EVALUATION OF SIGNAL TIMING AND COORDINATION PROCEDURES

Volume II: Field Manual

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. This Field Manual has been prepared to provide a concise and easily applied set of procedures. Detailed theory and logic behind the procedures are provided in a companion document entitled 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 (a)	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

EVALUATION OF SIGNAL TIMING AND COORDINATION PROCEDURES

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INTRODUCTION

This Field Manual is a summary of the contents of a much more comprehensive Technical Report bearing the same main title. It provides simplified, step-by-step procedures for timing pretimed and actuated signals at isolated intersections and general procedures for timing signal systems. Targeted users are field technicians and those responsible for maintaining signals in small cities and towns. For the most part, the Field Manual can be used without the companion Technical Report, which presents underlying theory and detailed discussions concerning signal timing.

General instructions regarding the use of these procedures are given below. These should be reviewed prior to undertaking the timing procedures.

1. THE TIMING VALUES DETERMINED FROM THESE PROCEDURES SHOULD BE CONSIDERED ONLY AS STARTING POINTS AT THE INTERSECTION. FIELD REVIEW OF THE RESULTING OPERATION AT THE INTERSECTION AND FINAL ADJUSTMENT OF THE SETTINGS ARE HIGHLY RECOMMENDED.

2. Due to the variety of signal manufacturers and equipment, some of the specific terms or labels on the controllers may differ from those in this report. In this case, the manufacturer's literature for the controller being timed should be reviewed closely to determine the relation among the terms being used. This is especially true for signal systems. Only the basic timing parameters are presented for systems, and the literature for the particular equipment being used must be reviewed to determine how to set some of these values.

3. The procedures for the actual setting of the timing values on the controller vary considerably. They can range from the placement of timing keys on the dial drum in the case of the old electro-mechanical controllers to keyboard data entry in the case of modern microprocessor controllers. Likewise, the precision of the timing value entered in the controller varies. For example, phase

lengths are set to the nearest percentage of the cycle length on the old electromechanical controllers, whereas the values on the modern equipment can be set to the nearest tenth of a second. Accordingly, the initial timing values developed from these procedures may need to be modified to reflect the limitations of the equipment being timed.

4. The procedures presented generally represent typical intersections and traffic volumes. If problems occur when using them, the user should refer to the Technical Report for a detailed discussion of the procedures. It is hoped that an understanding of the theory would enable resolution of the problems and continued use of the procedures. The Technical Report follows the same format as the Field Manual.

5. It is intended that the procedures be used primarily to evaluate the timing at existing signalized intersections; however, the procedures are generally applicable at the design stage.

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.

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.

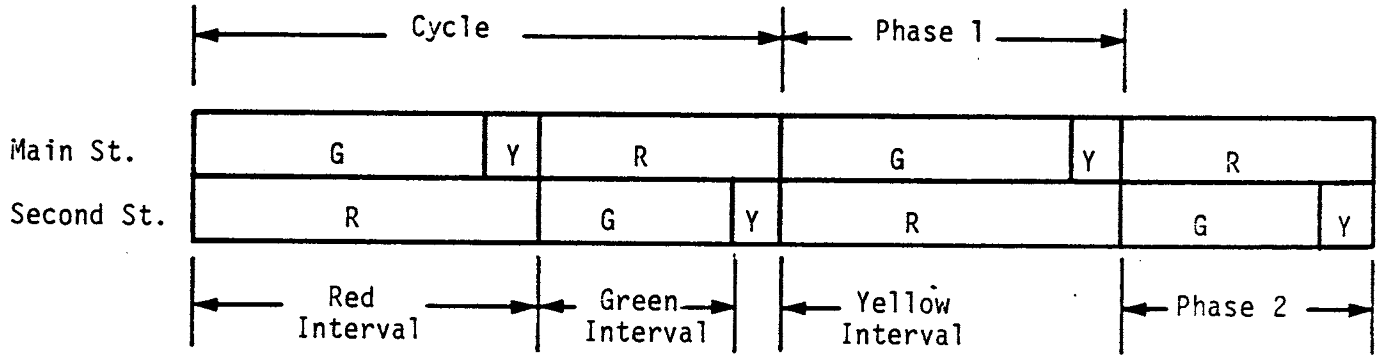


Figure 1. Timing sequence for simple two-phase controller.

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.

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. 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

I. Determine Number of Timing Plans

A different timing plan is needed for each significantly different pattern of traffic flow. 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 conditions -- inbound peak and outbound peak. Plans may be developed for both the a.m. and p.m. peak periods, an average or midday period, a late night or low-volume period, a weekend period, a shopping period, an evening period, and a special function period. The number of timing plans is frequently determined by the limitations of the controller coupled with local knowledge of the traffic conditions. It may be necessary to collect the intersection information described below in II. 1. and II. 2. in conjunction with this step.

II. Collect Necessary Intersection Information

1. Control equipment already at the intersection -- timing functions, limitations of controllers, timing values, timing plans, and phases.

2. Physical data -- number of approaches; number of lanes and type of flow (through, right turn, left turn, or combination) per lane per approach; width of lanes and medians; percent grade on approaches, if severe; speed limits; and location of parking, crosswalks, stop bars, bus stops, loading zones, etc.

3. Traffic and pedestrian data for each timing plan -- hourly traffic volume and pedestrian counts for each approach to the intersection; number of vehicles going straight and turning right or left on each approach; number of buses and large trucks on each approach, and average speed of traffic on each approach.

   Typical data collection and summary forms are shown in Figures 2, 3, and 4. Volumes for the peak morning and afternoon hours should be determined by analyzing either one-hour or 15-minute volume counts. Counts to determine average weekday volumes should be made on a Tuesday, Wednesday, or Thursday.

DIRECTIONAL TRAFFIC MOVEMENT – INTERSECTION OF ROUTES ____________________

COUNTY ____________________ LOCATION ____________________

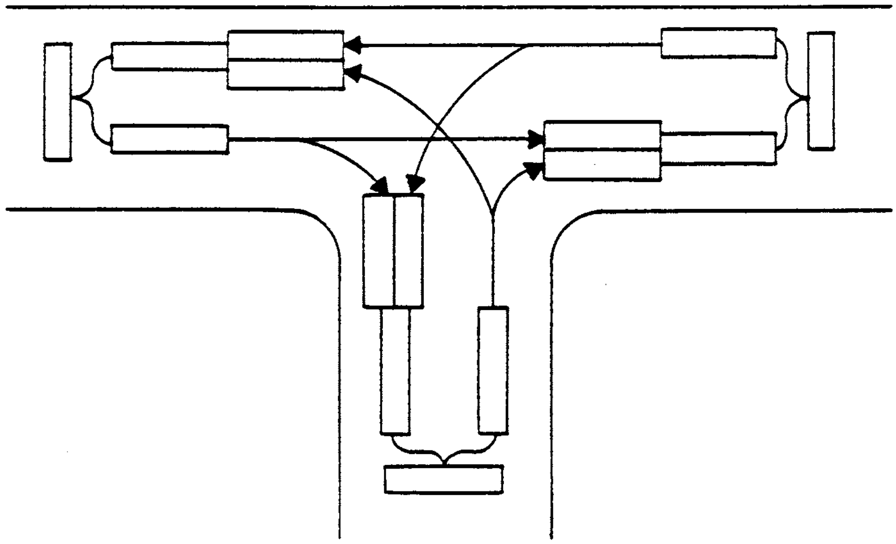
DATE ____ / ____ / ____ WEATHER ____________________

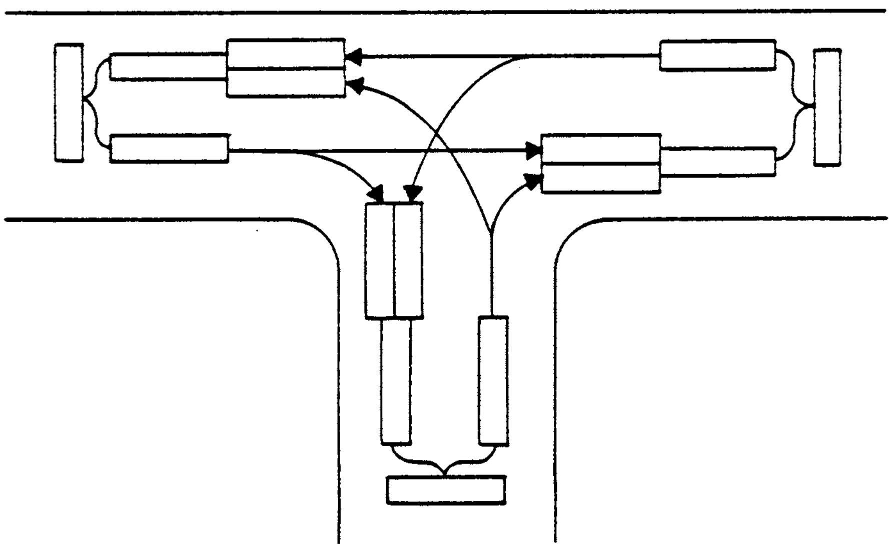
	APPROACHING INTERSECTION																
	ON ROUTE						FROM THE		ON ROUTE						FROM THE		
	EAST				WEST				NORTH				SOUTH				
HOURS	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	TOTAL
6:00 - 7:00																	
7:00 - 7:30																	
7:30 - 8:00																	
8:00 - 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 ____________________

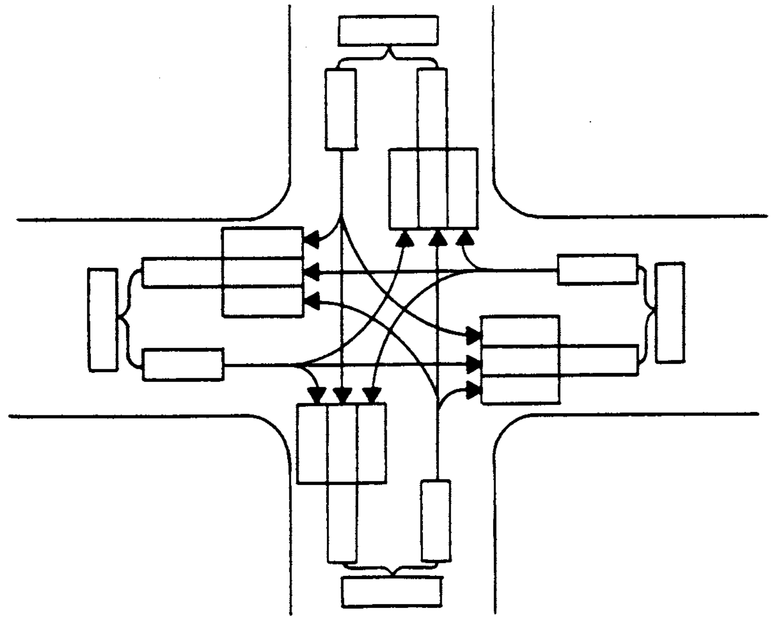
Figure 2. Typical data collection form.

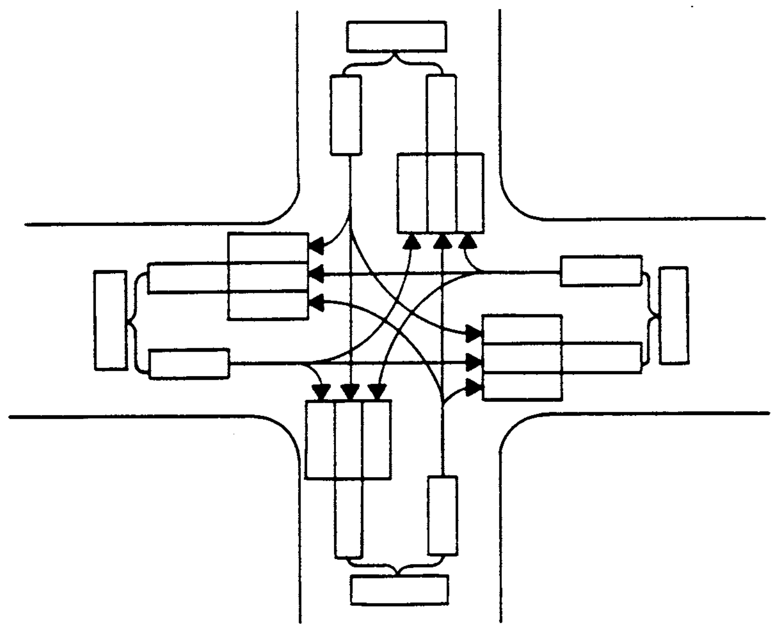




EIGHT MAXIMUM HOUR VOLUMES OF APPROACH VEHICLES									
TIME		RTE.		RTE.		RTE.		TOTAL	
FROM	TO	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.	VEH.	PED. CROSS.
TOTAL									

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.

III. Determine Number of Phases

As a general rule, the number of phases should be kept to a minimum. The number of phases needed is most often a left-turn issue; that is, the volume of left-turning vehicles and the volume of opposing vehicles are such that a separate phase in the cycle is needed to accommodate vehicles turning left. A separate lane providing left-turn storage may alleviate the need for a separate phase. The following guidelines, coupled with engineering judgement, may be used to determine the need for left-turn phases at intersections already having separate left-turn lanes.(1)

1. Volumes -- consider left-turn phasing on an approach when, during the peak hour, the product of the left-turn volume and opposing volume divided by the number of lanes 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. Appendix B in the Technical Report provides 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 5 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

   $R_c = 32.6 + 1.645\sqrt{32.6/M} - 0.5\,M$, 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.

IV. Calculate Passenger Car Equivalents (PCEs)

The use of PCEs accounts for the negative impacts of trucks, buses, and turning vehicles on the traffic handling capability of an intersection. For each approach, the PCE factor for the type of vehicle or traffic movement should be multiplied by the respective volume of the type of vehicle or traffic movement to obtain the passenger car equivalent. The PCE factors are shown in Table 1. By definition, passenger cars, motorcycles, and pickups have a factor of 1.0. Likewise, turning movements having separate phases, or no opposing conflicts, have a factor of 1.0.

Table 1

Passenger Car Equivalents (PCEs) 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 (or more than 10% right turns)	1.25

V. Find Critical Lane Volumes (CLV)

The critical or highest lane volume in PCEs per hour occurring during each phase must be determined. The CLVs per phase at the intersection must be summed to derive the total CLV for the intersection. If enough green time is provided to handle the traffic on the lane having the highest volume in a phase, then automatically sufficient green time exists to accommodate traffic on the other lanes. The following general rules apply to calculating lane volumes.

1. Right turns and left turns which have been adjusted to PCEs as appropriate are added to the through movement volume, unless there is an exclusive turning lane for the movement.

2. If there is an exclusive left-turn or right-turn lane without separate phasing, the adjusted left-turn or right-turn volumes are considered as separate lane volumes for a determination of the CLV.

3. If there is separate phasing provided for right-turn or left-turn movements, the PCE factor is 1.0 and the volumes are considered as separate lane volumes for a determination of the CLV.

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

VI. Determine Optimum Cycle Length

The optimum cycle length is determined from the graph in Figure 5. Enter the horizontal axis with the total CLV for the intersection as determined previously, read upward to the appropriate phase line, and read across to the vertical axis for the optimum cycle length in seconds. Cycle lengths should generally fall between 40 and 120 seconds.

VII. Calculate Cycle Splits

The amount of time within the optimum cycle that should be allocated to each phase is determined from Figure 6, 7, or 8, depending on the number of phases in the cycle. For each phase, calculate the ratio of the CLV for the phase to the CLV for the intersection. Then, enter the horizontal axis on the figure representing the correct number of phases at the intersection with the ratio for each phase, read upward to the line for the optimum cycle length in seconds determined previously, and read across to the number of seconds that should be allocated to that phase. This time includes the green and phase change intervals, and the total for all the phases should equal the previously determined optimum cycle length. The actual green time is calculated by subtracting the phase change interval determined in the next step.

VIII. 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 assignment of right-of-way; that is, the commencement of a red interval on their approach. The phase change interval should be long enough to allow a vehicle to decelerate to a safe stop or to proceed safely through the intersection prior to the red indication. The yellow interval is set to allow a safe stop whereas the red interval is set to allow a safe passage through the intersection. It is important that motorists have a reasonable expectation of the length of the yellow interval; therefore, it should always be between 3.0 and 5.0 seconds. The phase change interval is determined by entering Table 2 with the average approach speed and reading the time for the yellow change interval and for the total clearance needed. The all-red interval is the difference between the needed clearance and the yellow interval. If there is a significant grade on an approach, Equation 5 in the Technical Report should be used to calculate the phase change interval. An all-red interval is normally not used on an exclusive left-turn phase, except in the case of wide medians or high-speed approaches.

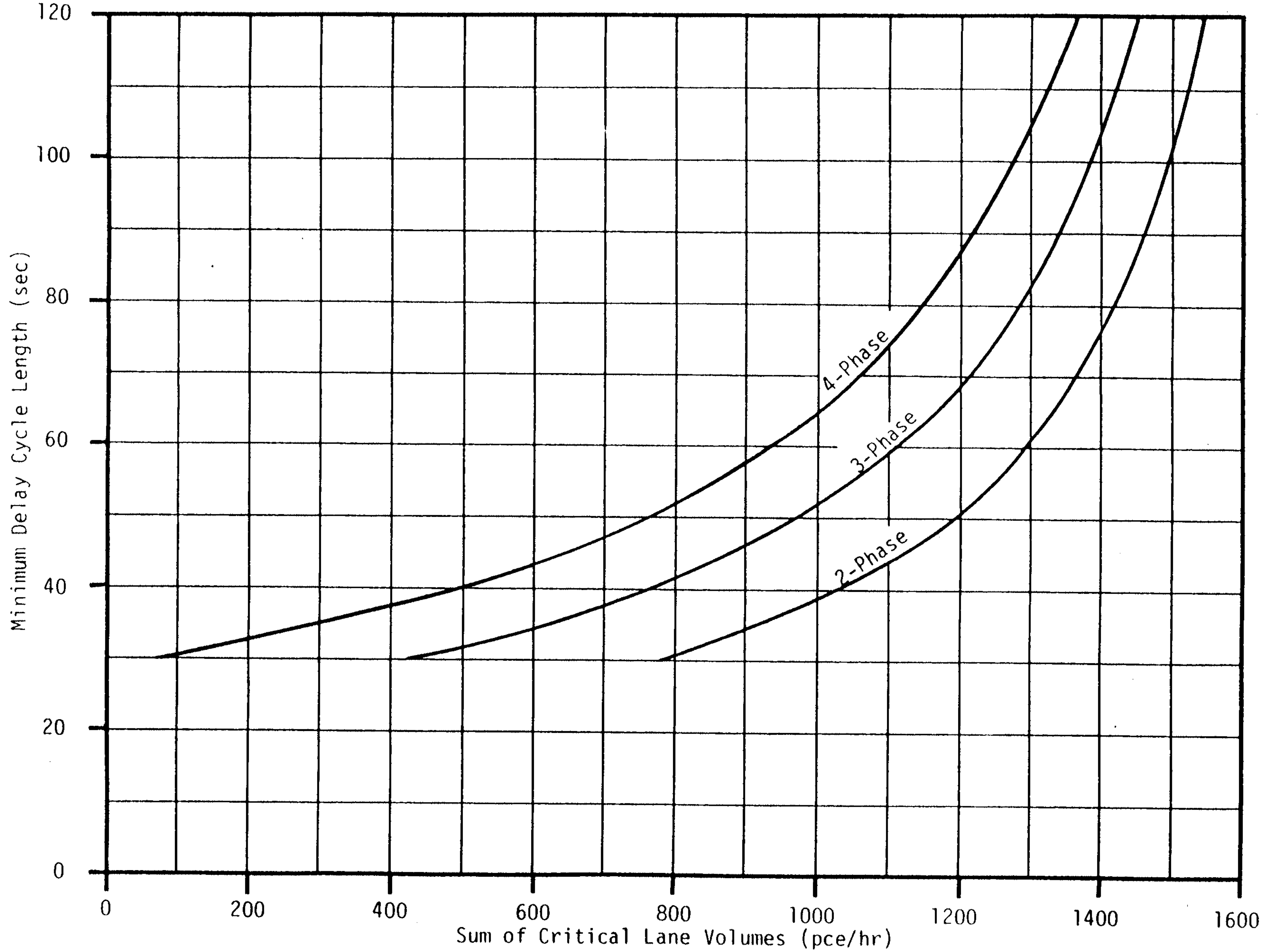


Figure 5. Optimum cycle length for pretimed control.
Source: Reference 2.

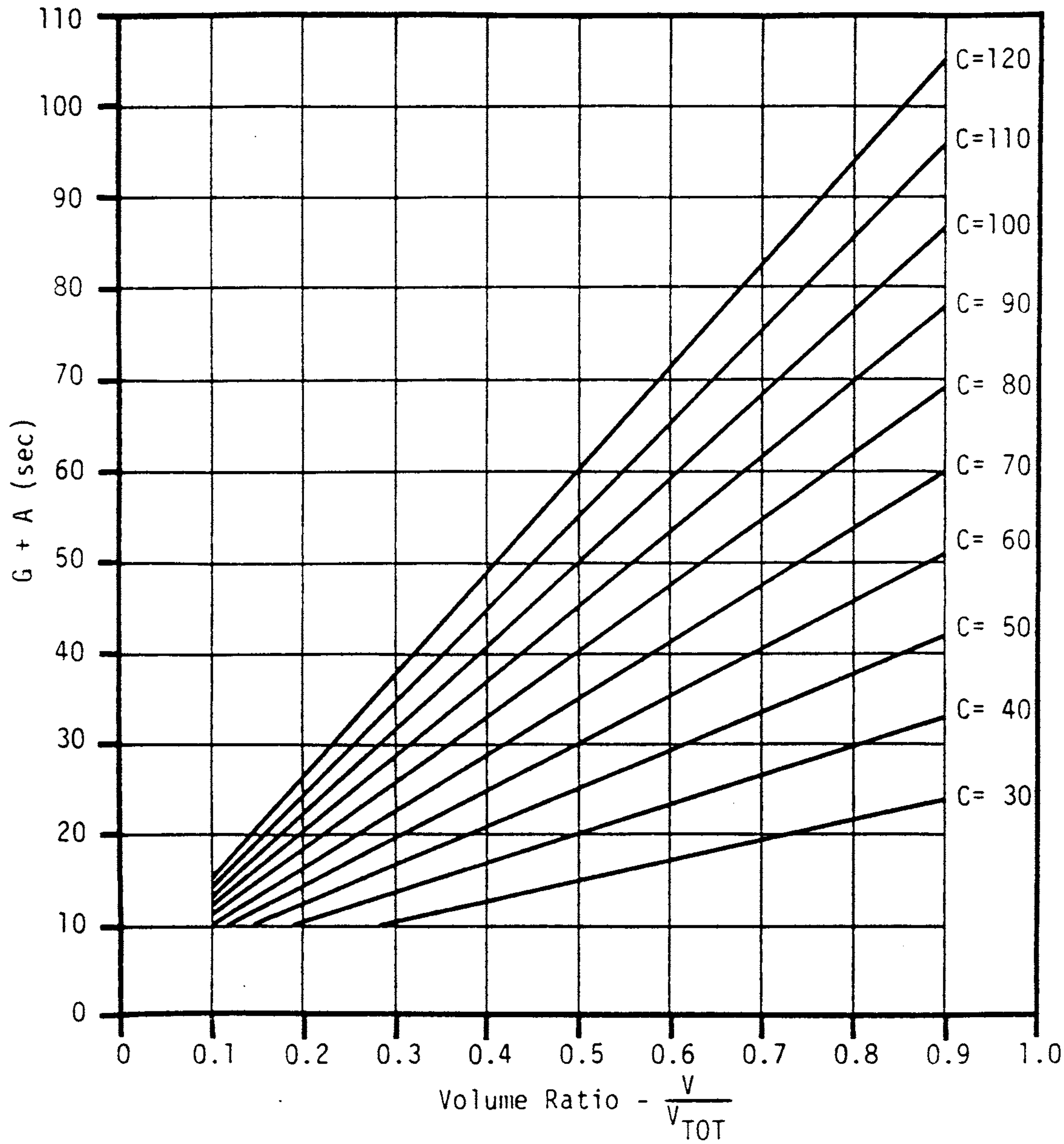


Figure 6. Cycle splits for two-phase pretimed control.
Source: Reference 2.

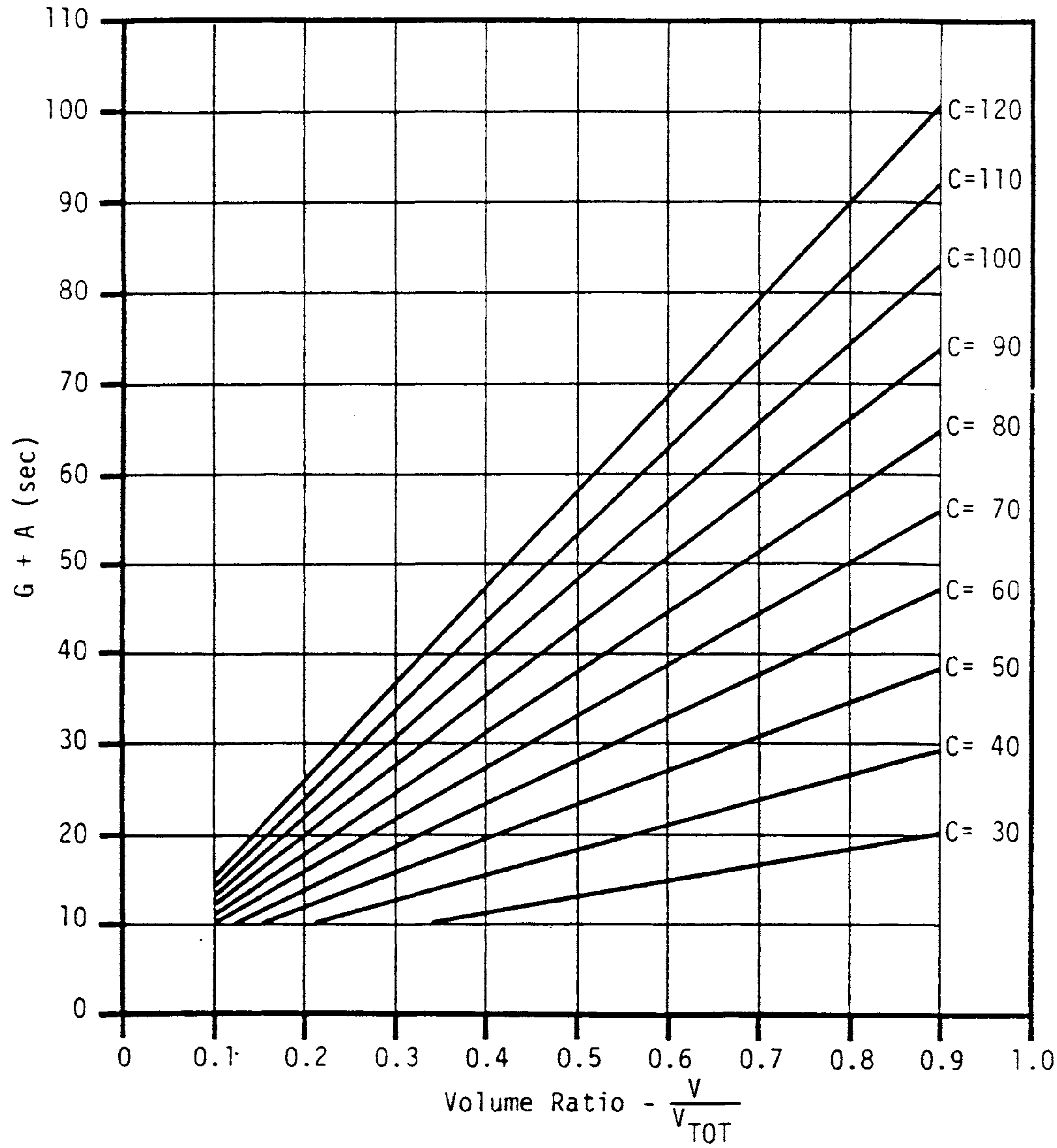


Figure 7. Cycle splits for three-phase pretimed control.
Source: Reference 2.

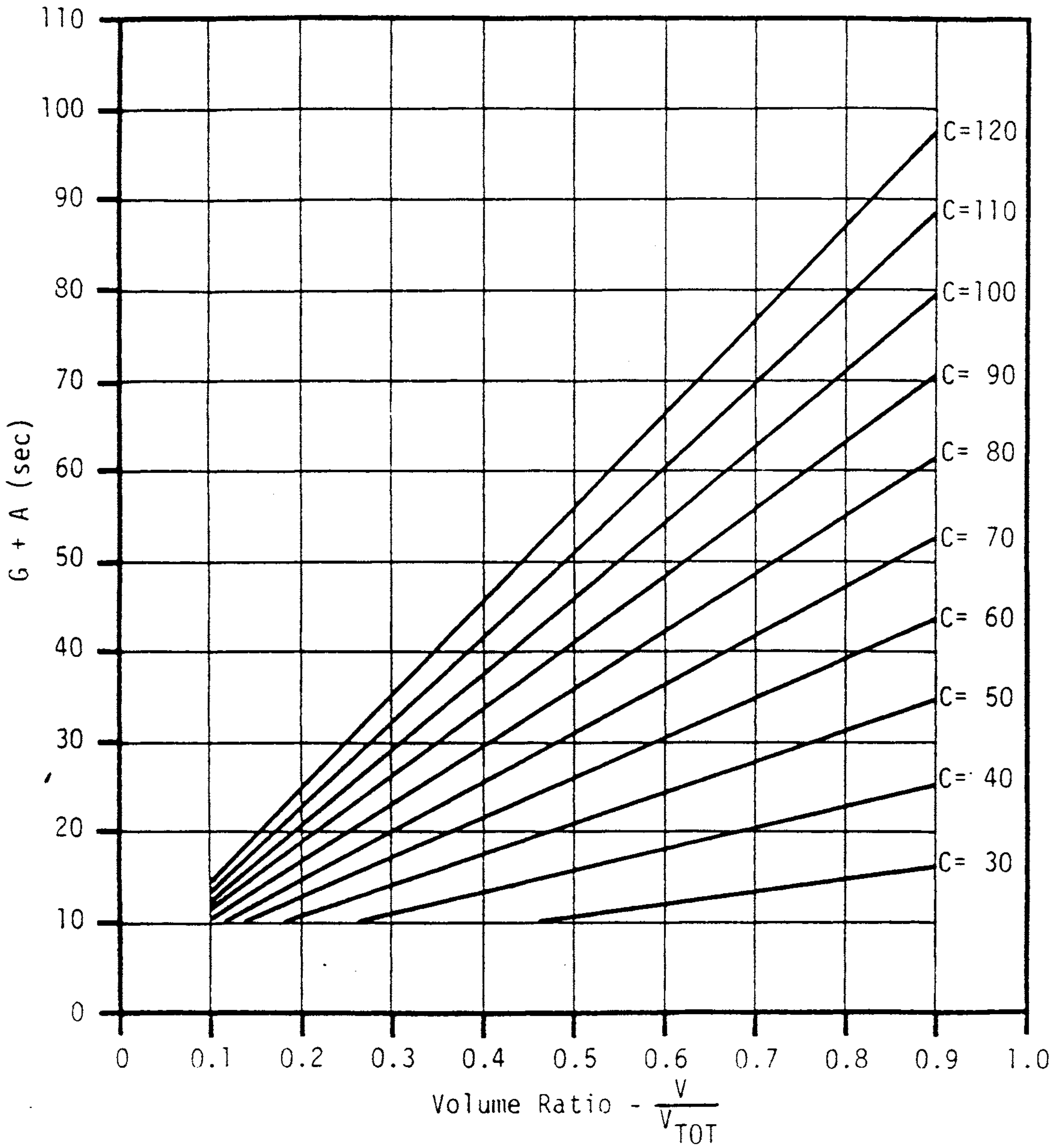


Figure 8. Cycle splits for four-phase pretimed control.
Source: Reference 2.

Table 2

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.

IX. 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. The previously determined splits should be checked to ensure they are not less than these minimums. If necessary, phase lengths should be increased to these minimums and the time added to the total cycle length.

X. Check for Minimum Pedestrian Requirements

Unless there is an exclusive pedestrian phase when all traffic is stopped, pedestrians must cross the street while traffic is stopped by a red indication for the street being crossed. This time occurs while the parallel vehicular traffic, or traffic on the street not being crossed, is receiving a green and clearance interval. Therefore, the sum of this green and clearance interval must be long enough to accommodate any pedestrian flow on the cross street.

Accordingly, if pedestrian movements must be accommodated, it is necessary to calculate the time required to cross the street and compare it with the sum of the green and clearance intervals on the street not being crossed. If this sum is less than the required crossing time, the green should be increased accordingly and the time added to the total cycle length.

Pedestrian crossing time consists of two components -- the walk interval, which is the time for the pedestrian to enter the intersection, and the clearance interval, which is the time needed to safely cross the street. The former interval is usually assumed to be between 4.0 and 7.0 seconds, while the latter interval is determined by dividing the width of the street in feet, W, by the walking speed, usually 3.5 to 4.0 feet/second. Although it is best to conduct a field survey of actual crossing times at the intersection, the following equation provides an estimate of the crossing time and, thus, minimum green plus clearance.

$$(G+A)min. = 5 + W/4, \tag{1}$$

where

(G+A)min. = minimum green plus clearance interval in seconds on approach not being crossed, and

W = width in feet of the street being crossed.

It is noted that the "5" and "4" should be replaced with a "7" and "3.5" where pedestrian volumes are high or in special cases such as in the vicinity of elderly housing.

XI. 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.

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.

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.

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

I. Collect Necessary Intersection Information

1. Control equipment already at the intersection -- timing functions, limitations of controller, timing values, and phases.

2. Physical data -- number of approaches; number of lanes and type of flow (through, right turn, left turn, or combination) per lane per approach; width of lanes and medians; percent grade on approaches, if severe; speed limits; location of parking, crosswalks, stop bars, bus stops, loading zones, etc.; and type, location, and, if applicable, size of detectors.

3. Traffic and pedestrian data -- hourly traffic volumes and pedestrian counts for each approach to the intersection; number of vehicles going straight and turning right or left on each approach; number of buses and large trucks on each approach; and average speed of traffic on each approach.

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 data over a period of time which will definitely include the peak-flow condition. Volume counts by 15-minute intervals during the peak-flow condition will enable the most accurate determination of peak-hour conditions.

Typical data collection and summary forms were provided previously in Figures 2, 3, and 4. These figures have been reproduced in this section for the convenience of the reader.

DIRECTIONAL TRAFFIC MOVEMENT – INTERSECTION OF ROUTES ______________________

COUNTY ______________________ LOCATION ______________________

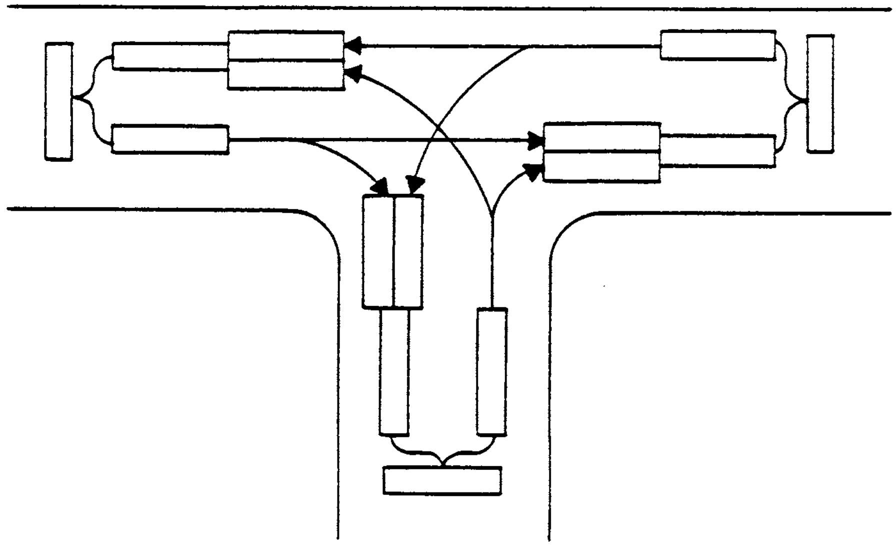
DATE _____ / _____ / _____ WEATHER ______________________

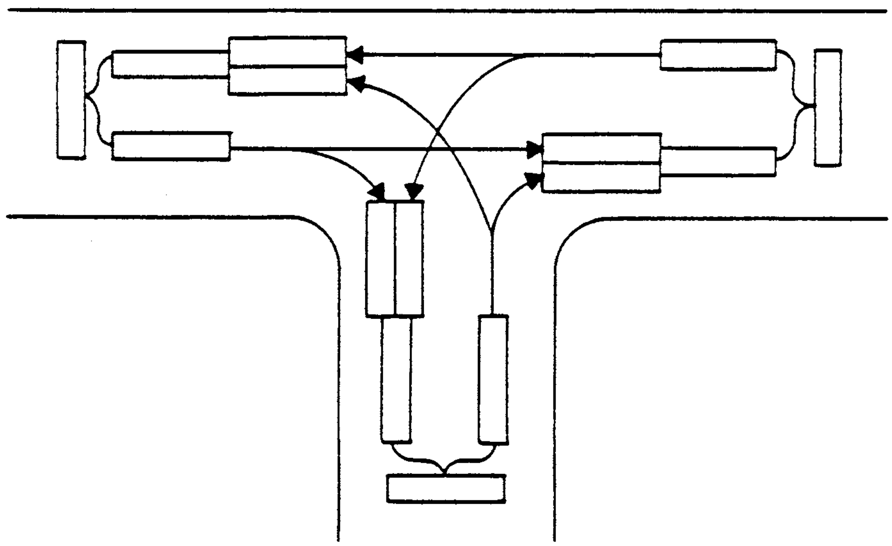
	APPROACHING INTERSECTION																
	ON ROUTE							FROM THE	ON ROUTE							FROM THE	
	EAST				WEST				NORTH				SOUTH				
HOURS	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	LT.	THRU	RT.	PED.	TOTAL
6:00 - 7:00																	
7:00 - 7:30																	
7:30 - 8:00																	
8:00 - 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 ______________________

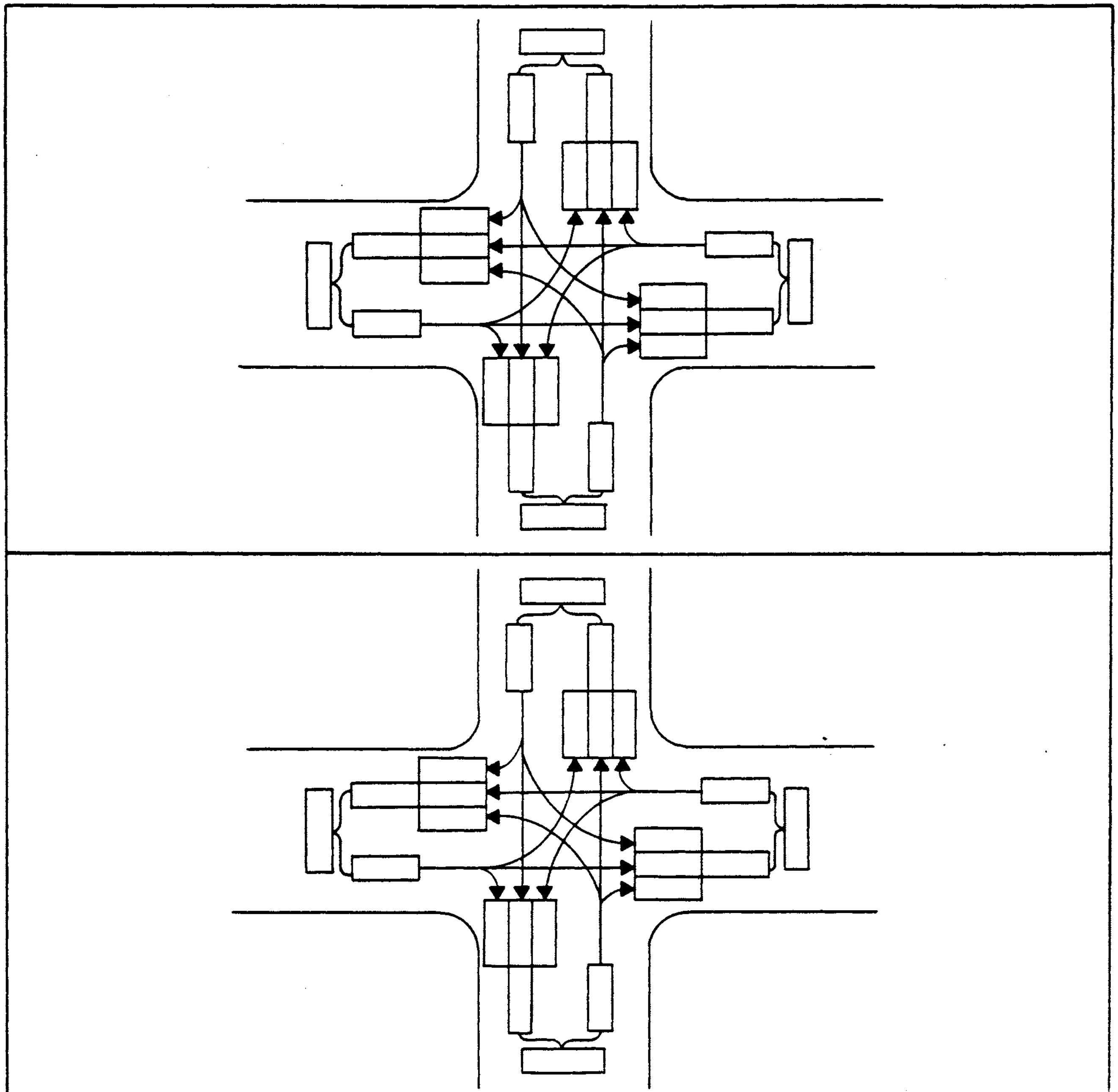
Figure 2. Typical data collection form.





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.

II. Determine Number of Phases

As a general rule, the number of phases should be kept to a minimum. The number of phases needed is most often a left-turn issue; that is, the volume of left-turning vehicles and the volume of opposing vehicles are such that a separate phase in the cycle is needed to accommodate vehicles turning left. A separate lane providing left-turn storage may alleviate the need for a separate phase. The following guidelines, coupled with engineering judgement, may be used to determine the need for left-turn phases at intersections already having separate left-turn lanes.(1)

1. Volumes -- consider left-turn phasing on an approach when, during the peak hour, the product of the left-turning volume and opposing volume divided by the number of lanes 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. Appendix B in the Technical Report provides 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 5 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.

III. 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 9, and a typical phase timing for older equipment is shown in Figure 10. 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.(4)

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

-- 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.

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

-- 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.

Passage Time (also, vehicle interval or unit extension interval)

Definition -- The passage time is the time needed for a vehicle moving at average speed to travel from the detector to and through the intersection. It also establishes the maximum gap in traffic, measured in seconds, at which the green interval is retained.

The approach speed that generally distinguishes high- and low-speed intersections is 35 mi/h; however, this breakpoint is subject to other considerations and should not be considered as absolute.

Timing -- Low-speed Intersections and Point Dectors -- Calculate the passage time interval by dividing the detector spacing, which is the distance between the detector and the stop bar, by the average approach speed; however, it should be no less than 3.0 seconds and no more than 5.0 seconds. Table 3 summarizes these rules.

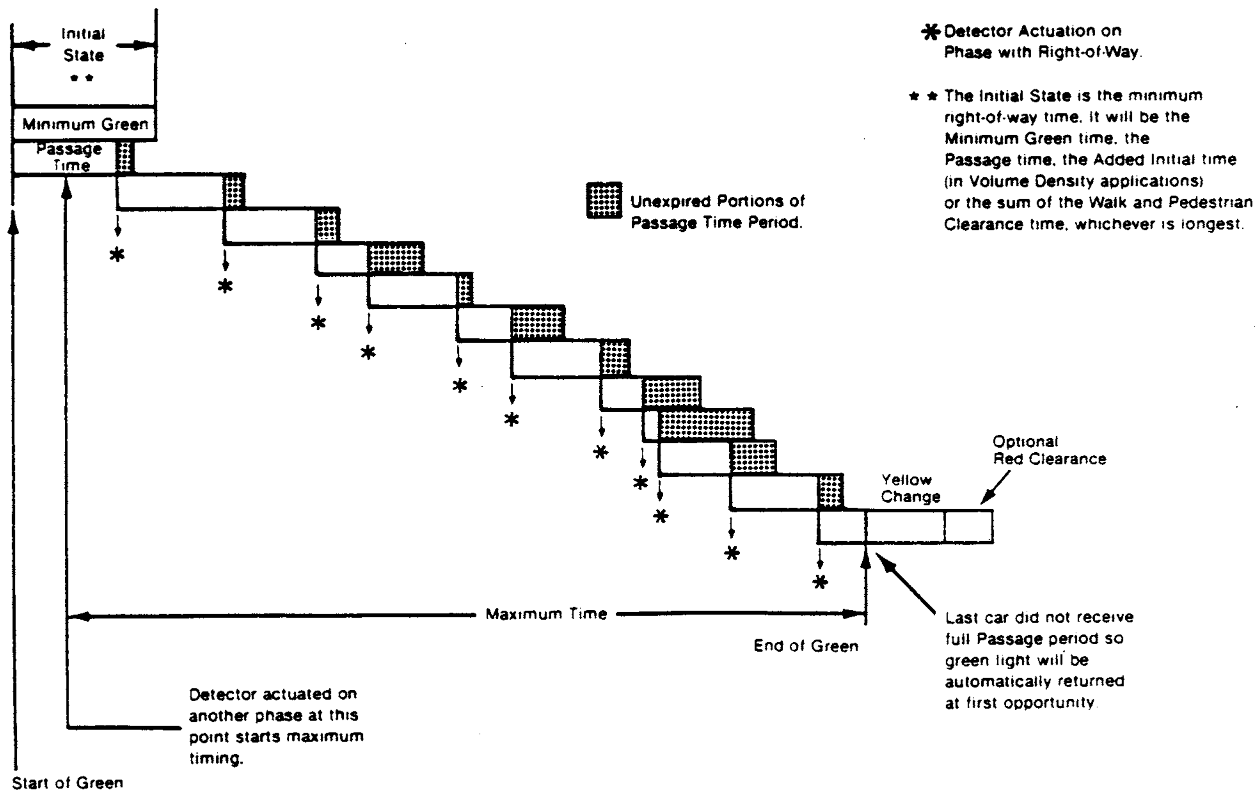


Figure 9. Typical timing diagram for a fully-actuated NEMA controller. Source: Reference 5.

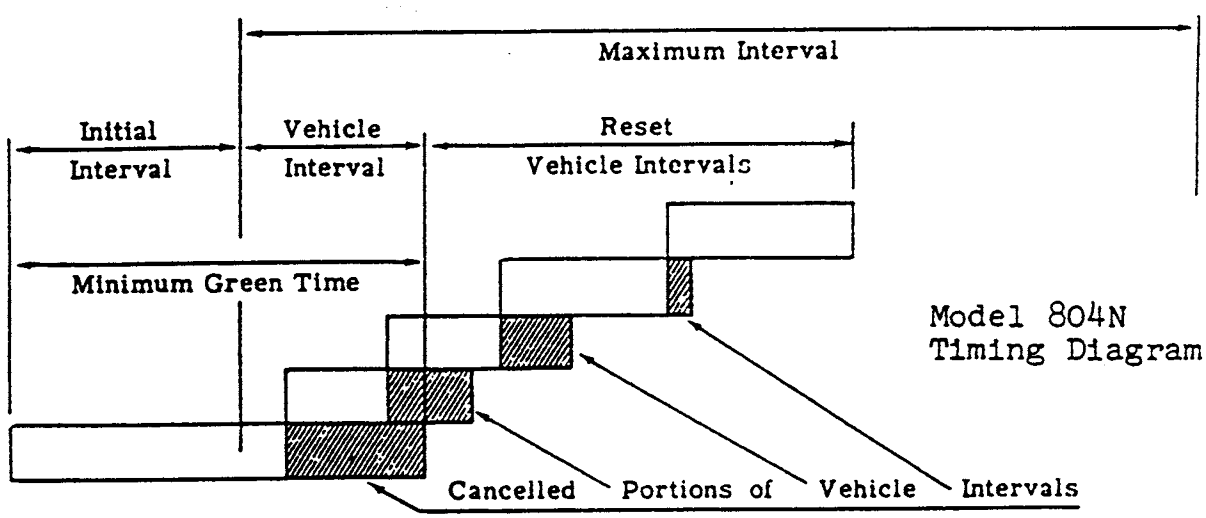


Figure 10. Typical timing diagram for a fully-actuated non-NEMA controller. Source: Reference 2.

Table 3

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 as calculated by dividing the detector spacing by the approach speed is greater than 5.0 seconds, it may be necessary to set the passage time interval above the recommended maximum for safety purposes.

Timing -- Low-speed Intersections and Presence Detectors at the Stop Bar -- Select the gap required to retain the green. As a general guideline, a gap of from 3.0 to 4.0 seconds is good for fast-paced, urban areas or where snappy operation is desired, and a gap of from 4.0 to 5.0 seconds is good for slow-paced, rural areas.

From Table 4 determine the built-in gap for the size of the detector used and the average approach speed.

Calculate the setting for the passage time interval by subtracting the built-in gap from the gap required to retain the green selected above. This setting is usually between 1.5 and 3.0 seconds. 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.

Table 4

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.

Timing -- High-speed Intersections with Two Point Detectors Per Lane -- Most high-speed intersections are controlled by the Virginia Department of Highways and Transportation; therefore, the designs described here and in the next section are typical of those found in Virginia.

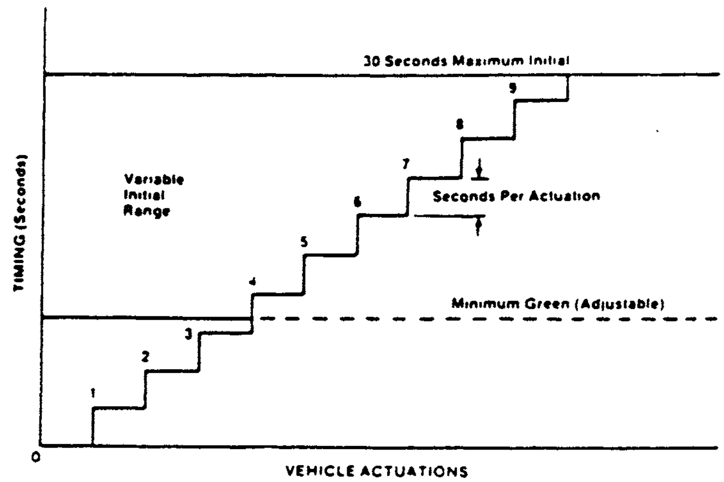
The use of two detectors per lane is intended to prevent, or at least reduce the chances of, a motorist being caught in the dilemma zone upon receipt of a yellow clearance interval. By design, the distance between the detectors is based on the passage time interval. Therefore, the passage time interval is calculated by dividing the distance in feet between the detectors by the average approach speed in feet/second. The setting is usually between 2.0 and 4.0 seconds. If there is a tendency for approaching motorists to slow down after crossing the second detector, the passage time may be set slightly higher than the design time to reduce the chances of a motorist being caught in the dilemma zone.

This type of design is generally used when the main line volumes and speeds are high as compared to those of the side street.

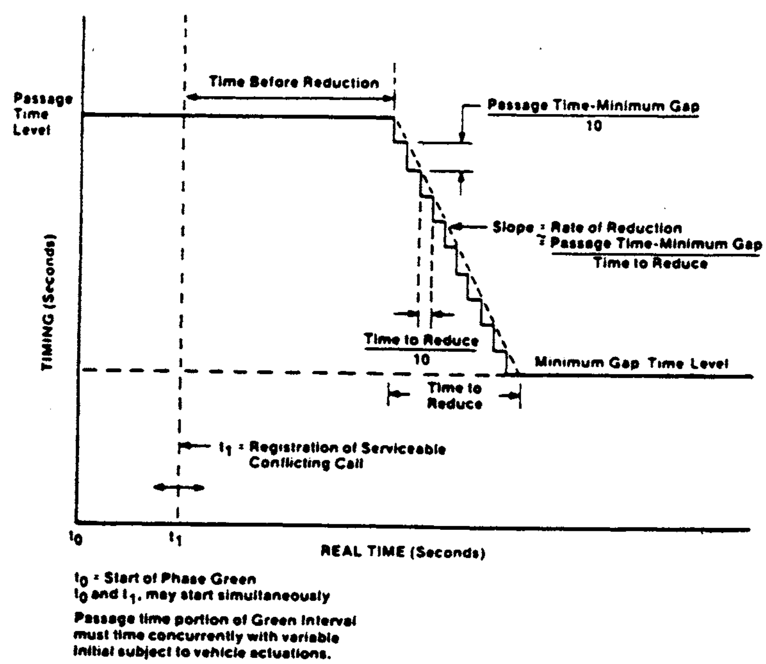
Timing -- High-speed Intersections with Volume-density Control -- If traffic volumes are approximately the same on all approaches, a volume-density operation is often used. Three of the four or five timing parameters associated with volume-density control pertain to the passage time. These are the time before reduction, the time to reduce, and the minimum gap. In operation, the passage time of from 3.0 to 5.0 seconds applicable to low-speed intersections with point detectors controls the gap needed to retain the green during the "time before reduction" period. Once the 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's operation. The lower portion of Figure 11 shows gap reduction in a schematic form. Note that gap reduction is initiated when a call is received on a conflicting phase.

Intersections with volume-density controllers normally use point detectors; therefore, the procedures described earlier for determining the passage time for low-speed intersections with point detectors are also applicable in this case.

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 11. 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

Definition -- 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. The minimum green timing parameter is shown schematically in Figures 9 and 10. It is important to note from Figure 10 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 below, and the initial interval is set by subtracting the vehicle (passage time) interval.

Timing -- Low-speed Intersections and Point Detectors -- 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.

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 5.

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

Table 5

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.

Timing -- Low-speed Intersections and Presence Detectors at the Stop Bar -- 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.

Timing -- High-speed Intersections with Volume-density Control -- Volume-density operation is applicable in this case because the detectors are normally located at a considerable distance from the intersection. A minimum green based on the assumed storage of vehicles in that distance creates an inefficient operation if traffic is light. One or two additional parameters are included--seconds per actuation and maximum added initial. In operation, a minimum green based on one vehicle is set. 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 11.

A value of 5.8 seconds is set as the minimum green interval for one vehicle. This is calculated by applying the formula $2.1\,n + 3.7$.

Based on this formula for a single lane approach, a value of 2.1 should be set for the seconds per actuation. Volume-density control is most often used at multilane intersections, and a value of 1.0 second per actuation should be used in this case.

If maximum added initial is available on the controller, it should be set to accommodate the maximum storage of vehicles in a single lane between the stop bar and detector. This setting can be determined from Table 5 or application of the formula 2.1 n + 3.7, with n being the detector spacing in feet divided by 20 feet.

Minimum Green -- Non-actuated Phase

Definition -- 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 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.

Timing -- Generally, if the side street demand is occasional and occurs randomly throughout the day, 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.

Specifically, minimum green can be calculated based on 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 previously 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. Other settings are determined based on the actuated signal procedures.

Maximum Green

Definition -- 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. 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.

Timing -- 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. These procedures were presented previously.

Figures 9 and 10 depict schematically how the maximum green is timed. As shown in Figure 9, the maximum green for NEMA controllers begins timing upon receipt of a call on a conflicting phase. As shown in Figure 10, 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

Definition -- 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.

Timing -- The phase change interval should be long enough to allow a vehicle to decelerate to a safe stop or to proceed safely through the intersection prior to the red indication. The yellow interval is set to allow a safe stop, whereas the red interval is set to allow a safe passage through the intersection. It is important that motorists have a reasonable expectation of the length of the yellow interval; therefore, it should always be between 3.0 and 5.0 seconds. The phase change interval is determined by entering Table 2 with the average approach speed and reading the time for the yellow change interval and for the total clearance needed. This table has been reproduced in this section of the report for the convenience of the reader. The all-red interval is the difference between the needed clearance and the yellow interval. If there is a significant grade on an approach, Equation 5 in the Technical Report should be used to calculate the phase change interval. An all-red interval is normally not used on an exclusive left-turn phase, except in the case of wide medians or high-speed approaches.

Table 2

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

Definition -- The walk and pedestrian clearance intervals allow time for the pedestrian to enter the intersection and cross the street, respectively. Unless there is an exclusive pedestrian phase when all traffic is stopped, this must be accomplished while the parallel vehicular traffic, or traffic on the street not being crossed, is receiving a green and clearance interval. Therefore, the sum of the green and clearance interval on an approach should be long enough to accommodate any pedestrians on the cross street. In operation, these intervals begin timing if actuated by a call from a pedestrian, and thus reestablish a minimum green for that phase.

Timing -- Although it is best to base these settings on field observations of pedestrian crossings at the intersection, approximate settings for these two intervals can be established. The walk interval is generally assumed to be between 4.0 and 7.0 seconds, with high values used when pedestrian volumes are high or the pedestrians are elderly.

The pedestrian clearance interval is calculated by first dividing the width in feet of the street being crossed by an assumed walking speed of 3.5 to 4.0 feet/second, with slow speeds used when pedestrian volumes are high or the pedestrians are elderly. This is the required crossing time. Next, the setting is determined by subtracting the yellow and all-red clearance intervals for the street not being crossed from the required crossing time.

It is important to remember that these intervals are set for the phase controlling traffic on the street not being crossed.

IV. 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.

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.

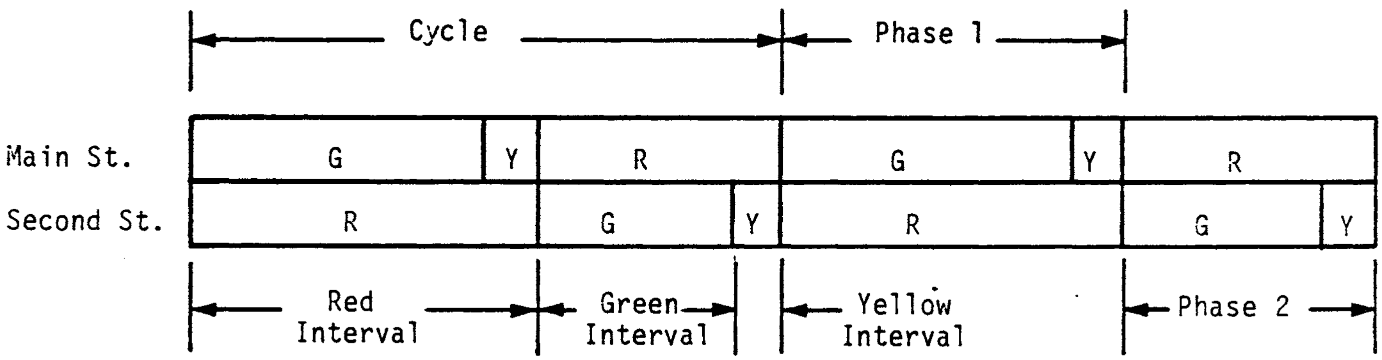


Figure 1. Timing sequence for simple two-phase controller.

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. 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.

   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 12.

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 12.

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 12.

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 12.

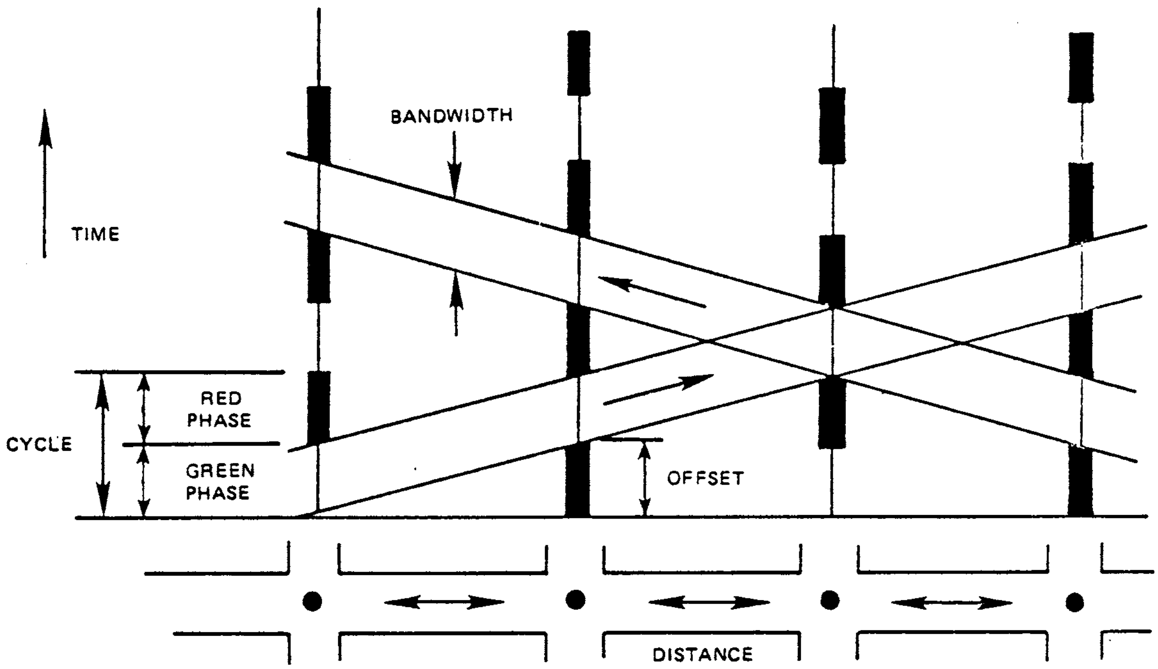


Figure 12. Time-space diagram.
Source: Reference 8.

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.

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.

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.

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.

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 section of the Technical 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. These procedures are for the most part duplicated from the Technical Report.

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

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 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 methodology 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

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

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

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

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 13. 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 13.

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 13.

4. Carefully fold the cycle split aid, Figure 14, 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 <u>red</u> 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 15 and 16.

7. Repeat the procedure described in 6 above for each signalized intersection in the system. Either artery red or artery green must be <u>centered</u> 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 17.

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.

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 18. 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.

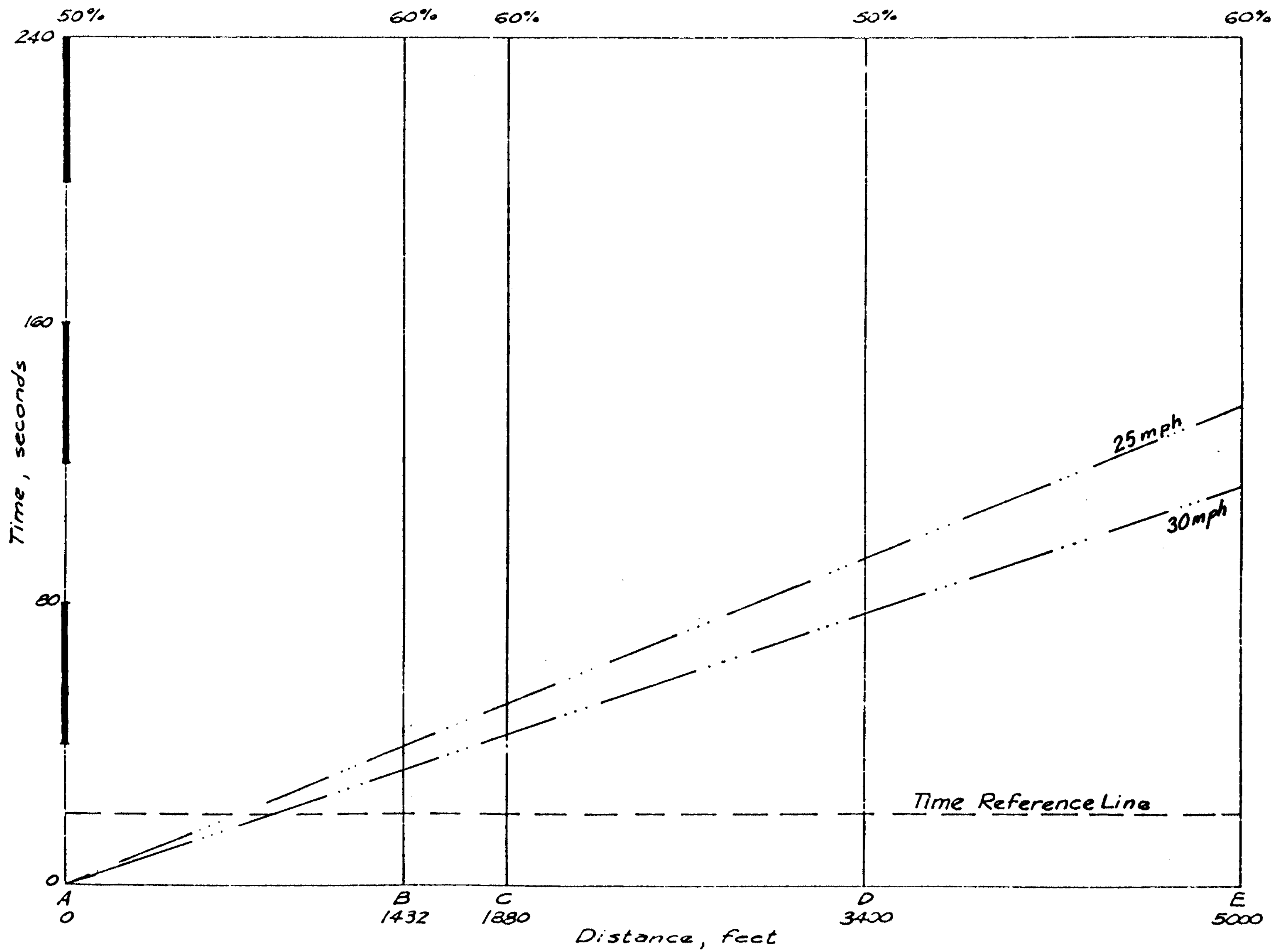


Figure 13. Time-space diagram construction.
Source: Reference 11.

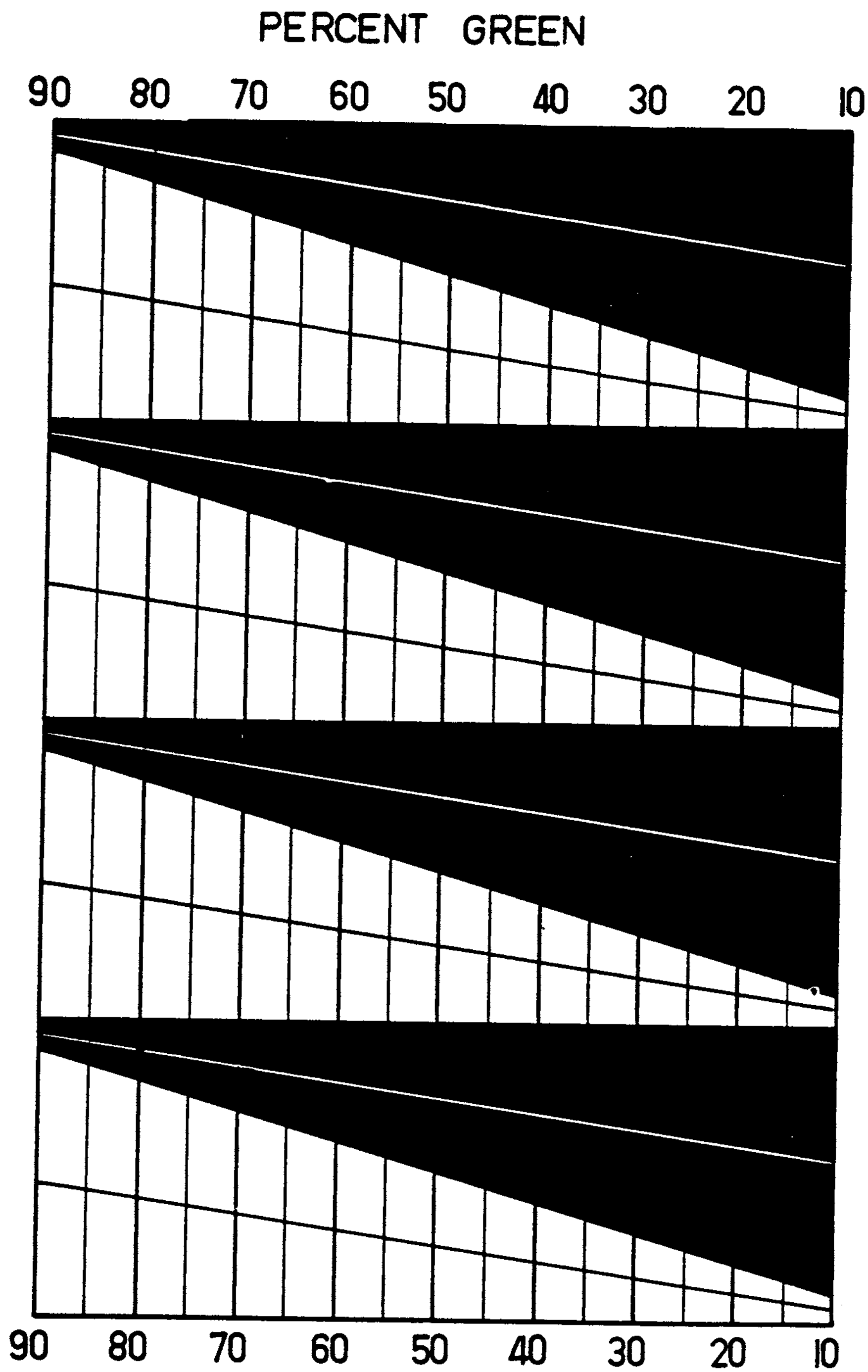


Figure 14. Cycle split aid.
Source: Reference 11.

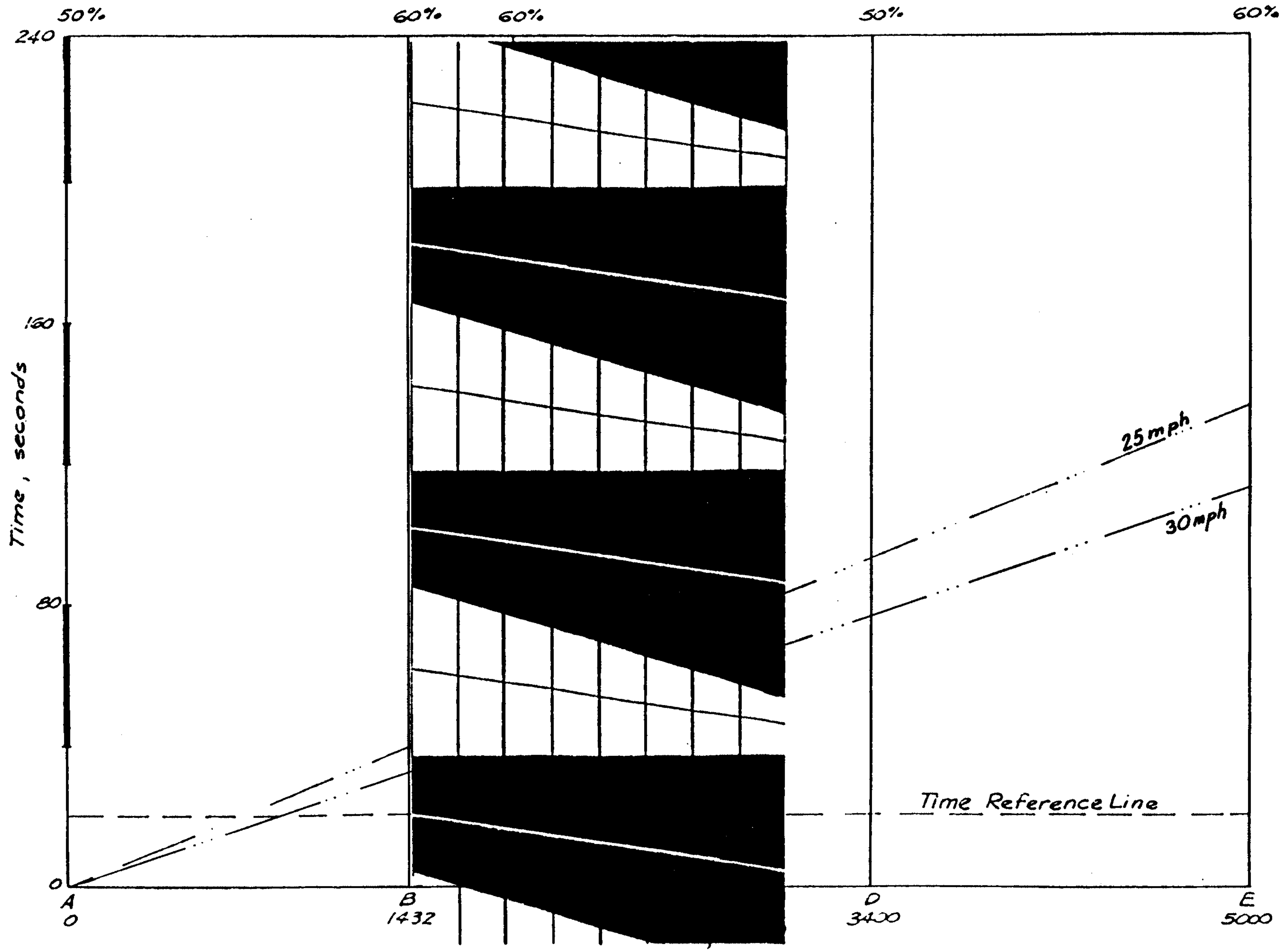


Figure 15. Time-space diagram construction.
Source: Reference 11.

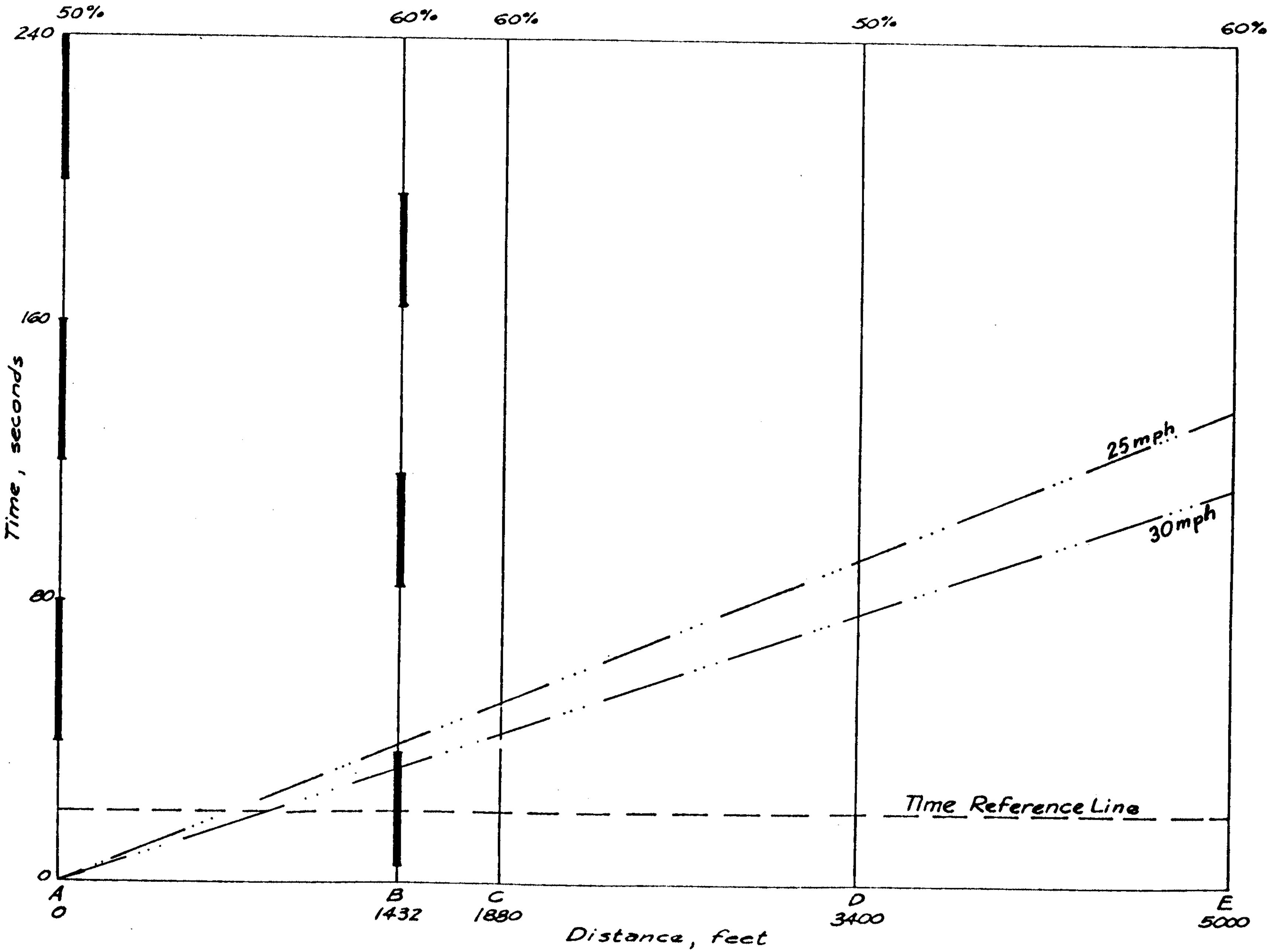


Figure 16. Time-space diagram construction.
Source: Reference 11.

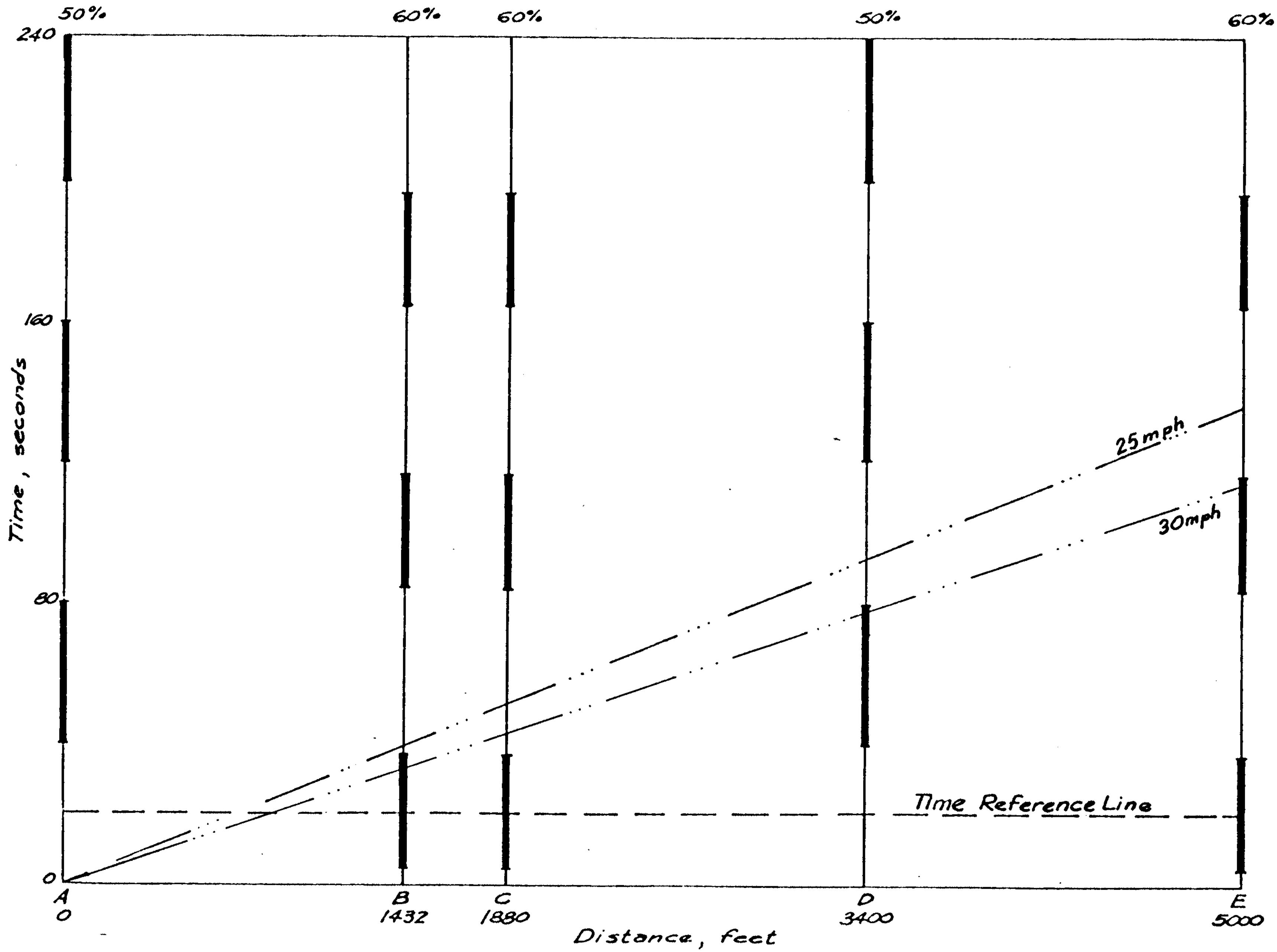


Figure 17. Time-space diagram construction.
Source: Reference 11.

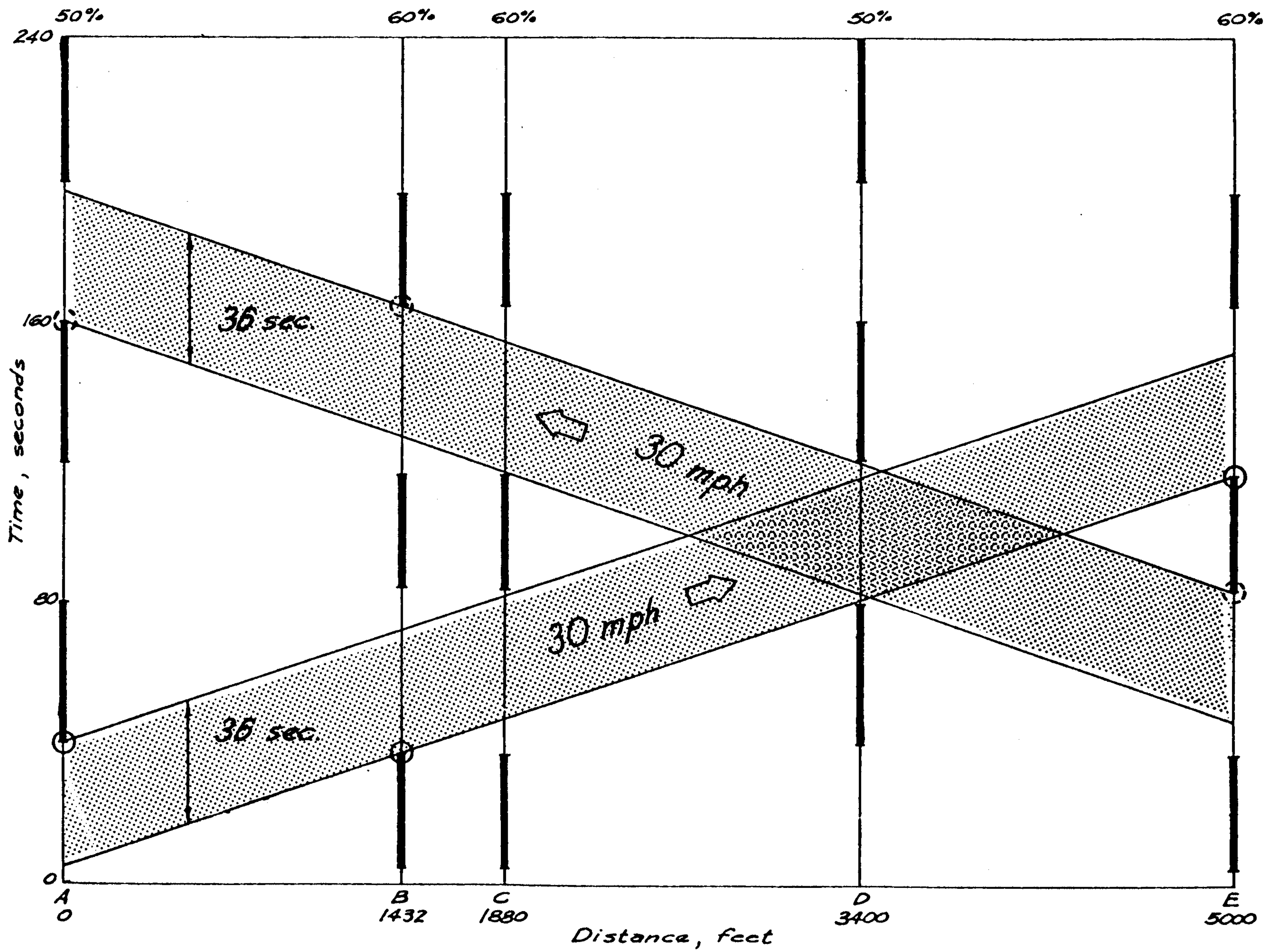


Figure 18. Completed time-space diagram.
Source: Reference 11.

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.

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 19.

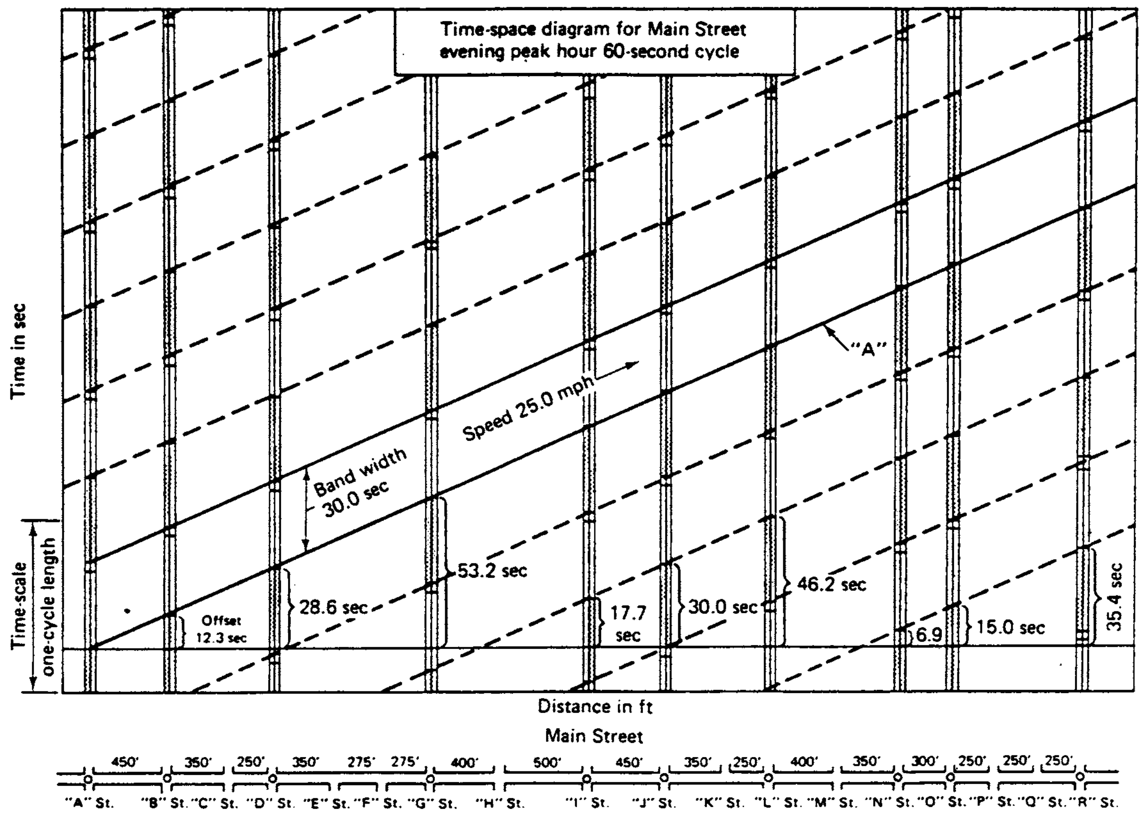


Figure 19. Completed time-space diagram favoring one direction of flow. Source: Reference 10.

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 19.

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.

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 of the Technical Report.

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REFERENCES

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.