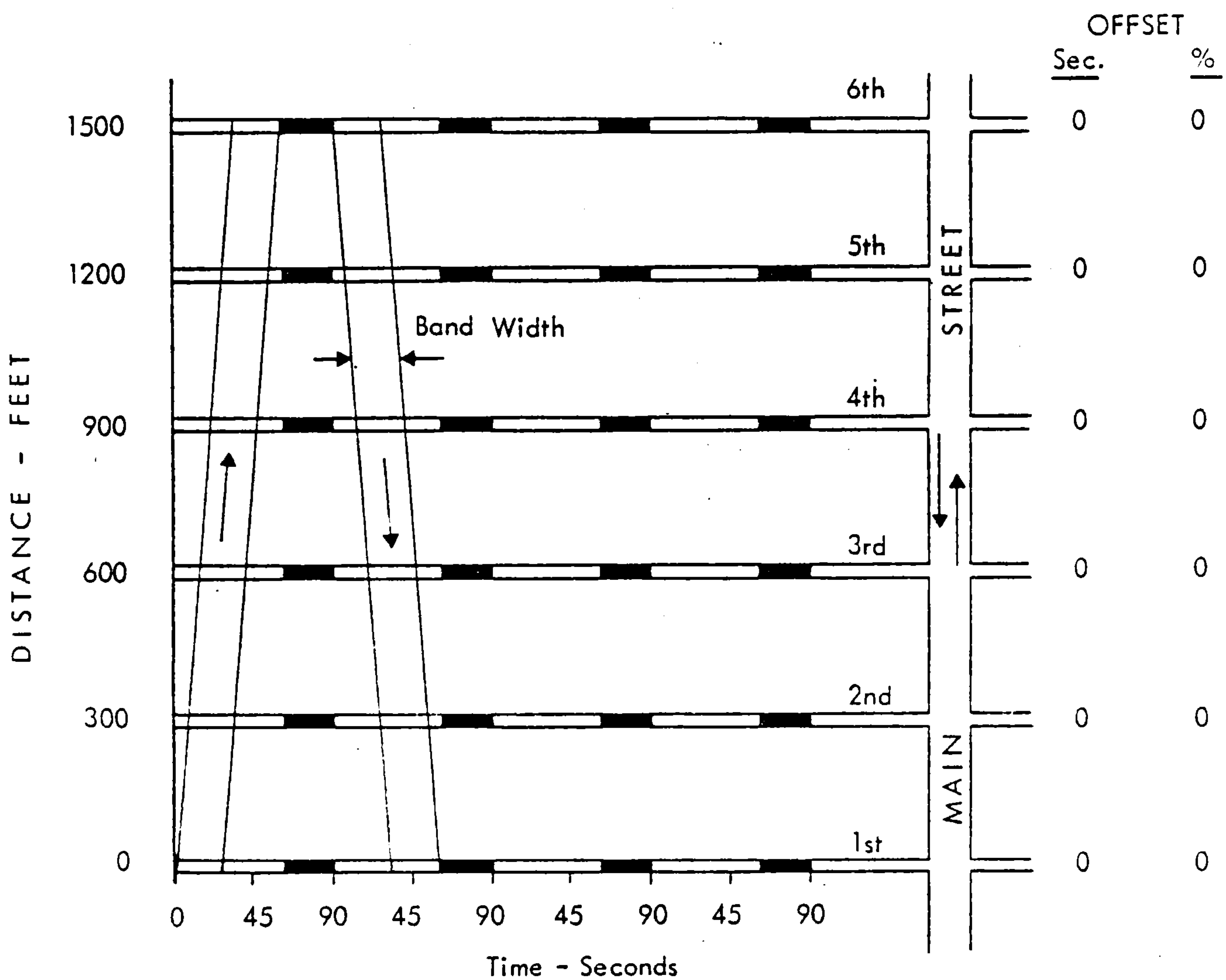


Figure 16. Simultaneous system.
Source: Reference 9.

CYCLE: 60 SECONDS

SPLIT: 50/50 PERCENT
30/30 SECONDS

SPACING: 1200 FEET

PROGRESSION SPEED:
1200/30 = 40 ft/sec. = 27 MPH

BAND WIDTH:
30 Sec. = 50%

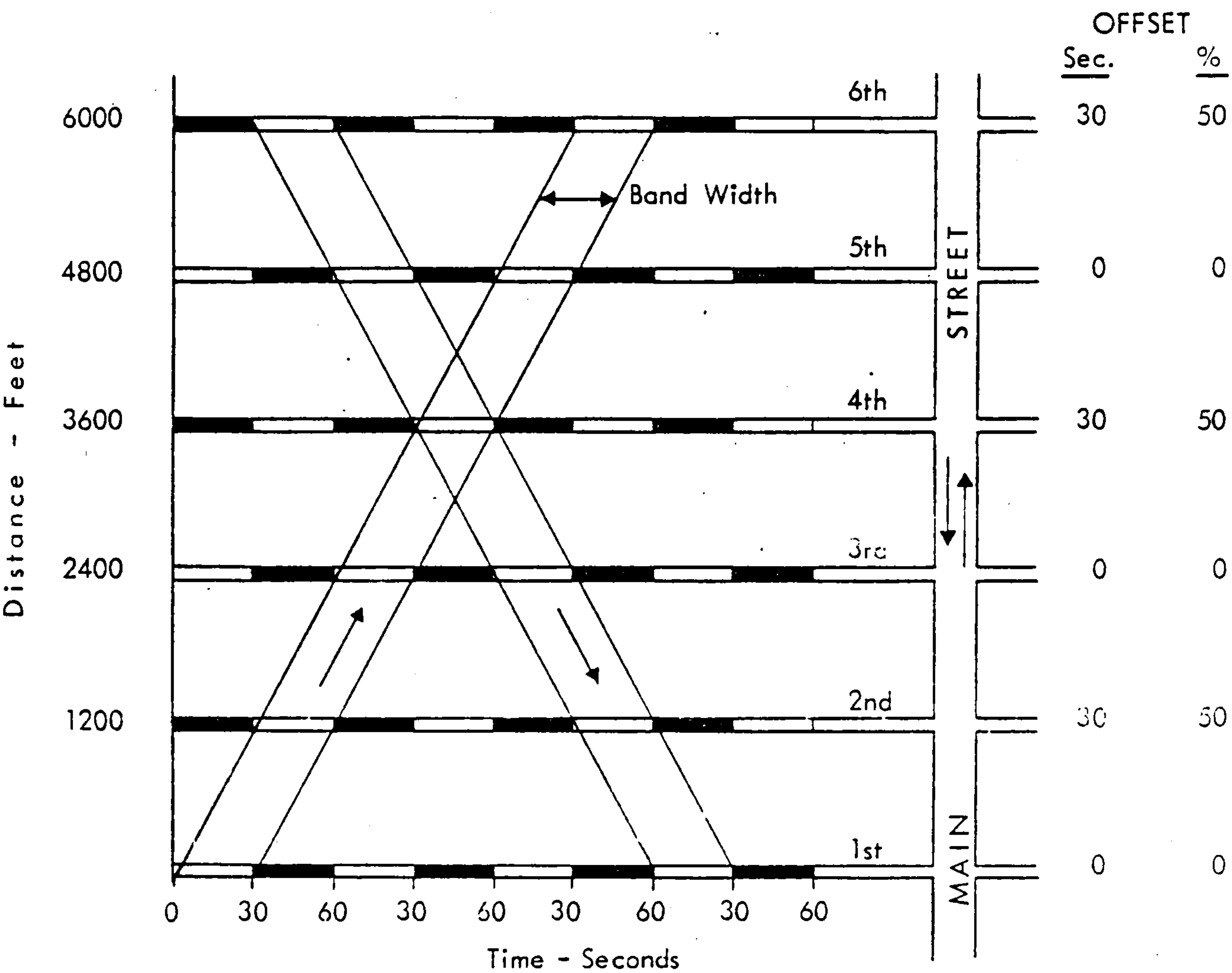


Figure 17. Single alternate system.
Source: Reference 9.

CYCLE: 60 SECONDS

SPLIT: 50/50 PERCENT
30/30 SECONDS

SPACING: 600 FEET

PROGRESSION SPEED:
1200/30 = 40 ft/sec. = 27 MPH

BAND WIDTH:
15 sec. = 25%

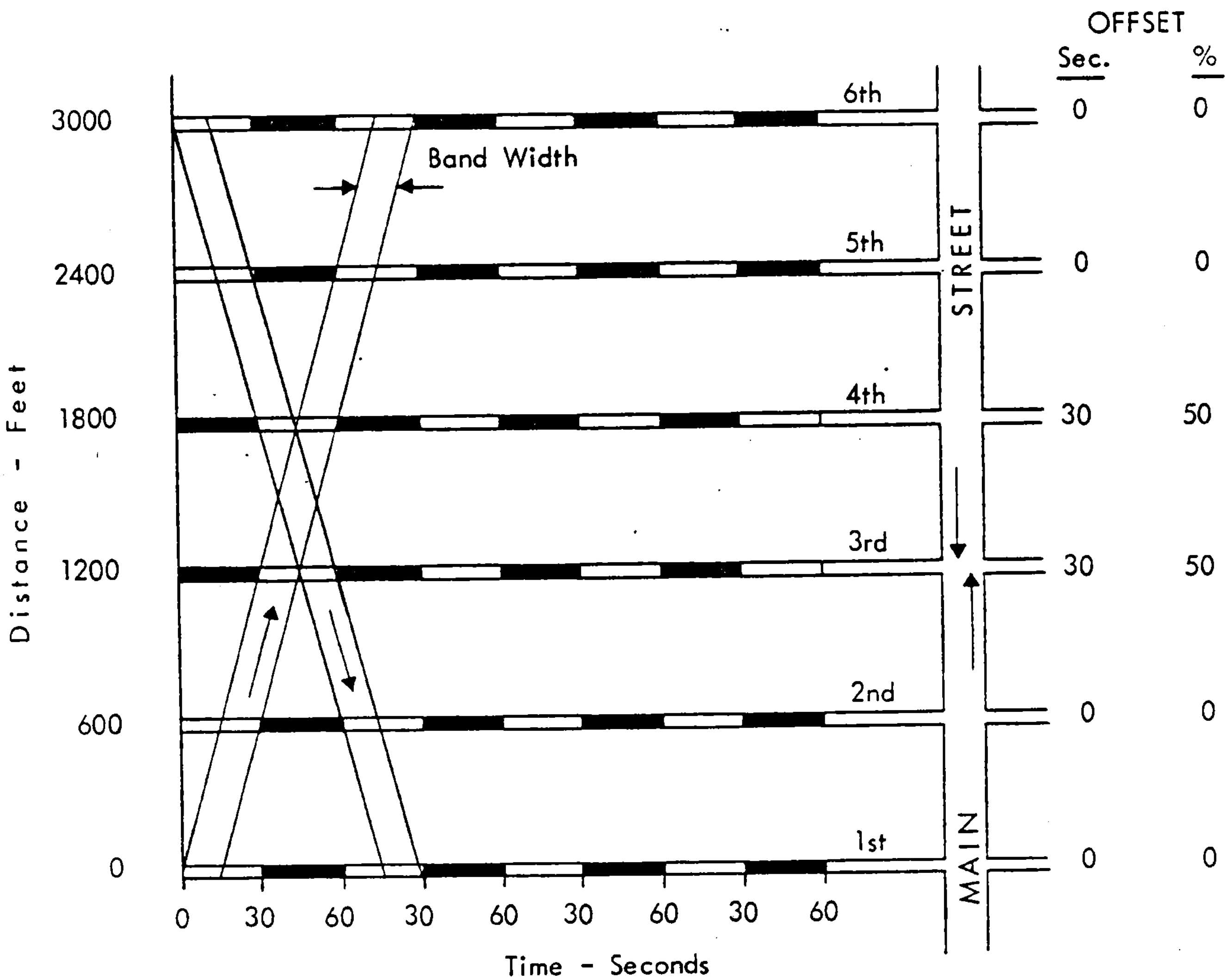


Figure 18. Double alternate system.
Source: Reference 9.

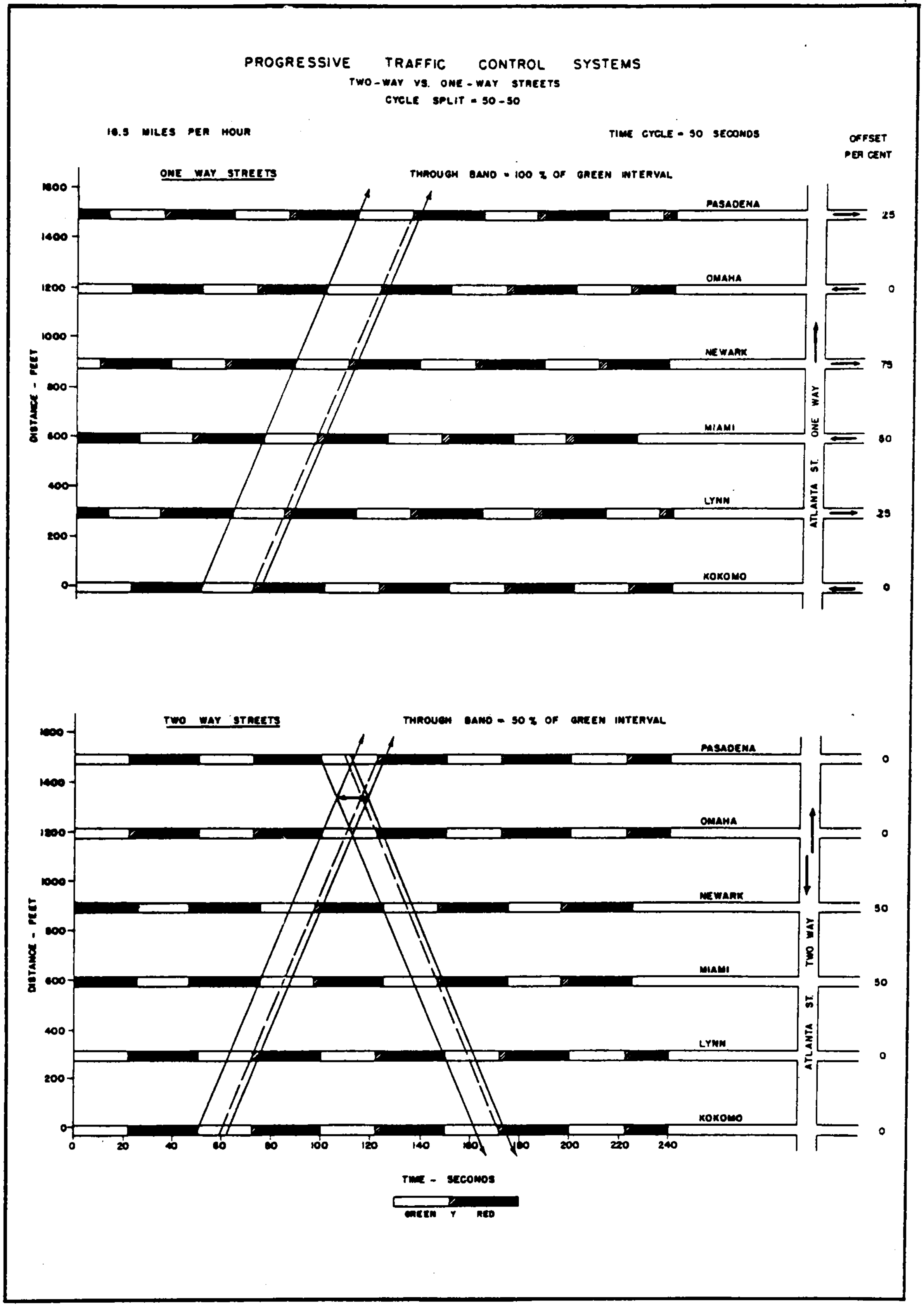


Figure 19. Progressive system.
Source: Reference 2.

Objective

The major objective of signal timing is to assign the right of way to alternate traffic movements so that all vehicles are accommodated with a minimum amount of delay to any single group. The specific objective of a signal system is to facilitate movement of vehicles through a series of signalized intersections. This is accomplished by coordinating the individual intersections in the system, primarily through the establishment of fixed time relationships between intersections.

Timing Procedures

Timing procedures for signal systems become very time-consuming and complex once simple system configurations are exceeded. In recent years computerized procedures for timing systems have been developed, and a separate section of this report presents summary information on the most common of these programs. Manual techniques are useful for relatively simple systems and when a computer is not available, and these are presented in the remainder of this section. Specifically, procedures for the following types of systems are discussed.

Arterial system

1. Uniform block spacing, two-directional flow
   - -- Cycle length not predetermined
   - -- Cycle length predetermined

2. Nonuniform block spacing
   - -- Two-directional flow
   - -- One-directional flow

Signal network

1. Open network

2. Closed network

It is important to note that due to the wide variety of hardware components, it is not feasible to relate timing parameters to specific dial settings. Therefore, the instructions for the equipment being utilized must be reviewed closely and related to the timing parameters developed. Generally, cycle lengths and phase parameters are set on the controllers, while offsets and force offs are set on coordinating units.

Data Collection

Depending on the type of equipment being used, the data requirements discussed previously for pretimed and actuated signals are also applicable to systems. A plot of 15-minute or hourly volumes by direction on the major arterial is a useful tool in setting system timing. A graph of this nature allows an easily visualized determination of when cycle lengths and offsets should be changed and if one-way or two-way progression is acceptable. Threshold volumes for changing the cycle length and offsets can also be selected directly from the graph.

Arterial Systems

It is feasible to manually develop timing plans for simple arterial systems. Two categories of arterial systems can be identified for purposes of timing--those with uniform block spacing and those with nonuniform block spacing. Following are procedures for these categories. The procedures have been excerpted from the Institute of Transportation Engineer's Transportation and Traffic Engineering Handbook, dated 1976,(10) and from the University of Texas' Center for Transportation Research report entitled Adding Signals to Coordinated Traffic Signal Systems.(11)

Uniform Block Spacing -- Two-directional Flow, Cycle Length Not Predetermined

The following methodology is used when a street that is not part of any other system and when the cycle length is restricted only by the traffic requirements at individual intersections along the route.

1. Select a desired speed of progression for the system.

2. Compute the time required to travel one block at the desired speed.

3. Select a single, double, or triple alternate system on the basis of time required for a round-trip from the first intersection to the second, third, or fourth intersection. If a round-trip to the second intersection results in an acceptable cycle length that satisfies the traffic requirements at all intersections, use the single alternate system; if the trip to the third intersection and back gives a good cycle length, use the double alternate system; if the round-trip to the fourth intersection gives a better cycle length, use the triple alternate system.

Example:

Uniform block spacing of 400 ft
Desired speed of 25 mi/h
25 mi/h = 36.7 ft/s

$$\text{Travel time per block} = \frac{400\text{ ft}}{36.7\text{ ft/s}} = 10.9\text{ sec}$$

Round-trip to second intersection = 21.8 sec
Round-trip to third intersection = 43.6 sec
Round-trip to fourth intersection = 65.4 sec
In this example, a double alternate system with a 45-sec cycle length would be used if the 45-sec cycle satisfies the traffic conditions at the individual intersections.

4. The offsets for all signals would be either zero or one-half the cycle length. For example, in a double alternate system with a 45-sec cycle, the first two intersections would have zero offset, the next pair 22.5-sec offsets, the next pair zero, etc. Non-signalized intersections are included when determining offsets.

5. The division of the cycle length, i.e., green, yellow, and red intervals, for individual intersections is obtained by analyzing each case. Thus, although the beginning of the green interval is synchronized to provide coordinated flow, the end of the green interval may present a slightly irregular pattern.

6. The through band width depends on the system that has been selected. For a single alternate system, the width of the through band is equal to the shortest green plus yellow period; for a double alternate, the width is one-half the green plus yellow; and for the triple alternate, the width is one-third the green plus yellow. The triple alternate should be used sparingly because of the reduction in the efficiency of the system.

Uniform Block Spacing -- Two-directional Flow, Cycle Length Predetermined

The following methododology is used when the cycle length is predetermined, e.g., one intersection may be part of an intersecting coordinated system.

1. Obtain block spacing and cycle length.

2. Determine speed of progression by dividing the block spacing by one-half, one-fourth, and one-sixth of the cycle length, respectively, for single, double, or triple alternate systems.

Example:

Uniform block spacing of 400 ft
Cycle length of 50 sec

Single alternate $\dfrac{400 \text{ ft}}{\frac{1}{2}(50 \text{ sec})}$ = 16 ft/s or 10.9 mi/h

Double alternate $\dfrac{400 \text{ ft}}{\frac{1}{4}(50 \text{ sec})}$ = 32 ft/s or 28.0 mi/h

Triple alternate $\dfrac{400 \text{ ft}}{\frac{1}{6}(50 \text{ sec})}$ = 48 ft/s or 32.7 mi/h

In this example, a double or, possibly, a triple alternate system would be used, depending on the desired speed.

Nonuniform Block Spacing -- Two-directional Flow

A time-space diagram is used to develop a timing plan for a system with nonuniform block spacing. Before the diagram can be constructed, however, the background or common cycle for the system and the needed splits at each intersection must be determined. Normally, the cycle required to handle the traffic at the highest volume intersection in the system is chosen as the background cycle. For a pretimed intersection, the optimum cycle length is calculated as described in the previous section on pretimed signals at isolated intersections. In the case of actuated control, the intersection is considered to operate at maximum loading, or in a pretimed manner, and thus the cycle length is also determined as described previously for pretimed control. Once the cycle length is determined, the splits are then calculated as described under pretimed control. Again, actuated signals are assumed to operate at force off or pretimed conditions.

Also, a desired speed of progression and the tolerable variations from this speed must be specified. The character of the arterial and its surroundings will guide the decision concerning reasonable speeds.

In the case of two-directional flow, equal opportunity for progression should be given to each direction. Specifically, the objective is to have the same speed of progression and band width in each direction; such is the case in the off-peak hours when the directional split is about the same.

A general graphical solution for determining the timing plan for off-peak signal timing was developed by James H. Kell. Symmetry in the slope and width of the through band on the time-space diagram is attained by centering either the red or the green arterial signal interval on a reference point such that the beginning of artery green will be offset properly for a speed of progression within the tolerable range.

The procedure for constructing a time-space diagram for an off-peak timing plan by Kell's Method is illustrated in the following steps for the series of intersections spaced as shown in Figure 20. For this example, the required cycle length is 80 seconds and the percentage of cycle time that will be allocated to artery green is given at the top of the diagram. The tolerance range for progression speed is from 25 to 30 mi/h. The yellow phase-change interval is included in the artery green.

1. Locate each signalized intersection along the horizontal axis using a scale such that all intersections in the section will fit on the long axis of the sheet (1 in = 60 ft) and draw a vertical line at each location. Identify each intersection A through E and note the cumulative distance from the beginning of the section to each intersection. Write the percentage of cycle time allocated to artery green at the top of each vertical line which locates the intersection.

2. Locate a vertical scale which makes 2 in equal to 80 sec (40 divisions per in) and graduate the vertical line at the first intersection into 80-sec time intervals. See Figure 20.

3. Calculate the time, T, required to travel the full length of the section (5,000 ft) at 25 mi/h and at 30 mi/h.

   $$T_{25} = (5000)(3600)/(25)(5280) = 136 \text{ sec}$$

   $$T_{30} = (5000)(3600)/(30)(5280) = 114 \text{ sec}$$

   Draw a speed-of-progression line from the origin to each of these times measured along the vertical time line at the 5,000-ft location. Note the speed on each line. See Figure 20.

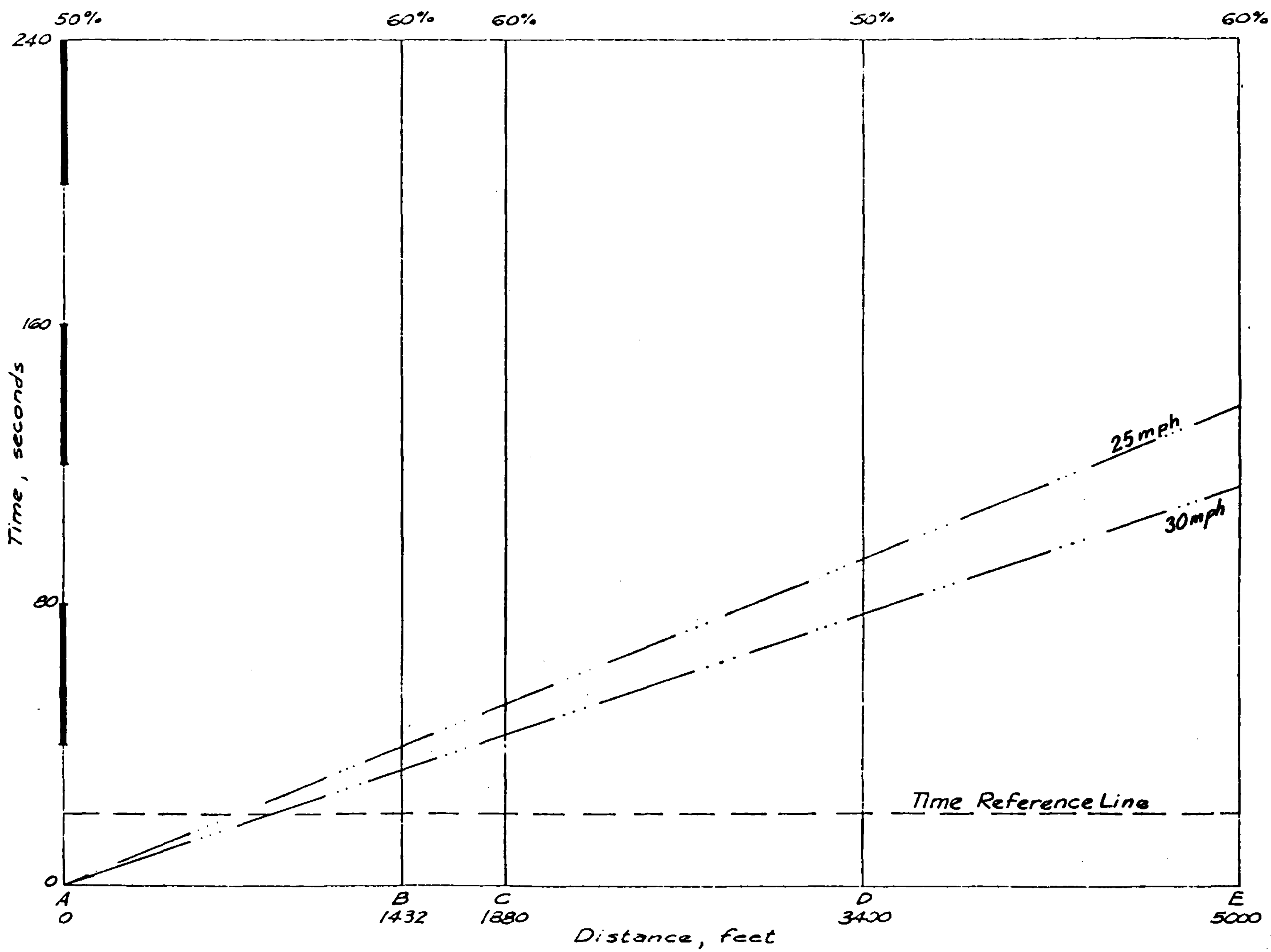


Figure 20. Time-space diagram construction.
Source: Reference 11.

4. Carefully fold the cycle split aid, Figure 21, vertically and crease the paper at each percentage green value shown at the top of the diagram. This aid was developed by Professor Clyde E. Lee at the University of Texas at Austin in the 1960s for constructing time-space diagrams. With the aid folded, the shading along the crease indicates artery green time by white and artery red time by black. The center of each of these intervals is marked on the aid.

5. Place the aid, folded at 50%, adjacent to the vertical time line at intersection A with the beginning of artery green (white on aid) at the origin. Mark heavy bars on the diagram along the vertical time line to show artery reds (black on aid), being careful to start and end these bars accurately. Also mark the center of the first green interval and draw a horizontal line on the diagram to serve as a reference time at the other intersections. NOTE: The aid may be used at the 5,000-ft intersection to locate the horizontal reference time line accurately on the diagram. The successive green and red signal indications that will be viewed by drivers on the artery as they approach intersection A are thus shown on the vertical time axis of the diagram.

6. Next, fold the aid to the percentage of artery green at intersection B and align the crease beside the vertical time line at this intersection location. Adjust the aid vertically to center the artery red indication on the horizontal time reference line and notice that the beginning of artery green is offset for a speed of progression of approximately 26 mi/h and that most of the artery green remains to accommodate a platoon from A. This is within the tolerable speed range; therefore, centering artery red is accepted for defining the offset at this intersection. Draw bars on the diagram at B with red centered on the time reference line to indicate the red intervals on the artery. If green is centered on the time reference line, only a few seconds of artery green will remain for the platoon from A and a very narrow band width would result. This is, therefore, not an acceptable offset. See Figures 22 and 23.

7. Repeat the procedure described in 6 above for each signalized intersection in the system. Either artery red or artery green must be centered on the time reference line. The decision as to which is based on the objective of allowing an acceptable speed of progression with a maximum band width (a function of the end of artery green). See Figure 24.

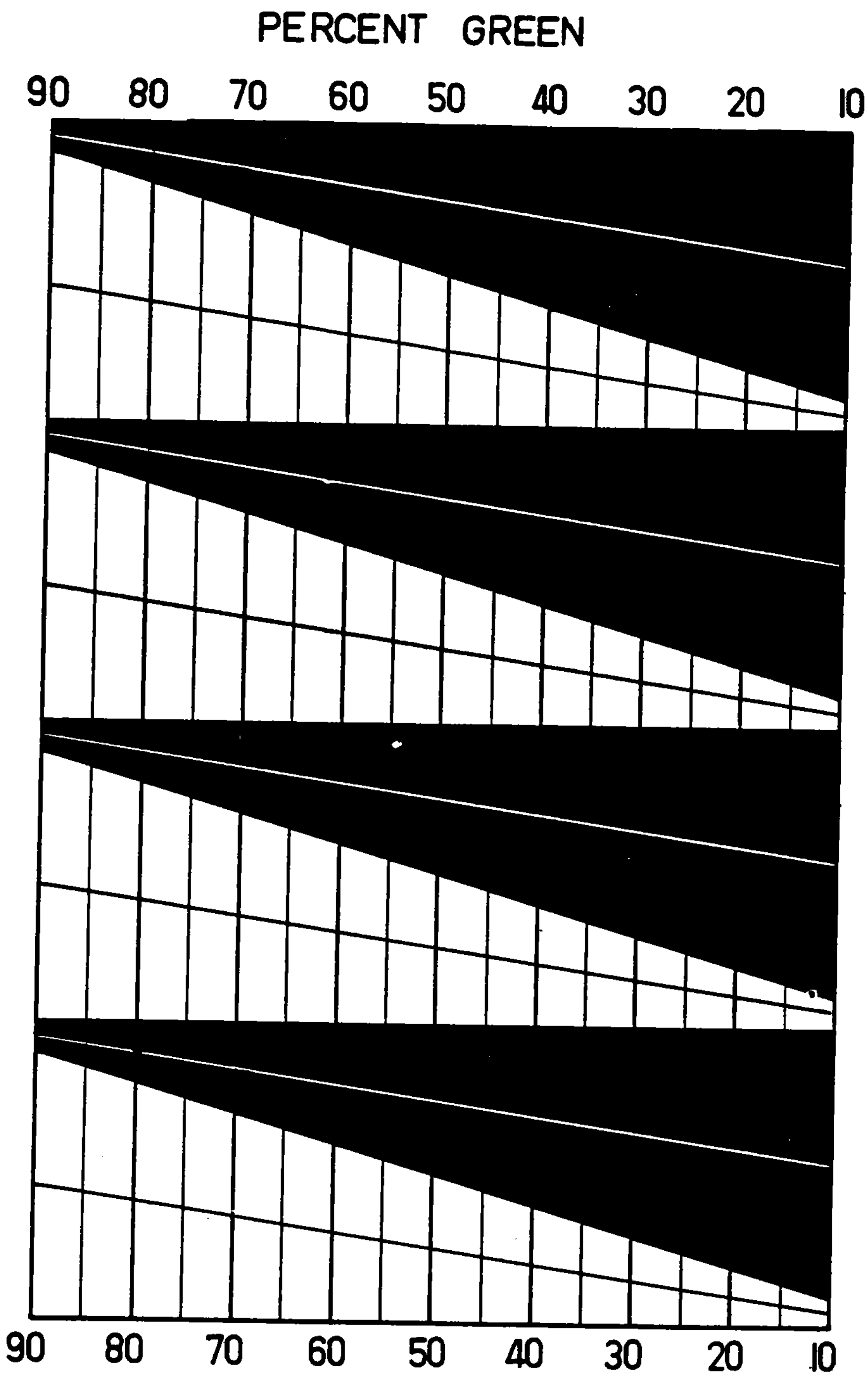


Figure 21. Cycle split aid.
Source: Reference 11.

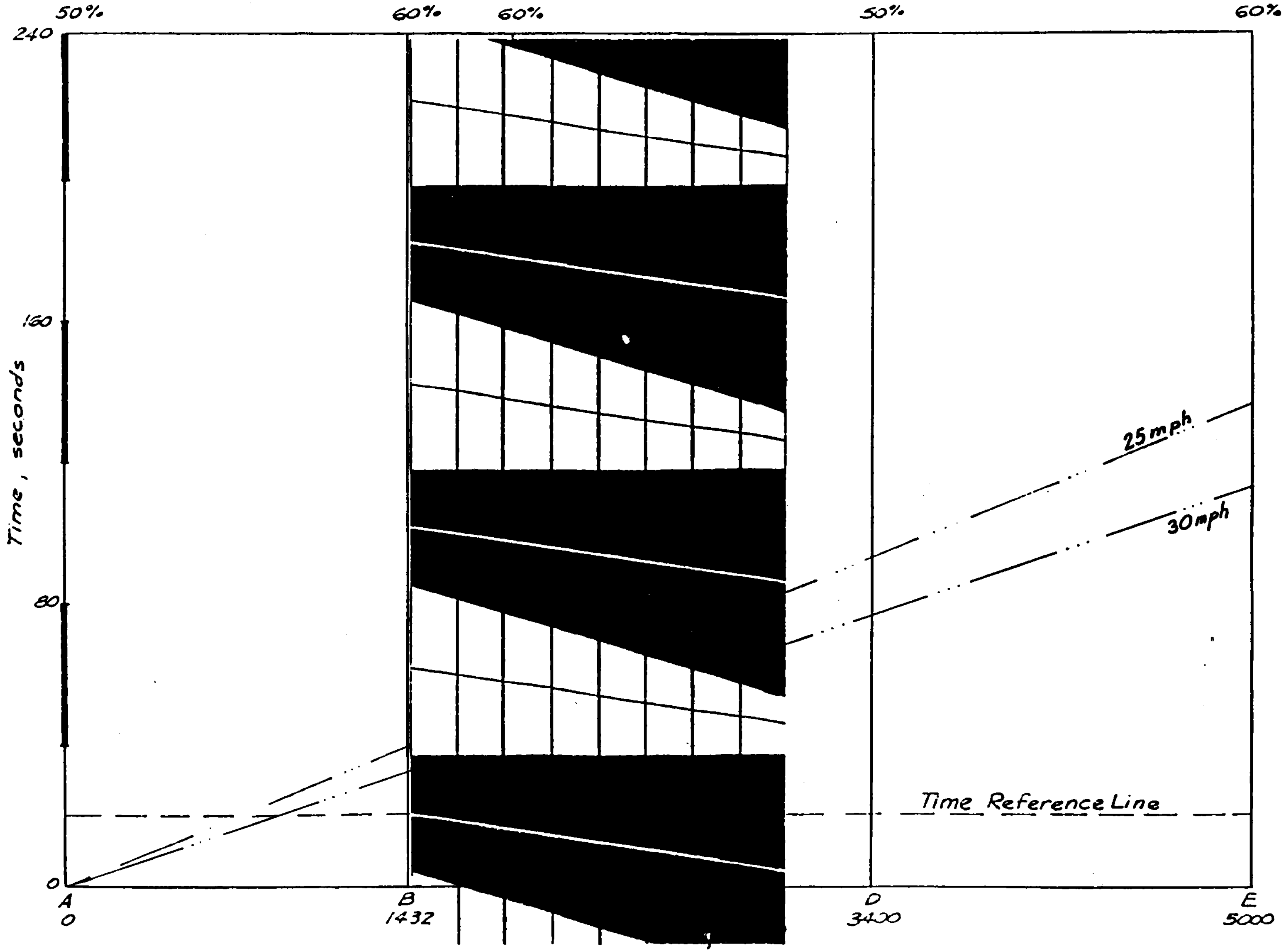


Figure 22. Time-space diagram construction.
Source: Reference 11.

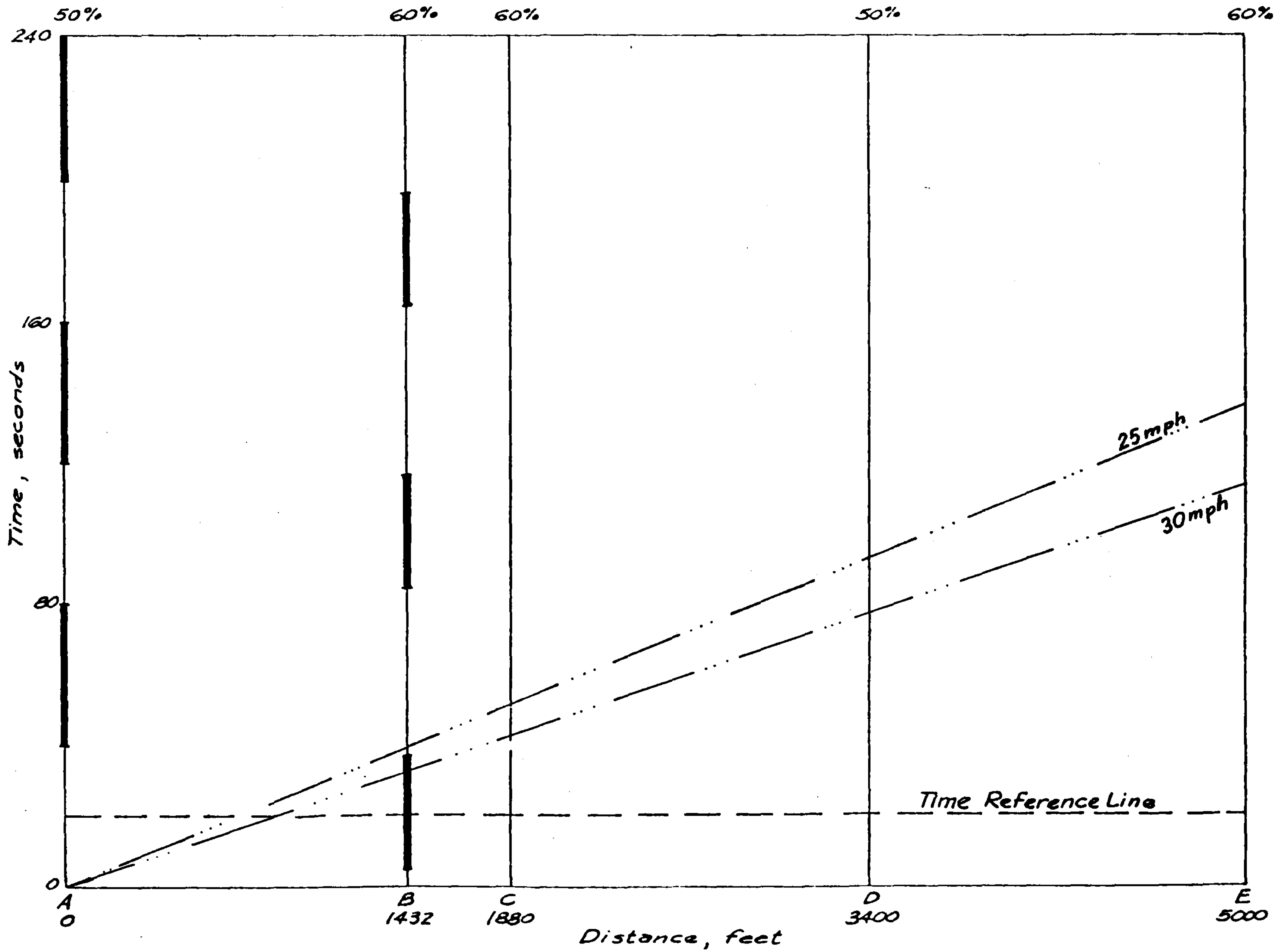


Figure 23. Time-space diagram construction.
Source: Reference 11.

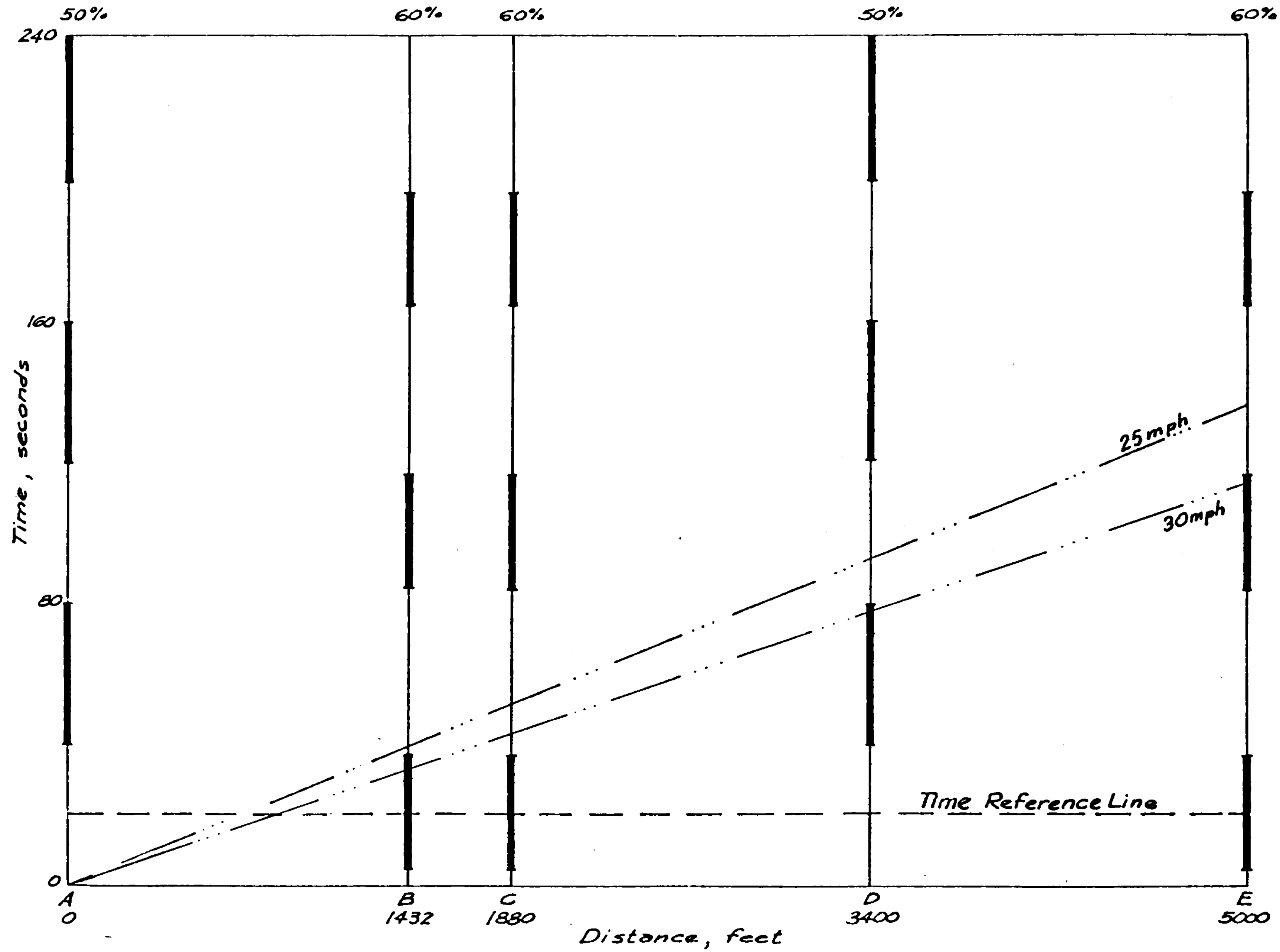


Figure 24. Time-space diagram construction.
Source: Reference 11.

8. Now, the uniform speed of progression for a platoon moving from A to E is determined by fitting a sloping straight line through the beginning of the two artery greens that will provide the highest speed of progression. In the example, B and E control this speed.

9. The band width is the time allowed for a platoon of vehicles to move completely through the system at uniform speed and is measured on the diagram along the vertical time axis. On the diagram, the band width is determined graphically by fitting a line parallel to the speed of progression line through the end of the artery green that limits the band width most. In the example, the band width for the platoon from A to E is controlled by the end of green at A. Draw the parallel line to define the band width. The actual band width can be measured in seconds on the diagram with a scale (1 in = 40 sec). The band width is 36 sec in the example. See Figure 25.

10. An exact mirror image of the through-traffic band from A to E can be drawn on the diagram for traffic moving from E to A. The controlling times are indicated by circles on the diagram. See Figure 25. This completes the construction of the time-space diagram.

11. Offsets for setting the signal controller at each intersection can be scaled from the diagram with adequate precision for practical purposes, but they can also be calculated from the relative time values shown on the diagram.

Nonuniform Block Spacing -- One-directional Flow

Progression for one-directional flow is applicable in systems on one-way streets or where there is heavy directional flow on the artery in the morning and evening peak periods. The procedure is to offset the beginning of the artery green at each intersection such that it coincides with the arrival of the lead vehicle in a platoon traveling on the artery at the desired progression speed. Traffic in the other direction may or may not experience progression through the system. The construction of a time-space diagram for the case of one-directional flow is illustrated in the steps below. Before beginning, however, the cycle length and splits are determined as they were for two-directional flow. Likewise, the speed of progression must be specified.

1. A basic layout for a time-space diagram is prepared with all signalized intersections located along the horizontal scale.

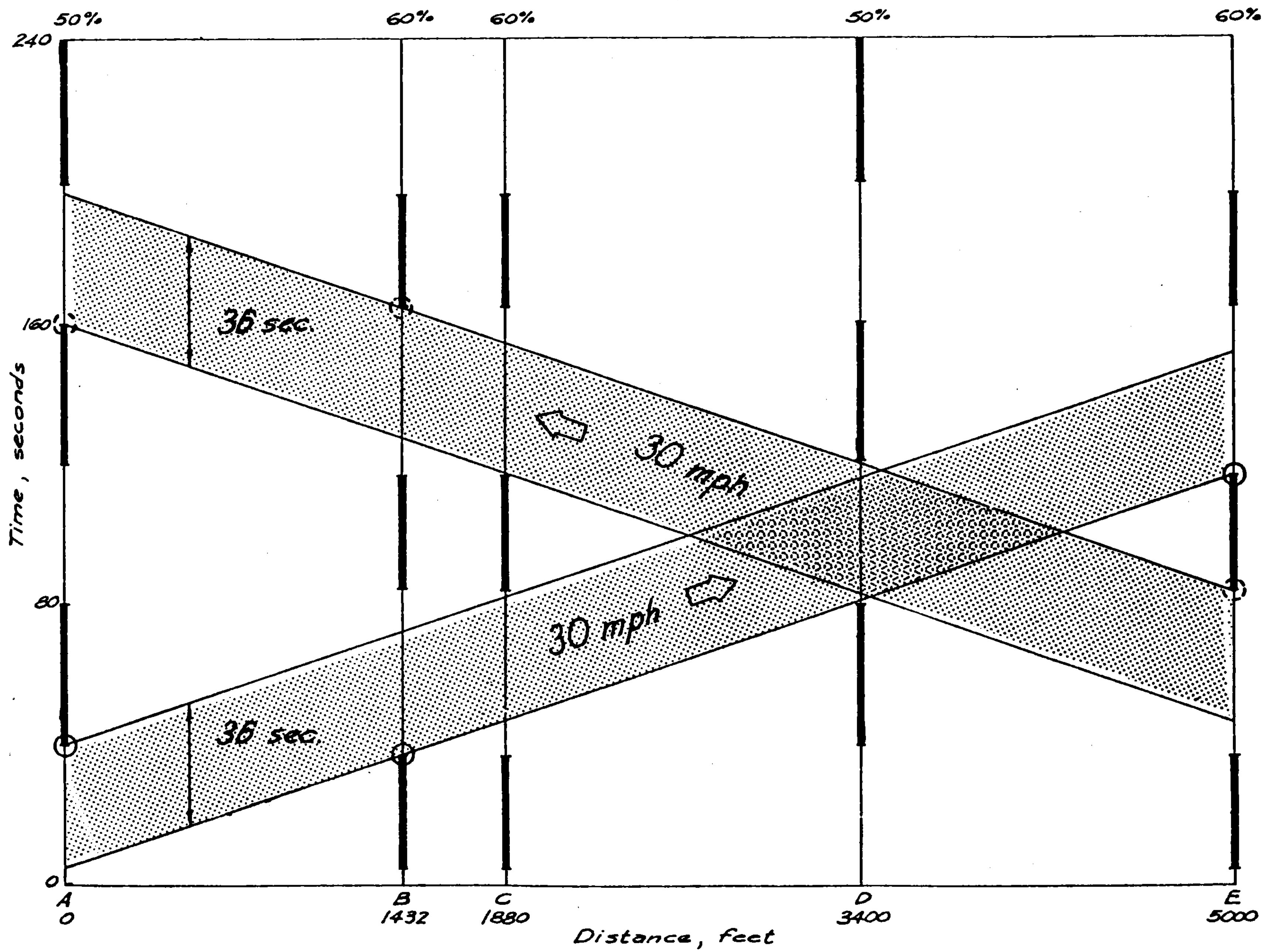


Figure 25. Completed time-space diagram.
Source: Reference 11.

2. A construction line is drawn across the diagram with a slope equal to the desired speed of progression. This line is the bottom line of the through band.

3. The phases are then constructed at each intersection so that the beginning of a green phase is placed on the construction line at each intersection.

4. The top line of the through band is placed parallel to the bottom line. If all signals have the same phase length, then the through band width is equal to the green plus yellow portion of the cycle. If the phase is not the same at all signals, the through band width is equal to the shortest green plus yellow period in the system.

5. Offsets are determined by measuring the displacement of the beginning of the green interval at individual intersections from the beginning of the green interval at the master station.

6. For the example system, assume a cycle length of 60 sec, a speed of progression of 25 mi/h, and direction of progression from A Street to R Street.

    a. Line A is first constructed with a slope equal to 25 mi/h with a 60-sec cycle. See Figure 26.

    b. Signal phases are laid out at each intersection with the beginning of green placed on line A.

    c. The top line of the through band is then drawn. Since in the example system there is a uniform split of 50%, the through band is equal to the green plus yellow period of 30 sec.

    d. Assuming A Street to be the master intersection with zero offset, the individual intersection offsets are as shown in Figure 26.

    e. It should be noted that although no recognizable through band exists in the opposite direction, opposing traffic may still travel through the system, but it will be stopped at one or more signals.

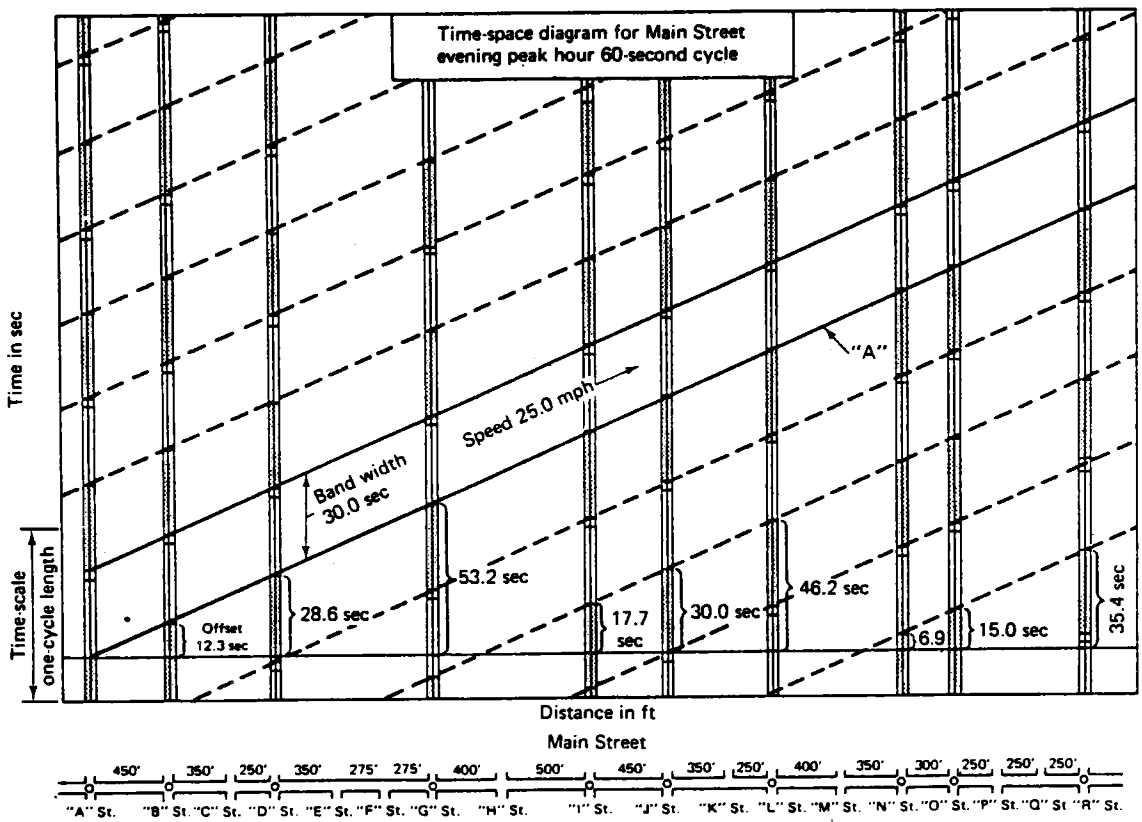


Figure 26. Completed time-space diagram favoring one direction of flow. Source: Reference 10.

Signal Network

The procedures described thus far relate to systems along a single route. As discussed previously, if two or more routes cross at a common intersection, the result is a signal network.

Open Network

An open network contains only one common intersection and, in general, the cycle length for the network is fixed by the requirements at this common intersection. The cycle length and splits at the common intersection are determined as described previously for pretimed control at isolated intersections. This cycle is then used to calculate the splits at all other intersections in the network, again using the previously described procedures for pretimed control. Development of the timing plan for each route then proceeds independently as described for arterial systems; that is, a progression speed is specified and the appropriate time-space diagram is constructed.

Closed Network

A closed network, or grid system, contains two or more common intersections. All signals in the network should have the same cycle length, which is the longest cycle required by any intersection in the network. After the cycle length is selected, the timing of each route should be developed separately. If necessary, adjustments are then made to the offsets or green and yellow times, or both, to achieve a balance. In other words, the sum of the offsets plus the green and yellow times taken in sequence around a closed network must be equal to the network cycle length or multiple thereof. Manual application of these procedures is difficult and generally not needed by the targeted group for this study; therefore, the procedures are not described further. In practice, those responsible for grid systems usually have access to a computer and the timing programs described in the next section of the report. However, a manual procedure for analyzing a simple closed network is provided in Appendix C. This procedure has been reproduced from the course notebook for a signal workshop conducted by the Georgia Institute of Technology.(12)

SIGNAL TIMING COMPUTER PROGRAMS

There are a variety of computer programs which calculate signal timing. There may be versions of the same program that run on mainframe, mini, or microcomputers, and the programs may be in the public domain, which are free except for processing charges, or may be for sale by private companies.

The following general points should be made about signal timing computer programs.

1. The procedures used by the computer programs are essentially the same as described in this report, and thus require the same data. The advantage, obviously, is that the procedures can be performed very quickly.

2. Several of the programs have been used extensively and, when used properly, produce valid results.

3. Several of the programs are available for or are being developed for use on microcomputers.

4. The programs do not replace the engineer; rather, they provide him with a valuable tool.

Following are brief descriptions of several of the most commonly known computer programs. The information, which is primarily from Transportation Research Circular No. 282(13), is intended merely to acquaint the reader with the programs, it is not comprehensive enough to allow one to determine whether the program is applicable to his specific needs.

SOAP 84

SOAP is a macroscopic analysis with the primary objective of developing signal control plans for individual intersections. It develops cycle lengths and splits which minimize a performance index.

SOAP can analyze up to 48 time periods of from 5 to 60 minutes each, and one intersection is simulated per run. Inputs include traffic flows per approach, truck and bus composition, left-turn data, signal-related data, and saturation flow rates. Basic outputs include delay, percent saturation, maximum queue, percent stops, excess fuel, and left-turn conflicts. More are available on request.

Program documentation is well written and the program is easy to use. It has been used extensively and is available for the micro.

MAXBAND

MAXBAND is a band width optimization program that calculates signal settings on arterials and triangular networks. The program produces cycle lengths, offsets, speeds, and phased sequencing to maximize a weighted sum of band widths. It can handle as many as 20 signals.

Basic inputs include the range of cycle lengths, network geometry, traffic flows, saturation flows, left-turn patterns, queue clearance times, and range of speeds. Outputs include a data field manual and a solution report that contains cycle time and band widths, selected phase sequencing splits, offsets, and travel times and speed on links.

The main advantage is the freedom to provide a range for the cycle time and speed. The main disadvantage is its use of band width as its optimization criterion. Other disadvantages include the limited experience with field testing and the lack of incorporated bus flows in the optimization. It runs only on the mainframe computer.

PASSER II-80

PASSER II-80 is a band width optimization program that calculates signal timings on linear arterials. The program uses a fixed-time scan search to produce the cycle length, phase sequencing, splits, offsets, and band speed that maximize band width in both directions for up to 20 intersections. A modified version of Webster's delay equation is used to approximate platoon effects.

Basic inputs include the range of cycle lengths, movement flows, saturation flows, left-turn patterns, queue clearance times, desired speeds, minimum green times, allocation of band width by direction, cross street phase sequences, and intersection distances. Outputs include cycle length, band widths, band speeds, a time-space diagram, delay, probability of queue clearances, offsets, splits, phase sequences, and volume-to-capacity ratios.

Its main advantage is its flexibility to vary cycle length and band speed and its ability to consider multiphase operation under a variety of sequencing strategies. Other advantages are ease of input and low run times. The main disadvantage is its use of band width as its optimization criterion. Also, it does not accommodate closed networks, and fuel consumption and emissions are not included. It has been used extensively and is available for the micro. PASSER II-84 is also available.

PASSER III

PASSER III produces the cycle length, splits, and phasing sequence for a pretimed diamond interchange that minimize average delay per vehicle, using a macroscopic, deterministic time scan optimization. It can also determine splits and offsets for interchange signals along a frontage road, using a band width procedure.

Inputs in addition to those required for PASSER II-80 include the interchange description for the isolated case and interchange spacing and progressive speed for the progression case. Outputs include signal settings plus value for delay, degree of saturation, etc. For the progression case, band width, speeds, efficiency, and time-space diagrams are provided. The main disadvantage is its use of band width as the progression criterion.

SIGOP

SIGOP produces the cycle length, splits, and offsets of signals in a grid network that minimize a delay in disutility function by using a macroscopic traffic flow model. It can handle up to 150 intersections.

Inputs include arrival flows and saturation flows (in terms of headways), minimum green times, yellow times, special phase times, and passenger car equivalent factors for trucks, buses, and turning vehicles. Outputs include time-space plots along selected arterials and link statistics.

Signals with up to four phases can be modeled. Disadvantages include run times for large networks that are no shorter than other programs, and SIGOP III (the latest version) lacks extensive field testing.

TRANSYT

TRANSYT produces splits and offsets for signals in a network that minimize a performance index by using a hill-climbing procedure and a macroscopic, deterministic flow model. Its dimensions are usually set to handle up to 50 intersections and 300 links. Numerous versions of the program have been produced, both by its originator and others. In most versions, the performance index is a user-specified balance between delay and stops. Phasing and cycle length are not optimized in most versions.

Basic inputs include cycle length, phasing, performance index weights, lost time, link lengths, either link travel times or speeds, link flows, turning movements, and saturation flows. Basic link outputs include percent saturation, total travel, travel time, delay, rate of stops, maximum queue lengths, offsets, and splits. Network outputs include similar statistics plus the value of the performance index. Flow profile plots are optional.

TRANSYT-7F uses North American nomenclature on input and output (rather than English). It also produces a time-space plot and estimates of fuel consumption. A recent revision optimizes cycle lengths and identifies potential intersection blockages.

TRANSYT's main advantage is that it uses a fairly realistic flow model without requiring outrageous run times. The main disadvantage is the extensive data collection required. It has been used extensively and is available in a microcomputer version.

Sources of Information

Detailed information on the previously mentioned programs plus other programs not mentioned can be found in the following sources.

1. Developments in Traffic Signal Systems, Transportation Research Circular Number 282, July 1984.

2. Handbook of Computer Models for Traffic Operations Analysis, Technology Sharing Report FHWA-TS-82-214, Federal Highway Administration, December 1982.

3. The Application of Traffic Simulation Models, Special Report 194, Transportation Research Board, 1981.

4. Microcomputers in Transportation, Transportation Research Record 932, Transportation Research Board, 1983.

5. Microcomputers in Transportation - Software and Source Book, UMTA Technical Assistance Program, Urban Mass Transportation Administration, February 1985.

6. STEAM - As part of the Federal Highway Administration's technical assistance activities, a user support group for microcomputer applications in traffic engineering has been established. The following information on the services provided by this group, which is called the Safety and Traffic Engineering Applications for Microcomputers (STEAM) User Group, is summarized from its newsletter.

Software Exchange

To increase the availability of public-domain software to members of the User Group, a clearinghouse has been established to distribute Federally developed software and also to collect, review and distribute software contributed by members. STEAM members can obtain software, documentation, and installation instructions at little cost.

Software Support

Federally-developed software packages and certain contributed packages are fully supported by the STEAM Support Center. Full support means that the packages will be maintained by correcting errors and making enhancements. Consultation and updates to users will also be provided.

User Group Software

Software packages that are submitted to the center and not fully supported can be copied and distributed to members in the "as-is" condition. Limited assistance will be given by the center's staff.

Software Costs

The pricing policy is to charge small fees to help defray the cost of diskettes, handling, and making copies of documentation.

Technical Assistance

User Group members can write, visit or call the Support Center with general questions about the User Group and Support Center, or with technical questions regarding hardware and software selection and use.

Data Base Information Service

The Support Center can serve as a liaison between users having similar problems with or questions about certain programs. Contacts with actual users of a program can be very beneficial.

Notice

Effective October 1, 1985, the Support Center for the STEAM user group is in transition to, as of this writing, an undetermined location. It is anticipated that the Support Center will be established in its new location by early 1986. In the meantime, the aforementioned services provided by the STEAM Support Center can be handled to a limited extent by the Federal Highway Administration's Systems and Software Support Team (HTO-23), located in the Office of Traffic Operations, 400 7th Street, S.W., Washington, D.C., 20590. The telephone number is (202)426-0411.

ACKNOWLEDGEMENTS

Many people assisted the author during the course of the study. Special recognition and appreciation are extended to Mark Hodges, Bob Yates, and Travis Bridewell for providing information on the Department's procedures and on practical applications of the procedures. Further, the author is indebted to Dr. Jim Hurley for his assistance with the pretimed procedures. Thanks also go to the Department's district highway and traffic safety engineers for their review of the procedures, and to John Shelor, Steve Blackwell, and Gwen Harris for assistance in analyzing the survey questionnaire. Finally, appreciation is extended to Jan Kennedy for typing the draft of the report, to Neal Robertson for his critique, and to Harry Craft and his staff for editing and preparing the final report.

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1. Cottrell, B. H., Jr., "Guidelines for Exclusive/Permissive Left-Turn Signal Phasing," VHTRC 85-R19, Virginia Highway and Transportation Research Council, Charlottesville, Virginia, January 1985.

2. Manual on Traffic Signal Timing, prepared by the Virginia Section, Institute of Transportation Engineers, sponsored by the Virginia Department of Transportation Safety, January 1982.

3. Kell, James H., and Iris J. Fullerton, Manual of Traffic Signal Design, Institute of Transportation Engineers, 1982.

4. Traffic Signal Manual of Installation and Maintenance Procedures, 2nd ed., International Municipal Signal Association, Fort Worth, Texas, 1981.

5. Principles of Traffic Actuated Signal Control, Traffic Control Division, Automatic Signal, LFE Corporation, May 1984.

6. Traffic Control Devices Handbook, U.S. Department of Transportation, Federal Highway Administration, 1983.

7. Transportation and Traffic Engineering Handbook, 2nd ed., Institute of Transportation Engineers, 1982.

8. Traffic Control Systems Handbook, Implementation Package 76-10, U.S. Department of Transportation, Federal Highway Administration, June 1976.

9. Signal System Timing, excerpt from a course notebook, The Traffic Institute, Northwestern University, Evanston, Illinois.

10. Transportation and Traffic Engineering Handbook, Institute of Traffic Engineers, 1976.

11. Machemehl, Randy B., and Clyde E. Lee, "Adding Signals to Coordinated Traffic Signal Systems," Research Report Number 260-1F, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, August 1983.

12. Manual Calculations for Network Coordination, excerpt from a course notebook, Georgia Institute of Technology, Atlanta, Georgia.

13. "Developments in Traffic Signal Systems," Transportation Research Circular, Number 282, Transportation Research Board, July 1984.

APPENDIX A

INVENTORY OF SIGNAL EQUIPMENT IN VIRGINIA

INVENTORY OF SIGNAL EQUIPMENT IN VIRGINIA

The Virginia Highway and Transportation Research Council of the Department of Highways and Transportation has prepared this questionnaire to obtain an inventory of signal equipment being utilized in Virginia. We are interested in both the kinds of equipment and their modes of operation. Specifically, the inventory has been divided into the categories of controllers at isolated intersections, controllers in systems, auxiliary equipment, and detectors.

Your assistance in completing the questionnaire would be greatly appreciated. Please feel free to add sheets if the space provided is not adequate. Thank you in advance for your help. Please call me at (804) 293-1931 if you have any questions.

Please return in the enclosed, postpaid envelope by April 12, 1985.

E. D. Arnold Jr.

E. D. Arnold, Jr.
Research Scientist

I. BACKGROUND INFORMATION

1. Jurisdiction ____________________

2. Name of person completing questionnaire ____________________

telephone no. ____________________

3. Do you have access to a computer? _____yes

_____no

If yes, what kind(s)?

_____Mainframe, brand ____________________

_____Mini, brand ____________________

_____Micro or personal, brand ____________________

II. SIGNAL CONTROLLERS AT ISOLATED INTERSECTIONS

Mode	Manufacturer	Model No.	No. Units
1. Pretimed	________	________	________
	________	________	________
	________	________	________
	________	________	________
	________	________	________
	________	________	________
	________	________	________

Mode	Manufacturer	Model No.	No. Units
2. Semi-Actuated			
3. Fully-Actuated			
4. Volume-Density			

III. SIGNAL CONTROLLERS IN A SYSTEM

System No. & Description	Manufacturer	Model No.	No. Units
System No. 1 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network ___Arterial	___	___	___
Type of controllers: ___Pretimed	___	___	___
___Semi-actuated	___	___	___
___Fully-actuated ___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units
System No. 2 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network ___Arterial	___	___	___
Type of controllers: ___Pretimed	___	___	___
___Semi-actuated	___	___	___
___Fully-actuated ___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units
System No. 3 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network ___Arterial	___	___	___
Type of controllers: ___Pretimed	___	___	___
___Semi-actuated	___	___	___
___Fully-actuated ___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units

System No. 4
(Check all applicable descriptors)

Type of system:
___Grid/network
___Arterial

Type of controllers:
___Pretimed
___Semi-actuated
___Fully-actuated
___Volume-density

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. 5
(Check all applicable descriptors)

Type of system:
___Grid/network
___Arterial

Type of controllers:
___Pretimed
___Semi-actuated
___Fully-actuated
___Volume-density

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. 6
(Check all applicable descriptors)

Type of system:
___Grid/network
___Arterial

Type of controllers:
___Pretimed
___Semi-actuated
___Fully-actuated
___Volume-density

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units
System No. 7 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network			
___Arterial	___	___	___
Type of controllers:	___	___	___
___Pretimed			
___Semi-actuated	___	___	___
___Fully-actuated			
___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units
System No. 8 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network			
___Arterial	___	___	___
Type of controllers:	___	___	___
___Pretimed			
___Semi-actuated	___	___	___
___Fully-actuated			
___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

System No. & Description	Manufacturer	Model No.	No. Units
System No. 9 (Check all applicable descriptors)	___	___	___
Type of system:	___	___	___
___Grid/network			
___Arterial	___	___	___
Type of controllers:	___	___	___
___Pretimed			
___Semi-actuated	___	___	___
___Fully-actuated			
___Volume-density	___	___	___

Type of interconnect:
___Hard wire
How coordinated?
___Street master controller
___Central computer
___Time-based

Comments

IV. AUXILIARY CONTROL EQUIPMENT (STAND ALONE)

Type	Manufacturer	Model No.	No. Units
1. Minor movement controller			
2. Advance green timer			
3. Pedestrian interval timer			
4. All-red timer			
5. Railroad preemption unit			

IV. AUXILIARY CONTROL EQUIPMENT (STAND ALONE) (CONT'D)

Type	Manufacturer	Model No.	No. Units
6. Fire preemption unit			
7. Bus preemption unit			
8. Time-based coordinator for wireless system			
9. Coordination unit for hard wire system			
10. Other (please list)			

IV. AUXILIARY CONTROL EQUIPMENT (STAND ALONE) (CONT'D)

Type	Manufacturer	Model No.	No. Units
10. Other (continued)			
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______
______	______	______	______

V. DETECTORS

Check the types you are using.

	No. (if known),	Otherwise Approximate % of Total
_____Inductive loop detector	______	______
_____Magnetic detector (standard)	______	______
_____Magnetometer	______	______
_____Radar detector	______	______
_____Sonic detector	______	______
_____Pressure sensitive detector	______	______
_____Other (please list)	______	______
______	______	______
______	______	______
______	______	______

THANK YOU

APPENDIX B

A TECHNIQUE FOR MEASUREMENT OF DELAY AT INTERSECTIONS

This appendix is excerpted from the user's manual which was developed as part of a research project conducted by JHK and Associates, "Definition and Measurement of Delay at Intersections." The project was conducted for FHWA under contract DOT-FH-11-8835. The user's manual presents an inexpensive and accurate method for measuring intersection performance.

The final report of the research project is in three volumes.

Volume No.	FHWA No.	Short Title
1	FHWA-RD-76-135	Technical Report
2	FHWA-RD-76-136	Data Summaries
3	FHWA-RD-76-137	User's Manual

Source: "Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution," U.S. DOT, FHWA, Office of Traffic Operations, Washington, D.C., May 1980.

1. INTRODUCTION

1.1 MEASURES OF PERFORMANCE

In traffic engineering work it often becomes necessary to report on the efficiency of operation of intersections controlled by traffic signals. The 1965 Highway Capacity Manual[1] describes intersection performance in terms of load factor and ratios of volume to service volume.

More direct and practical measures of intersection performance are vehicle delay and the percentage of vehicles having to stop. There are two fundamental reasons why delay and stops are good measures of intersection performance: motorists are keenly aware of and sensitive to the number of stops they are forced to make and to the length of time they are delayed; and, measures of stops and delay can readily be applied to estimates of road-user costs, fuel consumption, and environmental impacts of traffic flow.

1.2 SUMMARY OF THE METHODS

This manual contains complete instructions for the application of two methods which lead to estimates of vehicle delay and stops on approaches to signalized intersections. It is recommended that the two methods be applied simultaneously in the field, with a minimum of one observer used for each method.

The field method which yields an estimate of delay is termed the "Intersection Delay Study." This technique gives an estimate of the total stopped delay (see Section 2 for definition of terms), in vehicle-seconds, incurred by vehicles passing through an intersection. The study is based on a point sample of stopped vehicles. Its use in traffic engineering studies was originally developed and reported by Berry and Van Til.[2]

The field study of delay requires a one- or two-person team for each intersection approach and the duration of study will normally be from 13 to 30 minutes.

1 Highway Capacity Manual, Highway Research Board Special Report 87, Washington, D.C., 1965

2 "A Comparison of Three Methods for Measuring Delay at Intersections," Proceedings, California Street and Highway Conference, 1954

In performing the study the field team records the number of stopped vehicles on the approach at a given instant termed the "sampling point." After waiting a set interval of time, such as 13 seconds, the team again records the number of stopped vehicles. The sampling continues for the duration of the study period and the total stopped delay for all vehicles is computed as the product of the interval between samples, in seconds, and the sum of the number of vehicles included in the samples. This product is then multiplied by a modifying factor of 0.92[3] to yield an estimate of stopped delay.

The field method which gives a measure of stops and also an estimate of total volume is termed the "Percent Stopping" Study. This study leads to an estimate of the number of vehicles having to make at least one stop on the intersection approach, as a percentage of the total number of vehicles entering the intersection. The same study also gives an estimate of total volume.

As noted before, it is recommended that the Percent Stopping Study be conducted simultaneously with the Intersection Delay Study. One or two observers are assigned to the Percent Stopping Study for an intersection approach and they count all vehicles which cross the STOP line and move into the intersection during the period of study. Each vehicle is classified into one of two categories and is counted only once, regardless of the number of stops or amount of delay the vehicle may have suffered. The two categories are "stopping" and "not stopping." By summing the two categories an estimate of total volume is obtained and can be used with results from the Intersection Delay Study to put stopped delay or approach delay on a "per vehicle" basis.

3 Volume 1 of this report contains research findings which explain various delay types and their interrelationships. Specifically, the 0.92 and 0.96 modifying factors for converting field data from the Intersection Delay Study and the Percent Stopping Study, respectively, to accurate estimates of the true value for each measure are developed.

2. DEFINITIONS

Following are definitions for terms used in the Intersection Delay Study and the Percent Stopping Study.

Approach Delay - The total amount of time, in vehicle-seconds, lost by vehicles due to traffic conditions on the approach to a signalized intersection. For an individual vehicle, approach delay is the amount of time used to pass through the approach minus the amount of time used by an unimpeded vehicle moving at free flow speed to pass through the approach.

Approach Delay Per Vehicle - approach delay divided by the total number of vehicles passing through the intersection approach during a period of time, in vehicle-seconds per vehicle.

Interval Between Samples - the time, in seconds, between each successive point sample of stopped vehicles taken in the Intersection Delay Study.

Not Stopping - a vehicle which proceeds along the intersection approach and enters the intersection without coming to a stop.

Percent of Vehicles Stopping - the proportion of the approach volume expressed as a percent, which has stopped one or more times on the intersection approach.

Point Sample - a count of the number of vehicles stopped on the intersection approach or in designated lanes at a given instant in time.

Sampling Point - the instant in time at which a point sample is taken.

Stopped Time - the time, in vehicle-seconds, during which a vehicle is stopped with locked wheels on the intersection approach.

Stopped Delay - the total amount of stopped time, in vehicle-seconds, for all vehicles using an intersection approach during a given period of time.

Stopped Delay Per Vehicle - stopped delay divided by the total number of vehicles passing through the intersection approach during a period of time, in vehicle-seconds per vehicle.

Stopping - a vehicle which comes to a stop one or more times on the intersection approach.

3. INTERSECTION DELAY STUDY

3.1 STUDY OBJECTIVES

The principal objective of the Intersection Delay Study is to collect data on the approach to a signalized intersection such that an accurate estimate of approach delay per vehicle and stopped delay per vehicle can be made. The Percent Stopping Study (see Section 4 for description) must be taken simultaneously with the delay study in order to calculate these two measures of performance on a "per vehicle" basis.

3.2 STUDY REQUIREMENTS

A step-by-step approach should be followed in the design of an Intersection Delay Study. The following elements must be considered.

Select Time Period To Be Studied - for most applications a peak traffic period and an off-peak period should be studied to give a balanced view of intersection operation.

Select Length of Study Period - a minimum of 60 point samples should be taken for each study. This represents a 15- or 13-minute period, depending on the interval between samples used. If an entire intersection is to be studied, it is recommended that each approach be observed for 60 point samples, with the field crew moving from approach to approach until all have been studied. This procedure can be repeated to obtain an additional 60 point samples on each approach if time permits. It is recommended that lengths of studies be either 60, 90, or 120 point samples.

Determine Interval Between Samples[4] - if a signal is operating in a pretimed or system mode, use a 13-second interval for cycle lengths of 45, 60, 75, 90, 105, 120, 135, or 150 seconds. For all other cycle lengths in a pretimed or system mode, use a 15-second interval between samples. For all traffic actuated signals not operating in a system, use a 15-second interval.

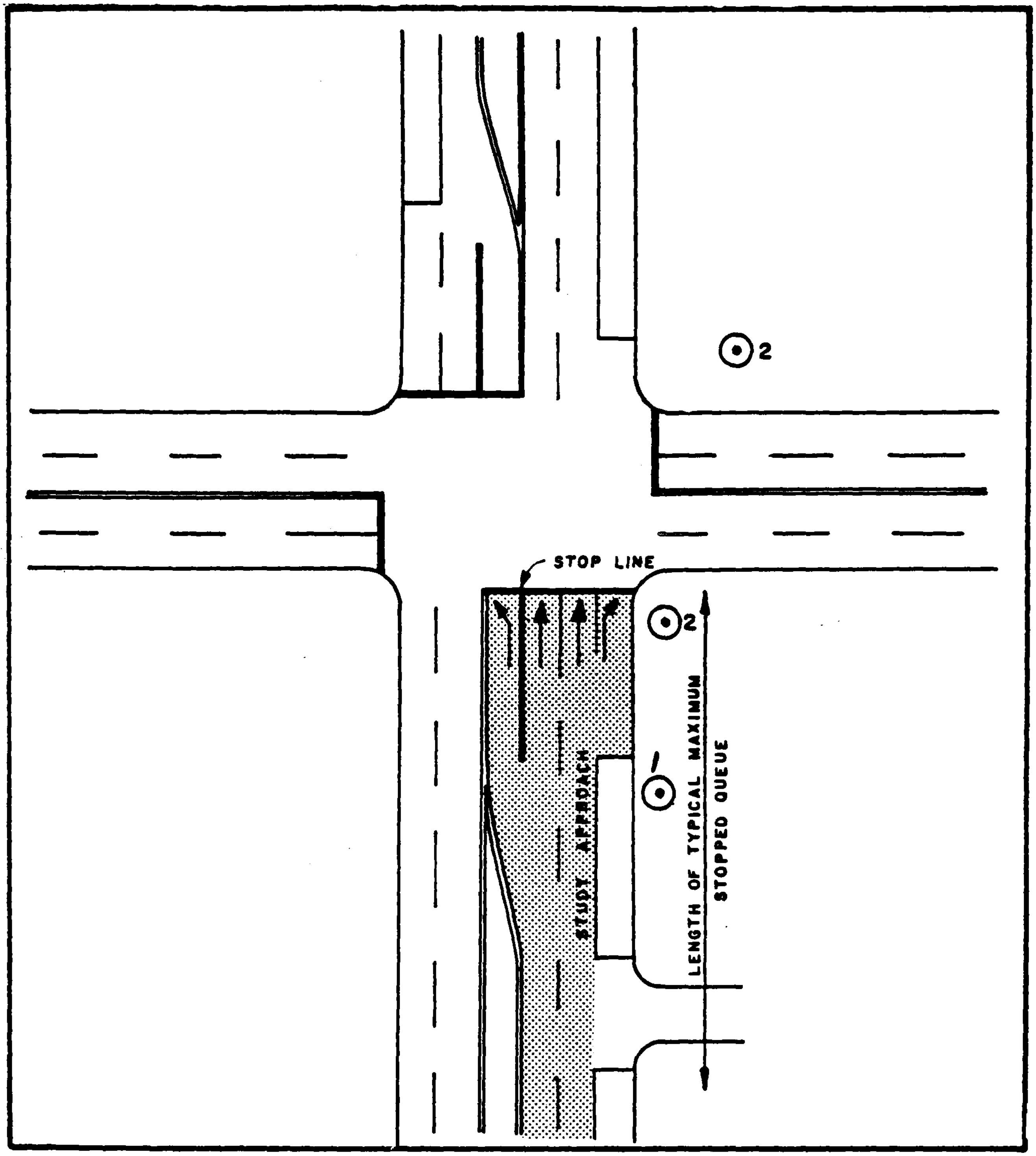
Select Observation Point - Usually the best location is on the right-hand side of the approach, in the shoulder or sidewalk area. However, if the site is hilly, other locations may be better. Figure B.1 shows possible locations. If a vehicle is used it must be positioned so as not to be conspicuous or hazardous to traffic using the intersection. Rooftops or buildings offer good locations.

---

4 For traffic signals operating on a fixed cycle length, the interval between samples should not be an even divisor of the cycle length. This restriction is not important when the cycle length is greater than 150 seconds.

FIGURE B.1

LOCATION OF FIELD OBSERVATION POINTS



Legend: 1 = Recommended observation point for Intersection Delay Study, midway along length of average maximum stopped queue to be observed.

2 = Preferred observation points for Percent Stopping Study.

4. PERCENT STOPPING STUDY

4.1 STUDY OBJECTIVES

The objectives of the Percent Stopping Study are: to develop an estimate of the "percent of vehicles stopping" on approaches to signalized intersections; and, to develop an estimate of total volume on these approaches. The volume estimate is used with values derived from the Intersection Delay Study (see Section 3) to report delay on a "per vehicle" basis.

4.2 STUDY REQUIREMENTS

Because the Percent Stopping Study will almost always be performed in conjunction with the Intersection Delay Study, much of the study design will be accomplished as part of the delay study.

Hand Counters - each percent stopping observer may be equipped with two hand counters. The counter is used to register two categories of count: stopping and not stopping. If such counters are not available, the observers simply use tally marks to record the count on the field data sheet.

Timing Device for Sampling Points (1 per team) - it is recommended that a small battery-powered cassette recorder or other audio device be used to provide an audible cue at each sampling point. The tape should start with the word "begin" to signify the zero point of study. Then, a cue (the word "now" is suggested) is given at each sampling point.

Other Equipment - each team member needs a clipboard, pencils, and enough data sheets for the periods to be studied. Each data sheet accommodates 120 point samples. A blank sheet is found at the end of this Appendix.

5. DATA REDUCTION

In the office, a data reduction form is filled out for each study period. This form, an example of which is given as Figure B.4, contains space for reduction of data from both the Intersection Delay Study and the Percent Stopping Study. A blank data reduction form is found at the end of this Appendix.

Step 1 - the values for stopping and not stopping are taken from the percent stopping study field data sheets (Figure B.3) and entered on lines i and ii of Figure B.4. If two observers were used to perform the Percent Stopping Study, the sum of the values from both observers is used. Lines i and ii are summed to give a value for total volume (line iii, Figure B.4).

Step 2 - the value for "stopping is divided by total volume to give a measure of percent stopping (line iv).

Step 3 - line iv is multiplied by 0.96 (see footnote 3, page   for comment on this factor) to be an estimate of percent of vehicles stopping (line v).

Step 4 - the total volume on line iii is entered on line   of the data reduction form so that delay can be computed on a "per vehicle" basis.

Step 5 - using data from the field data sheet for the intersection delay study (Figure B.2), lines 1 and 5 are filled in on the data reduction form. If two observers were used for the Intersection Delay Study, it will be necessary to add the values from each of their field sheets to arrive at a total for the entire study approach.

Step 6 - if one or more point samples are missed in the field, a correcting procedure is used (lines a through f, middle section of the data reduction form). The average value for all samples taken during each period of 30 samples is used as the estimate for any missing values during that same period, and is entered on line 6.

Step 7 - lines 7 through 13 are completed as per instructions on the data reduction form itself.

FIGURE B.2

INTERSECTION DELAY STUDY, FIELD DATA SHEET

INTERSECTION DELAY STUDY

POINT SAMPLE, STOPPED DELAY METHOD

Intersection *Tucson Blvd/22nd St.* Study Traffic On *Tucson Blvd.*

City and State *Tucson, Az.* Agency *City of Tucson, Traffic Engineering Div.*

Day, Date *Mon., Aug. 2, 1976* Study Period *1340-1353* Observer *L. Wiles*

Traffic Approaching From *N* (N,S,E,W) Weather *Clear and Hot*

If more than one person is studying same approach, explain division of responsibilities. ______

INTERVAL BETWEEN SAMPLES = *13* SECS.

START	2	6	7	4	0	0	0	1	4	6
	0	0	0	0	2	7	11	6	1	0
	0	0	0	3	4	6	2	0	0	0 (30)
	1	7	13	12	4	1	0	0	4	9
	11	8	2	0	0	0	3	7	7	4
	0	0	0	4	11	4	1	0	2	(60)
										(90)
										(120)

NOTE: ONE SAMPLE MISSING

OBSERVED TOTAL, ALL SAMPLES *187*

(30) DENOTES 30TH SAMPLE

COMMENTS: *At last sample, stopwatch reading was 13 minutes, 02 seconds.*
One sample missed in second group of 30 samples.

FIGURE B.3

PERCENT STOPPING STUDY, FIELD DATA SHEET

PERCENT STOPPING STUDY

Intersection Tucson Blvd/22nd St. Study Traffic On Tucson Blvd.

City and State Tucson, Az. Agency City of Tucson, Traffic Engineering Div.

Day, Date Mon. Aug. 2, 1976 Study Period 1340-1353 Observer L. Burke

Traffic Approaching From N (N,E,S,W) Weather Hot

If more than one person is studying same approach, explain division of responsibilities.

STOPPING *

~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~
~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~
~~||||~~ ~~||||~~
~~||||~~ ~~||||~~
~~||||~~ ||||

NOT STOPPING *

~~||||~~ ~~||||~~ ~~||||~~

* IF TALLY MARK IS USED, ~~||||~~ DENOTES A COUNT OF "5"

TOTAL STOPPING 79

TOTAL NOT STOPPING 15

COMMENTS:

FIGURE B.4

DATA REDUCTION FORM

DATA REDUCTION FORM

INTERSECTION DELAY AND PERCENT STOPPING STUDIES

Intersection TUCSON BLVD/22ND ST. City & State TUCSON, ARIZONA

Study Approach On TUCSON BLVD Traffic From N (N, S, E, W)

Day, Date, Time MON., AUG. 2, 1976 1340-1353

PERCENT STOPPING STUDY

(i)	Total no. of vehicles "stopping"	79 vehs.
(ii)	Total no. of vehicles "not stopping"	15 vehs.
(iii)	Total volume = (i) + (ii)	94 vehs.
(iv)	Observed Percent Stopping = [(i) ÷ (iii)] x 100	84 %
(v)	Actual Percent of Vehicles Stopping = (iv) x 0.96[5]	81 %

CORRECTION PROCEDURE FOR MISSED SAMPLES IN DELAY STUDY

		Corr.* No. 1	Corr.* No. 2
(a)	Total no. of point samples taken in field during 30-sample period	29	
(b)	30 - (a)	1	
(c)	Sum of point sample values for 30-sample period on field data sheet	115	
(d)	Value of each missing sample = (c) ÷ (a), round to nearest whole number	4	
(e)	Total value for all missing samples in 30-sample period = (b) x (d)	4	
(f)	Total value for all missing samples in study period = sum of (e) for all corrections		4

* Use one correction factor for each 30-sample period in which the field data sheet has one or more missing values.

INTERSECTION DELAY STUDY

(1)	Total no. of point samples taken in field	59
(2)	Total no. of point samples missed, from (b) above	1
(3)	Total no. of point samples used in calculations = (1) + (2)	60
(4)	Interval between samples	13 secs.
(5)	Sum of observed point sample values	187 vehs.
(6)	Sum of calculated "corrected" point sample values, from (f), above	4 vehs.
(7)	Sum of all point sample values = (5) + (6)	191 vehs.
(8)	Total Stopped Time = (4) x (7)	2483 veh.-secs.
(9)	Stopped Delay = (8) x 0.92[5]	2284 veh.-secs.
(10)	Approach Delay = (9) x 1.3[5]	2969 veh.-secs.
(11)	Total Volume = (iii)	94 vehs.
(12)	Stopped Delay Per Vehicle = (9) ÷ (11)	24 veh.-secs./veh.
(13)	Approach Delay Per Vehicle = (10) ÷ (11)	32 veh.-secs./veh.

5 See footnote 3, page 2 of this manual for comment on these modifying factors.

6. PRESENTATION OF RESULTS

The measures which can be estimated from the Intersection Delay Study and the Percent Stopping Study are (note that "line" numbers refer to data reduction form):

- Percent Stopped (line v)
- Stopped Delay, in vehicle-seconds (line 9)
- Approach Delay, in vehicle-seconds (line 10)
- Stopped Delay Per Vehicle, in vehicle-seconds per vehicle (line 12)
- Approach Delay Per Vehicle, in vehicle-seconds per vehicle (line 13)

The latter two measures require a volume count for their computation. This volume count will normally be obtained by using the Percent Stopping Study.

In presenting results, an explicit identification of the delay type is essential and the above mentioned terms, rather than the vague term "delay", should be used.

INTERSECTION DELAY STUDY

POINT SAMPLE, STOPPED DELAY METHOD

Intersection ____________________ Study Traffic On ____________________

City and State ____________________ Agency ____________________

Day, Date ______________ Study Period ______________ Observer ______________

Traffic Approaching From __________ Weather ____________________
N,S,E,W

If more than one person is studying same approach, explain division of responsibilities. ____________________

INTERVAL BETWEEN SAMPLES = ____ SECS.

START →

									30
									60
									90
									120

OBSERVED TOTAL, ALL SAMPLES ______

30 DENOTES 30TH SAMPLE

COMMENTS: ____________________

PERCENT STOPPING STUDY

Intersection ____________________ Study Traffic On ____________________

City and State ____________________ Agency ____________________

Day, Date ______________ Study Period ______________ Observer ______________

Traffic Approaching From __________ Weather ____________________
N,E,S,W

If more than one person is studying same approach, explain division of responsibilities. ____________________

STOPPING *	NOT STOPPING *

* IF TALLY MARK IS USED , ~~THL~~ DENOTES A COUNT OF "5"

TOTAL STOPPING ___ TOTAL NOT STOPPING ___

COMMENTS : ____________________

DATA REDUCTION FORM

INTERSECTION DELAY AND PERCENT STOPPING STUDIES

Intersection ______________________ City & State ______________________

Study Approach On ______________________ Traffic From ______________________
N, S, E, W

Day, Date, Time ______________________

PERCENT STOPPING STUDY

(i) Total no. of vehicles "stopping" . . . . ________ vehs.

(ii) Total no. of vehicles "not stopping" . . . . ________ vehs.

(iii) Total volume = (i) + (ii) . . . . ________ vehs.

(iv) Observed Percent Stopping = [(i) ÷ (iii)] x 100 . . . . ________

(v) Actual Percent of Vehicles Stopping = (iv) x 0.96 . . . . ________

CORRECTION PROCEDURE FOR MISSED SAMPLES IN DELAY STUDY

	Corr. * No. 1	Corr. * No. 2
(a) Total no. of point samples taken in field during 30-sample period	____	____
(b) 30 - (a)	____	____
(c) Sum of point sample values for 30-sample period on field data sheet	____	____
(d) Value of each missing sample = (c) ÷ (a), round to nearest whole number	____	____
(e) Total value for all missing samples in 30-sample period = (b) x (d)	____	____

(f) Total value for all missing samples in study period = sum of (e) for all corrections . . . . ____

* Use one correction factor for <u>each</u> 30-sample period in which the field data sheet has one or more missing values.

INTERSECTION DELAY STUDY

(1) Total no. of point samples taken in field . . . . ________

(2) Total no. of point samples missed, from (b) above . . . . ________

(3) Total no. of point samples used in calculations = (1) + (2) . . . . ________

(4) Interval between samples . . . . ________ secs.

(5) Sum of observed point sample values . . . . ________ vehs.

(6) Sum of calculated "corrected" point sample values, from (f), above . . . . ________ vehs.

(7) Sum of all point sample values = (5) + (6) . . . . ________ vehs.

(8) Total Stopped Time = (4) x (7) . . . . ________ veh.-secs.

(9) Stopped Delay = (8) x 0.92 . . . . ________ veh.-secs.

(10) Approach Delay = (9) x 1.3 . . . . ________ veh.-secs.

(11) Total Volume = (iii) . . . . ________ vehs.

(12) Stopped Delay Per Vehicle = (9) ÷ (11) . . . . ________ veh.-secs./veh

(13) Approach Delay Per Vehicle = (10) ÷ (11) . . . . ________ veh.-secs./veh

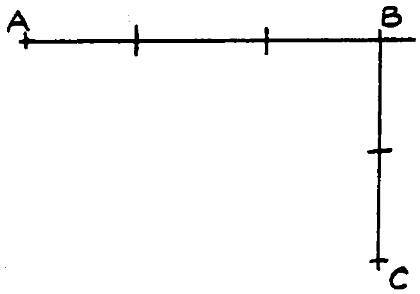
APPENDIX C

CALCULATIONS FOR NETWORK COORDINATION

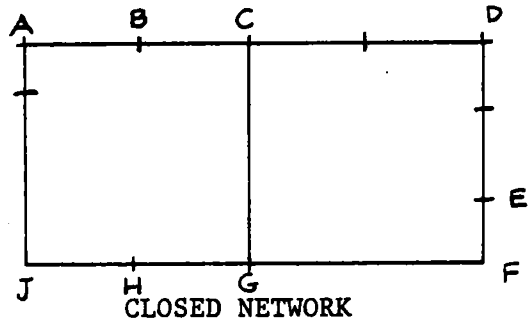
Source: Reference 12.

CALCULATIONS FOR NETWORK COORDINATION

1. When two streets having signal systems intersect, we have a "signal network." This may be an "open network," or a "closed network."



OPEN NETWORK

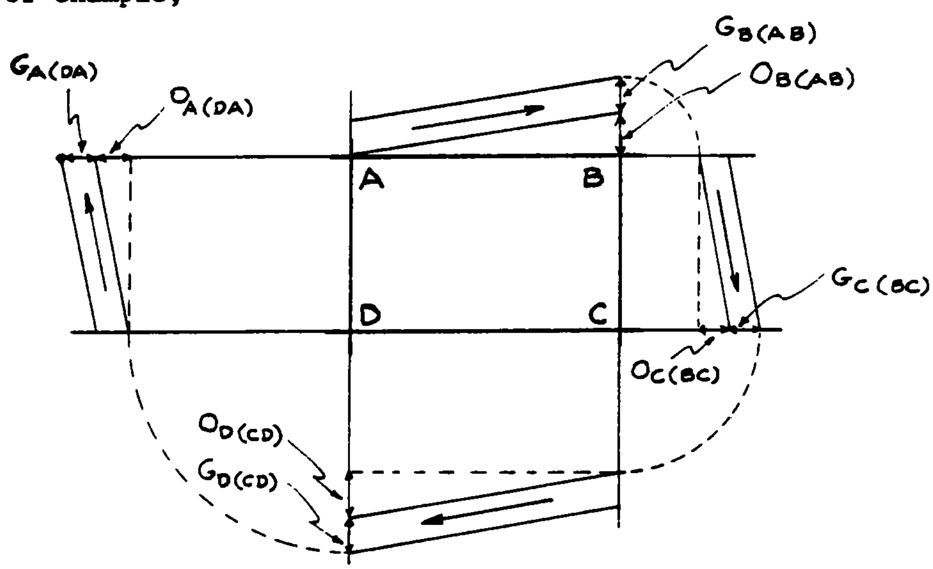


CLOSED NETWORK

2. In a closed network, the timing in each direction on each street must be studied to maintain some progression in each direction, if possible.

3. Time-space diagrams should be developed for each street separately in order to determine desirable offsets at each intersection along a street.

4. Since it is difficult to show time-space diagrams for a network of signal systems, it is possible to use line drawings of the street network, with notations at each intersection of the length of "green plus yellow" for each through movement, and the time offset in relation to a time reference point. For example,



5. In order for this example network to be coordinated,

$$O_{B(AB)} + G_{B(AB)} + O_{C(BC)} + G_{C(BC)} + O_{D(CD)} + G_{D(CD)} + O_{A(DA)} + G_{A(DA)}$$

must be equal to NC

where $O_{B(AB)}$ = offset in seconds of the green at intersection B, along street AB.

This offset is measured from the start of green at the first intersection (A) of the network.

$G_{B(AB)}$ = green (plus yellow) time in seconds at intersection B, along street AB.

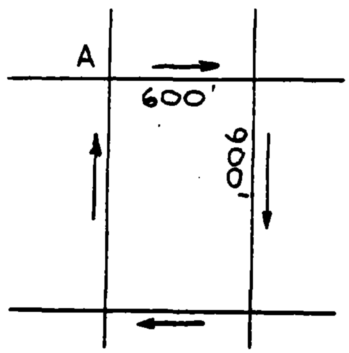
C = length of cycle in seconds

N = a whole number.

Since $G_{B(AB)} + G_{C(BC)} + G_{D(CD)} + G_{A(DA)}$ must be equal to two cycles = 2C,

it is seen that $O_{B(AB)} + O_{C(BC)} + O_{D(CD)} + O_{A(DA)}$ = NC-2C = C(N-2) or $C = \frac{\Sigma \text{ Offsets}}{N-2}$

6. Example:



Desired speed = 20 mph = 30 fps
Cycle split = 50-50

Required: calculate cycle time and offsets

(1) Calculate offsets:

$O_{B(AB)} = \frac{600}{30} = 20$ sec.

$O_{C(BC)} = \frac{900}{30} = 30$ sec.

$O_{D(CD)} =$ 20 sec.

$O_{A(DA)} =$ 30 sec.

Σ Offsets = 100 sec.

(2) Determine trial cycle length:

$$C = \frac{\Sigma \text{ offsets}}{N - 2} = \frac{100}{N-2}$$

If	N = 3	C = 100	Very high
	N = 4	C = 50	
	N = 5	C = 33.3	Very low

Try C = 50

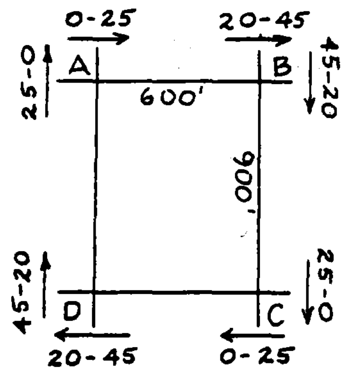
(3) Check equation, and adjust offsets if necessary:

$$\Sigma \text{ offsets} + 2C = NC$$

$$100 + 2(50) = 4(50)$$

$$200 = 200$$

(4) No adjustment is necessary since system is balanced. The following line diagram illustrates the offset and green time notations which balance this network:



0	
+20	offset AB
=20	start of gr. at B
+25	green at B
=45	end of gr. at B
+30	offset BC
=75	start of gr. at C
-50	= C
=25	

The notations at each intersection are called "designators." The first number is the offset from the previous intersection; the second is the time at the end of the green. If any number exceeds a cycle length, C is subtracted from it to keep the numbers small; for example, at C the start of green is at 75 seconds as measured from start of green at A; however, 75-50 = 25 is used at C as the first designator.

This problem indicated that we can have our desired speed of 20 mph: 30 fps provided we are satisfied to have a cycle length of 33 or 50 or 100 seconds.

7. Suppose, however, that it is necessary to use a 70-second cycle at these intersections because of capacity requirements or because these streets must coordinate with other intersections operating at a 70-second cycle. In this case it is necessary to adjust the offsets in order that the network will still balance; this adjustment will in turn change the speed of progression.

C = 70 seconds; cycle split still is 50-50; desired speed still is 30 fps.
G = 35, R = 35 secs.

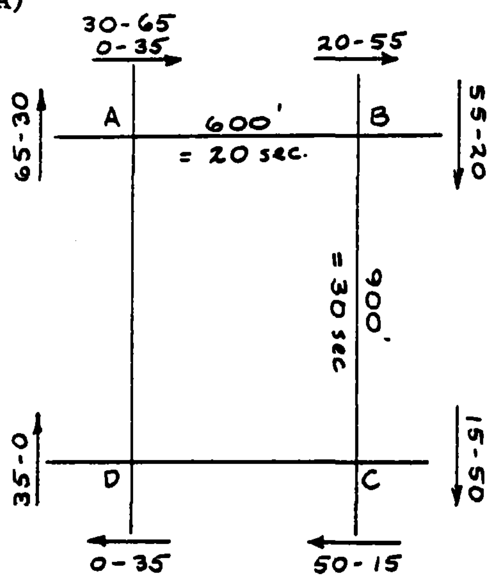
Calculate offsets desired; they are the same as before because speed desired has not changed:

$O_{B(AB)} = 20$

$O_{C(BC)} = 30$

$O_{D(CD)} = 20$

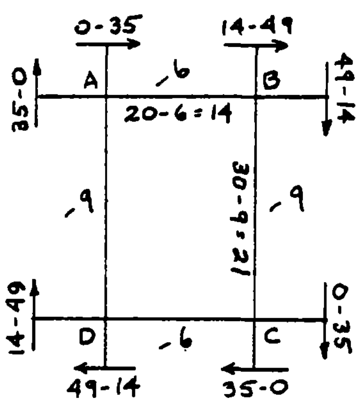
$O_{A(DA)} = 30$



0		
+20	off	
20		
+35	G	
55		55
+30	off	+35
85		90
-70	= C	-70
15		20
35	G	
50		
+20	off	
70		
-70	= C	
0		
+35	G	
35		
+30	off	
65		
35	G	
100		
-70	= C	
30		

The final designator of 30-65 means that our offsets have given a start of green at A, along AB, that is 30 seconds too late (or 40 seconds too soon). We choose the smaller of the two, and adjust our offsets to move traffic around the network 30 seconds faster:

Adjusted offsets:



0			
14	off		
14			
35	G		
49		49	
21	off	35	
70		84	
-70	= C	-70	= C
0		14	
35	G		
35		14	
14	off	21	off
49		35	
35	G	35	G
84		70	
-70	= C	-70	
14		0	

Check new speeds:

AB and CD: $\frac{600 \text{ ft.}}{14 \text{ sec.}} = 43 \text{ fps} = 29 \text{ mph}$

BC and DA: $\frac{900 \text{ ft.}}{21 \text{ sec.}} = 43 \text{ fps} = 29 \text{ mph}$

So a required C = 70 will result in speeds higher than the desired 20 mph.