METHODS OF CYCLE LENGTH SELECTION
AND
SIGNAL TIMING

by

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B. S., Kansas State University, 1960

A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1964

Approved by:

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INTRODUCTION

The use of traffic signals as a method of assigning right-of-way at busy street intersections has been in existence for a number of years. However, traffic signals have been used wrongfully in some cases as a means of traffic control when the traffic condition could have been remedied by the use of some other type control device.

The primary objective in the signalized operation of an intersection is to achieve the required capacity for each direction of flow with minimum delay to the user. The full value of a signal operation is realized only when it is operated in a manner consistent with the traffic requirements. A signalized operation may be at an isolated intersection or incorporated into a series of two or more intersections to form a progressive system. The use of unduly long signal cycles or improper division of cycle times should be avoided when selecting cycle times as they may cause undue delay and eventually disobedience of the signal installation.

In order to select the proper cycle for a given intersection, it is necessary to first observe the intersection to obtain the physical and operational characteristics. These characteristics will include such factors as speed and volume studies, turning movements, width of crossing, time-spacing of vehicles, and delays of vehicles. From these data the minimum pedestrian crossing time and the required yellow light time can be obtained. Then some method of cycle length selection is applied to the data and the optimum cycle length obtained.

After a cycle length has been obtained, it is necessary to determine how much of the total time available at the intersection should be apportioned to each flow of traffic movement. The selection and arrangement of simultaneous flows of movement is known as "phasing" (8). The primary objective of phasing is to accommodate all traffic movement with increased safety and minimum delay. In order to properly phase an intersection, green time or "go" intervals must be assigned to each movement of traffic flow in the proportion which will optimize the movement in both directions.

This report will primarily be concerned with methods of cycle length selection and signal timing. Two methods of cycle length selection and one method of progressive timing are presented in this report to illustrate methods of approach to signal timing. It must be remembered however, that after a signal cycle length has been selected and the proper cycle division assigned to each approach, the installation must be observed and studied to insure that the performance of the system is adequate for the traffic demand.

REQUIREMENTS OF TRAFFIC SIGNALS

At intersections where the demand for movement with safety heavily taxes the supply of road space - especially during certain periods of the day - the right of way is frequently assigned alternately to the diverse traffic flows. This assignment of traffic movement is often accomplished by the use of automatic traffic control devices. Any control device used for this purpose must fulfill five basic requirements: attention, meaning, time for response, respect, and fill an important need. In addition, the control device must also apportion available time among road users. Traffic signals are widely used in the assignment of right of way at intersections.

A highway traffic signal is defined as any power operated traffic control device, except a sign or a flasher, by which traffic is warned or is directed to take some specific action (1). A traffic signal is a valuable device for control and safe facilitation of vehicle and pedestrian traffic. After a signal has been timed, it exerts a direct influence on traffic flow and for this reason it is very essential that the timing program be checked to see that it meets the requirements of the traffic.

In order to insure that the five basic requirements of any control device are met, the following should be emphasized (1):

1. Design - The size, shape, color, and simplicity of message should be combined to produce a clear meaning. These factors combined with placement of the control device should provide adequate time for reaction.
2. Placement - The placement of the control device shall be such that it will be in

the cone of vision of the user. It must also be located next to the object or condition to be controlled.

3. Maintenance - The control devices must be maintained to high standards if they are to fulfill the need for which they are intended. Also, if a control device is no longer needed at a location, it should be removed. The control devices must be kept clean and legible at all times in order to convey the proper message.
4. Uniformity - The uniformity of control devices aids in instant recognition for road users. Similar situations should be treated in the same manner. Imaginative creation and engineering judgment should be used to obtain uniformity.

Highway traffic signals, properly located and operated, usually have one or more of the following advantages (1):

1. They provide for orderly movement of traffic. Where proper physical layouts and control measures are used, they can increase the traffic handling capacity of the intersection.
2. They reduce the frequency of certain types of accidents.

3. Under conditions of favorable spacing, they can be coordinated to provide for continuous, or nearly continuous movement of traffic at a definite speed along a given route.
4. They can be used to interrupt heavy traffic at intervals to permit other traffic, pedestrian or vehicular, to cross.
5. They represent a considerable economy, as compared with manual control, at intersections where the need for some definite means of assigning right of way first to one movement and then to another is indicated by the volumes of vehicular and pedestrian traffic, or by the occurrence of accidents.

Highway traffic signals may also have one or more of the following disadvantages:

1. Average traffic delay may be increased, if traffic volume is light.
2. Rear end and turning movement collisions will usually increase.
3. Improper installation of a signal may cause disobedience.

Traffic signals should conform in all respects to the standards set forth in the "Manual on Uniform Traffic Control

Devices," (1) or the state equivalent. These standards specify the size of lenses, number of lenses per signal face, color and positioning of lenses, arrow shape, meaning of color and arrow indications, illumination of lenses, location of signal faces, height of lenses and minimum warrants justifying the installation of the signal.

Traffic control signals should be installed and operated on public highways only by legally delegated or constituted public authority. Suitable legislative models which govern these operations are presented in the "Uniform Vehicle Code" (2) and in the "Model Traffic Ordinance" (3). The Code and the Ordinance direct the State Highway Commission and the city traffic engineer, or similar public officials or bodies, to place and maintain a uniform system of traffic control devices correlated with and, so far as possible, conforming to the system currently approved by the American Association of State Highway Officials.

FACTORS AFFECTING SIGNALIZED OPERATION OF INTERSECTIONS

In order to understand the mechanics of signal timing, it is necessary to first acquire a general understanding of the factors that influence the capacity of an intersection. The maximum capacities of the approaching roadways are controlled by the green phases of the traffic signals. In signal timing, the objective is to provide maximum capacity with minimum delay and inconvenience to the users. Pedestrian safety must also be considered in signal timing.

The following is a list of various factors which influence the capacity of a signalized intersection:

Physical (4):

1. Number of lanes in each approach to the intersection.
2. Width of approach lanes.
3. Lateral clearance and shoulder width.
4. Grade and alignment of each approach.
5. Sight-distance at intersection.
6. Divided or undivided roadways and width of median, if any.
7. Channelization lanes.
8. Type of signal device: fixed-time, semi-actuated, fully-actuated, etc.
9. Access control, i.e., presence of driveways and other roadside interference.

Operational:

1. One-way or two-way traffic on approaches.
2. Location of bus stops.
3. On street parking.
4. Speeds on intersection approaches.
5. Volume of turning movements.
6. Number and size of commercial vehicles in traffic stream.
7. Pedestrian traffic.

8. Signal timing.
9. Driver behavior.

INTERSECTION CAPACITIES

The capacity of an intersection approach, as described in the "Highway Capacity Manual" of 1950 (5), is broken down into two commonly used categories and is defined as follows:

Possible capacity - the maximum number of vehicles that actually can be accommodated under the prevailing conditions with a continual backlog of waiting vehicles.

Practical capacity - the maximum volume that can enter the intersection during one hour with most of the drivers being able to clear the intersection without waiting for more than one complete cycle.

The Highway Capacity Manual of 1950 is under review and revision and it is planned that the new edition will state the capacities as a function of "level of service" (4). The level of service could be measured in terms of travel time, speed-change frequency, accident hazards, delay or other quality measures acceptable to the designer. It is felt that capacities based on these levels of service will have more meaning to the engineer than terms such as "most of the drivers" and "prevailing conditions".

It has been found that capacity of intersections-at-grade varies in almost direct ratio with the width of approach, measured from the curb line. However, a number of adjustments are necessary when applying the information for average intersection conditions to an intersection where conditions are not average. The most important adjustments for capacity and the adjusting factors are:

For Urban Signalized Intersection Capacity (6).

1. Possible and practical capacities.
    a. Possible capacity: On the average, possible capacities are about ten per cent higher than the average rates represented in Fig. 1 (two-way chart).
    b. Practical capacity: On the average, practical capacities are about ten per cent lower than the average rates represented by Fig. 1.
2. Commercial vehicles: Subtract or add one per cent for each per cent by which per cent commercial is above or below ten per cent of the total traffic.
3. Turning movements.
    a. Right turns: Subtract or add one-half per cent by which right turns are below or above ten per cent of the total traffic. (Maximum reduction ten per cent).

b. Left turns: Subtract or add one per cent for each per cent by which left turns are below or above ten per cent. (Maximum reduction 20 per cent.)

Note: Maximum reduction for right and left turns combined is 20 per cent.

4. Bus stops and parking near intersection.

a. On streets where parking is prohibited

(1) No bus stop - add five per cent

(2) Bus stop on near side - subtract three per cent in downtown areas and 15 per cent in intermediate areas

(3) Bus stop on far side - subtract three per cent in downtown areas and 15 per cent in intermediate areas.

b. On streets where parking is permitted except at bus stops

(1) Bus stop on near side - add one-quarter per cent for each one per cent turns, maximum increase not to exceed six per cent

(2) Bus stop on far side - make no correction

c. On streets where parking is permitted and there are no bus stops

(1) Subtract one-quarter per cent for each one per cent turns, but maximum reduction not to exceed six per cent

Note: For one-way streets - same as above, except left turns have same effect as right turns and use Fig. 2 for intersection capacity.

When adjusting the volumes shown in Fig. 1 for conditions that are not average, each adjustment must be made as a separate step. To accomplish this when a number of adjustments are necessary, each adjustment can be calculated and added to or subtracted from 1.00, and a total factor then obtained from these individual factors by multiplying them together. The example that follows illustrates the application of the method.

<u>Example No. 1.</u>

<u>Problem</u>

What are the possible and practical capacities of one approach to an intersection on a two-way street that has the following characteristics:

Street width - 40 feet curb to curb

Area - downtown

Parking - permitted

Turns - 5 per cent right turns

12 per cent left turns

Commercial traffic - 2 per cent at peak hour

Bus stops - far side

Green time - 27 seconds out of 50-second cycle

Directional movement - 60 per cent one direction during peak hour

<u>Solution</u>

From Fig. 1 the capacity on one approach on the average street 40 feet wide from curb to curb in a downtown area with parking permitted is 950 vehicles per hour of green.

The following adjustments are required because conditions are not average:

Cause	Effect	Factor
Right turns	(10-5) x ½% = +0.025	1.025
Left turns	(10-12)x 1% = -0.02	0.98
Commercial vehicles	(10-2) x 1% = +0.08	1.08
Bus stop - far side	No correction	1.00

Total factor = 1.025 x 0.98 x 1.08 x 1.00 = 1.085

Green time - 27 seconds out of 50-second cycle

$= 27/50 = 0.54$

Directional movement - 60 per cent one direction during peak hour $= \frac{1}{0.60}$

Therefore,

Possible capacity $= \frac{1.10 \times 1.085 \times 0.54 \times 950}{0.60} = 1020$ vehicles per hour in the direction of heavier flow.

Practical capacity $= \frac{0.90 \times 1.085 \times 0.54 \times 950}{0.60} = 835$ vehicles per hour in the direction of heavier flow.

Other examples of intersection capacity problems are

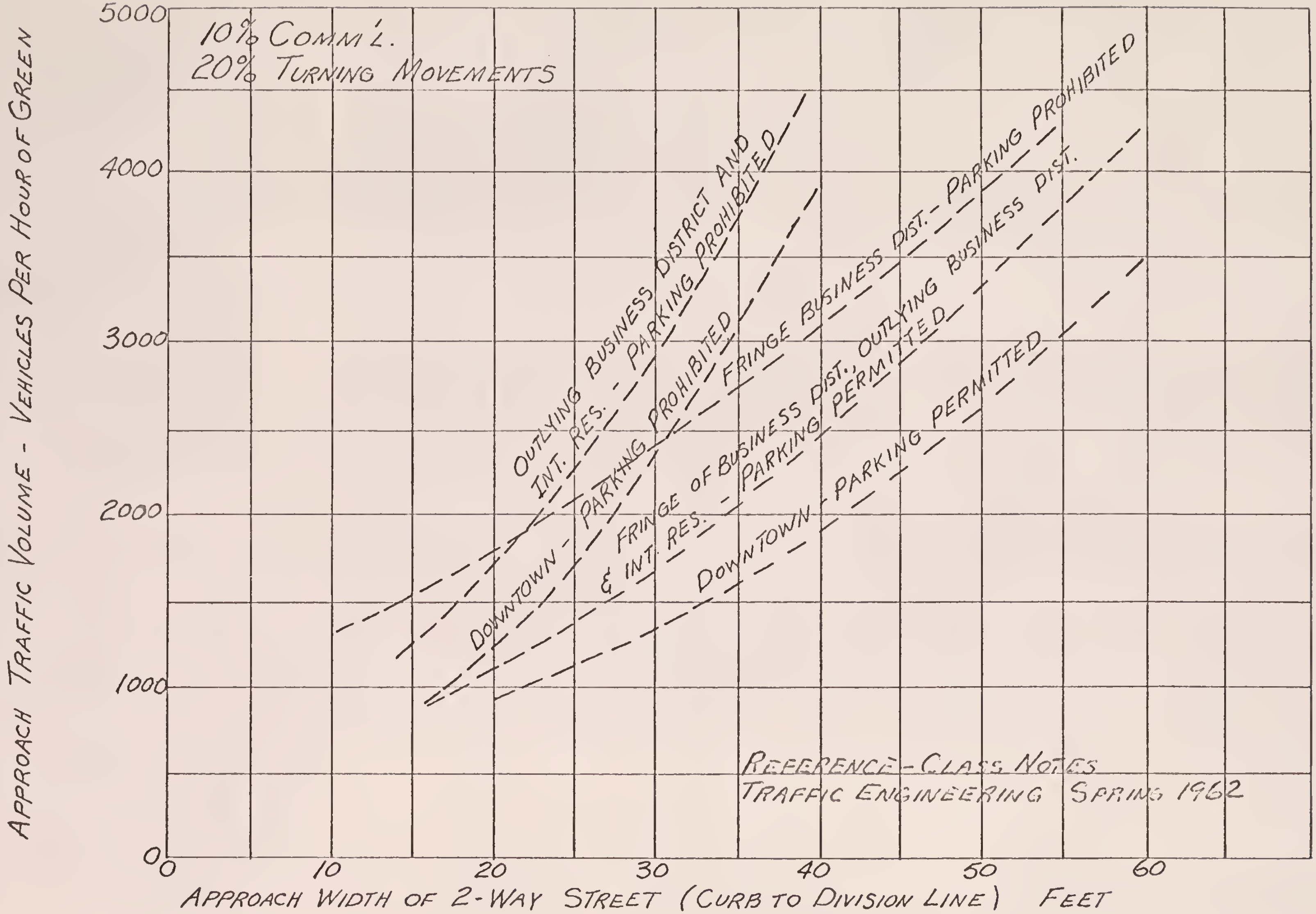


FIGURE 1 - INTERSECTION CAPACITIES OF 2-WAY STREETS, FIXED-TIME SIGNALS

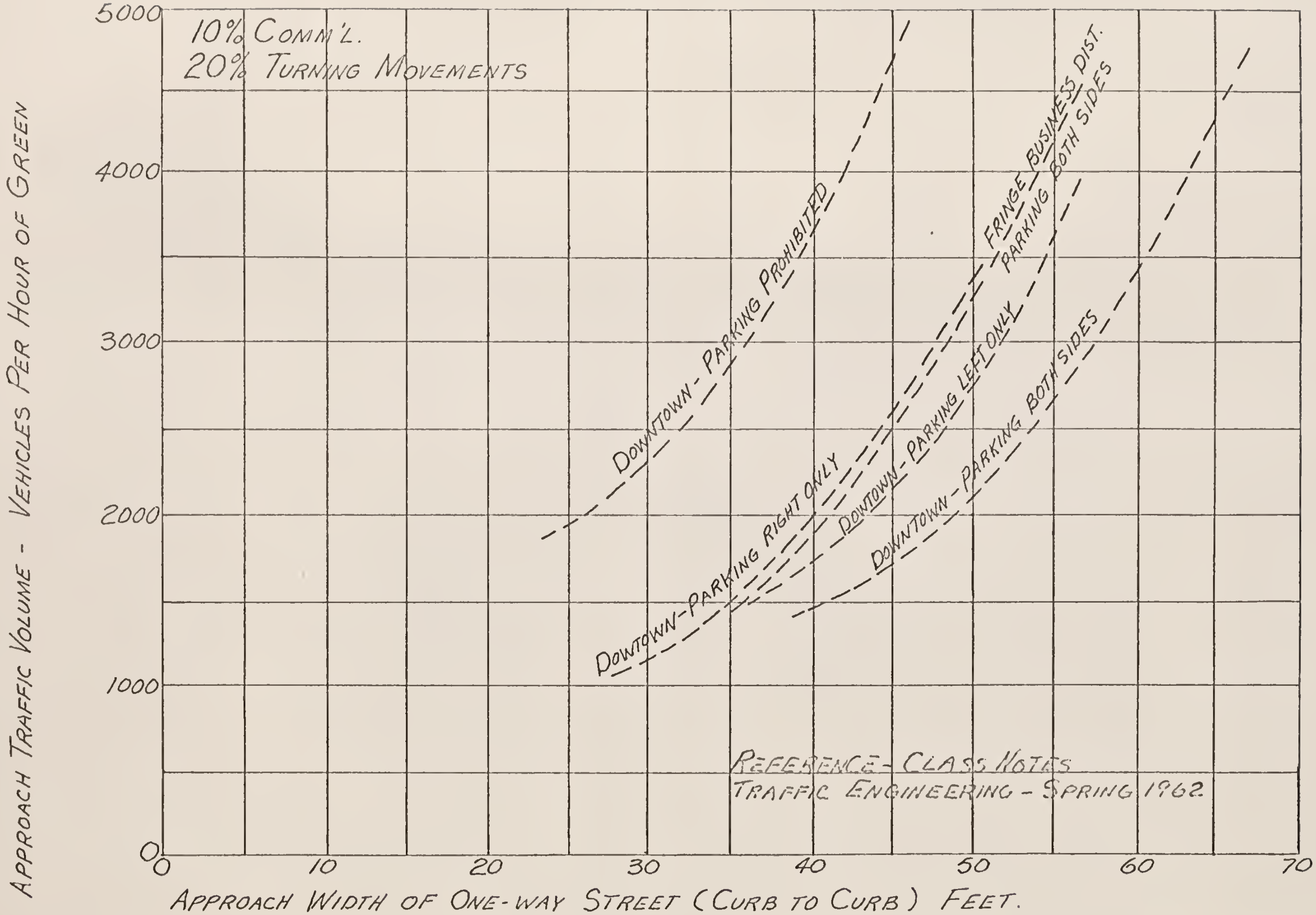


FIGURE 2 - INTERSECTION CAPACITIES OF ONE-WAY STREETS, FIXED-TIME SIGNALS

given in the Highway Capacity Manual of 1950 (5).

The capacity of the intersection can be altered by eliminating certain movements, changing cycle times, rerouting commercial vehicles, or changing the physical aspects of the intersection. It is up to the engineer to study the intersection and make the necessary changes in order to meet the demand of the traffic. In some instances one change (such as eliminating left turns) may be the only change necessary to increase the capacity of the intersection to that desired.

EQUIVALENT VEHICLE VOLUMES

In order to use the formulas based on passenger car units, the effect of turning movements and commercial vehicles will be accounted for by converting these effects to equivalent passenger car units. This method is approximate but it is widely used because it is easy to apply and it produces satisfactory results for signal timing and performance evaluation. However, if the turning movement or commercial vehicle traffic has unusual characteristics, they should be examined and considered separately.

Effect of Turning Movements

The fact is well known that left-turning vehicles take more time to clear an intersection than straight-through vehicles. Vehicles making left turns also block vehicles in the lane unless a turning lane is provided or there is sufficient space to pass. The amount of congestion depends upon the amount of volume in the opposing lane. If the opposing traffic is light, the blocking may be negligible. When it is heavy, the blocking may be such that it causes the left-turning vehicles to wait until the end of the green signal and make the turn on the amber signal. When this condition exists, a separate left turn signal may be warranted.

In a study reported by Greenshields (4), an average value of 1.3 seconds was added to the green time for each left turn. This time increment was added to the sum of the left turns including those in the opposing direction that entered the intersection on the same green phase. Therefore, if it is assumed that the time spacing of the straight-through vehicles, exclusive of starting delay, is 2.1 seconds per vehicle, then if a left-turning vehicle requires 1.3 seconds more, its headway becomes (2.1 + 1.3) or 3.4 seconds or 3.4/2.1 = 1.6 times that of a straight-through vehicle. Thus, each left turn will be equivalent to 1.6 straight-through vehicles (4). The effect of right-turning vehicles is commonly neglected in signal timing. However, extra green time is required if the turning radius for a right turn is less than 50 feet, then each right-turn vehicle is equivalent to 1.4 straight-through vehicles (4). Where there is an appreciable number of pedestrians crossing with the green light, there should be allowance for time lost by right-turning vehicles in yielding right of way to the pedestrians.

Effect of Commercial Vehicles

The effect of commercial vehicles varies widely among different sizes and weights. Trucks require more time to clear an intersection because their length is greater and generally their rate of acceleration is less. Greenshields suggests that an average commercial vehicle is equivalent to 1.5 passenger cars approaching a signalized intersection. The conditions assumed for this factor are level roadways and a predominance of medium or light trucks (4).

METHOD I - FIXED-TIME SIGNAL CYCLE LENGTH SELECTION

This method of cycle length selection utilizes techniques which can be applied to specific cases to optimize signal timing in terms of level of service measured by (a) vehicle delay, (b) the number of vehicles backed up, and (c) the probability of entering the intersection during the first green phase (4).

The application of the method begins by making a first approximation of the cycle length producing minimum delay. Green time is then apportioned according to the maximum vehicle flow in each phase. The delay per vehicle for the critical approach lane and the total delay for the entire intersection are then determined for the approximated cycle length. The expected length of queue and the probability of entering the intersection during the first green phase is next computed for the critical approach for each green phase.

These calculations are made for the desired range of possible cycle lengths. Finally the cycle length is selected which produces the best combination of total delay, short queues, and high probability of entering the intersection during the first green phase. The selection of the optimum cycle length requires good judgment and intuitive thinking on the part of the traffic engineer. Proper selection of cycle lengths is usually associated with traffic engineers who have had a great deal of experience in the field of traffic engineering.

Preliminary Computations for Selection of Cycle Length

The following preliminary computations are necessary prior to evaluating the three criteria for the selection of a cycle length.

Amber Time. The length of amber time usually ranges from three to six seconds, depending on the approach speed, stopping time, and the width of the roadway to be crossed. A formula which adequately accounts for these variables is (4):

$$t_A = t_D + Vo/_{2a_2} + (W + L)/Vo$$

in which

t_A = amber interval, in seconds

t_D = reaction - decision-making time of driver, in seconds

V^o = approach speed of vehicle, in feet per second

W = width of intersection to be traversed, in feet

L = vehicle length, in feet

a_2 = constant rate of braking deceleration, in feet per second per second (if the coefficient of friction is assumed to be 0.5 then a_2 becomes 16.1 feet per second per second from the equation $Wf = \frac{W}{g}a$)

The component of the formula $\frac{Vo}{2a_2}$ represents time to traverse stopping distance at uniform speed Vo and $\frac{W + L}{Vo}$ represents time to cross the intersection. The component of the formula t_D is as described above and is usually considered to be approximately 0.8 seconds (7).

The formula above represents the condition where the signal turns to amber when the vehicle is at a point in the approach lane. At this point the vehicle could stop before entering the intersection, but instead it continues at a uniform speed Vo to and across the intersection.

After the amber interval for an intersection approach has been determined, it must be decided during how much of the amber phase traffic flow occurs. Any portion of the amber time in which traffic flows must be included in effective green time and is a factor in computation of delay, queues, and entering probabilities. The remainder of the amber time interval is included as lost time. It seems reasonable to say also that the amount of amber time used for traffic flow will vary during the day, depending upon the density of traffic and other factors.

Headways of Vehicles Entering Intersections. When a signal turns green there is usually an initial time lag or starting delay for a queue of vehicles to get started moving through an intersection, followed by nearly equal time gaps between vehicles as long as a continuous supply of vehicles is maintained.

This sequence indicates two time components: the time lost in getting the traffic stream moving, and a nearly constant rate of saturated flow at time headways of approximately 2.1 seconds. The starting time appears to be a function of the alertness of drivers as well as the accelerating capabilities of vehicles. During peak hours, and at signals that are chronically overloaded, drivers usually cut their starting time to a minimum - even by anticipating the green light. In the off peak hours when signals are not likely to be overloaded, drivers are less alert and less hurried.

A usable value for starting time, lag and saturation flow rate can usually be obtained in the specific locality by making observations with a stop watch.

Apportionment of Green Time. Before a given cycle length can be evaluated it is necessary to determine how much of the total time available at the intersection should be apportioned to each flow of traffic movement. The selection and arrangement of simultaneous flows is known as "phasing" (8). The time available for distribution to various phases is the cycle length minus the fixed time elements of the cycle, i.e., those which are independent of cycle length.

The available green time (gt) may be expressed as follows (4):

$$gt = C - R - a_1 - a_2 - \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot - a_n - nl$$

where

gt = total green time available for distribution, seconds

C = cycle length, seconds

R = total all red time or pedestrian interval, if any, seconds

a_1 = length of amber, phase 1, seconds

a_2 = length of amber, phase 2, seconds

a_n = length of amber, phase n, seconds

n = number of green phases

nl = lost time per phase due to starting delay, seconds

The distribution of the total green time to each phase is made in proportion to the ratios of maximum approach lane flow to the saturated flow. These ratios are expressed as (4):

$Y = V/S$ where

V = volume in the lane of maximum flow for one phase, vehicles per second

S = saturated flow, vehicles per second, i.e., the maximum rate of flow that could be discharged during one green phase

After the total green time has been proportioned, the starting delay is added to obtain the number of seconds for one full green phase, as follows:

$$g_1 = \frac{Y_1}{Y} gt + 1$$

$$g_2 = \frac{Y_2}{Y} gt + 1$$

$$g_n = \frac{Y_n}{Y} gt + 1$$

where

g_1, g_2, g_n = length of green phases, seconds

Y_1, Y_2, Y_n = ratio of design flow to saturated flow for the most saturated approach lane during each phase, seconds

Y = sum of Y_1, Y_2 . . . . . Y_n

n = number of phases

If the saturation flows were the same in all approaches, the design volumes could be used in the above formulas instead of flow rates.

Determination of Delay. Cycle Length for Minimum Delay - An approximate formula (referred to in reference 4) for optimum cycle length in terms of minimum delay is as follows:

$$Co = \frac{1.5\ L + 5}{1 - Y}$$

where

Co = optimum cycle length, seconds

L = nl + R (assumes amber time as green time)

and

n = number of phases

l = average lost time per phase (usually starting delay only), excluding any all-red periods

R = time during each cycle when all signals display red, including separate pedestrian intervals, if any

Y = summation for the whole intersection of the y values corresponding to each phase where y = ratio of design flow to saturation flow ($\frac{V}{S}$) for a given phase.

The cycle length obtained with the above formula closely approximates the optimum for minimum delay under the given conditions of lost time and relative saturation.

Delay per Vehicle - Webster's formula for evaluating average delay per vehicle, with some modifications in form and nomenclature is (4):

$$d = (cA + \frac{B}{V}) (\frac{100 - c}{100})$$

where

d = average delay per vehicle in seconds (for a given approach lane)

c = cycle length in seconds

V = approach volume in vehicles per second per lane = $\frac{V_L}{3600}$ where V_L = number of vehicles per hour in the lane of maximum flow for an approach

A = term depending on λ and x_s (Table 2 in reference 4)

B = term depending on x_s (Table 3 in reference 4)

C = term depending on λ , x_s and M (Table 4 in reference 4)

λ = proportion of cycle length which is effectively green for a particular approach = g^1/c

g^1 = "effective green" = $g + a - 3.5$ seconds

g = green interval for approach, seconds

a = amber interval for approach, seconds

s = saturation flow or maximum rate of discharge per lane in vehicles per second from queue during the green period = $1/G$ where G = seconds of headway between vehicles at maximum (saturation) flow to be obtained from field data.

x_s = degree of saturation, or ratio of actual average flow to maximum flow for an approach lane = $V/\lambda s$

M = average number of vehicles arriving per cycle in an approach lane = vc

Total delay per lane is computed as follows:

$$D_T = d \frac{Vact}{3600}$$

in which

D_T = total delay per lane, in vehicle-hours per hour of signal operation

d = average delay per vehicle per hour, in seconds

Vact = actual volume of vehicles in the design hour

The actual delay criteria by which the signal timing should be judged is the total delay for the entire intersection. This is obtained by adding the total delay (D_T) for all approach lanes at the intersection.

It can readily be seen that manual techniques for computing delay would be awkward and time consuming. Therefore, computer programs have been written for speed of computation (4).

<u>Length of Queue.</u> The length of a line or queue of waiting vehicles at a signalized intersection is of primary importance in urban areas where the intersections are close together. The lines of waiting vehicles may be subjected to excessive delay and may also block cross traffic at intersections.

An approximate formula for the average length of queue at the beginning of the green period is as follows (4):

$N = \frac{vr}{2} + vd$, or $N = vr$ (whichever is larger) where

N = number of vehicles in queue

v = rate of flow, in vehicles per second

r = red phase, in seconds

d = average delay per vehicle, in seconds

<u>Probability of Arriving Vehicles Entering the Intersection During Their First Green Phase.</u> Usually the drivers in a traffic stream desire to get through a traffic signal on the first green light. Therefore, another criterion for determining cycle length is make the green interval long enough so that nearly all the vehicles will pass through the intersection on the first green interval.

Use is made of the Poisson probability distribution to express the number of vehicles that will arrive at an intersection in a given time interval. The Poisson distribution formula is as follows (4):

$$P_{(x)} = \frac{m^x e^{-m}}{x!}$$

where

$P_{(x)}$ is probability of exactly x vehicles arriving in any time interval t.

m = average number of vehicles arriving in t

$x!$ = x factorial

e = base of natural logarithms

The preceding computations are made over the desired range of cycle lengths using amber time as green time and then using amber time as lost time. The optimum cycle length is chosen, which, in the judgment of the engineer, best fits the conditions of traffic characteristics.

Detailed examples of the above method of cycle length selection are given in reference 4. The selection of the optimum should be made with all characteristics of the intersection and traffic flow well in mind. This requires that the traffic engineer be experienced in the field of signal timing and theories of traffic flow. Again, it should be noted that after a cycle length has been selected and the system put into operation, frequent checks of the installation should be made to insure that the timing is adequate for the traffic demand.

METHOD II - FIXED-TIME SIGNAL CYCLE LENGTH SELECTION

This method of cycle length determination is essentially the method as described in the 1950 edition of the Traffic Engineering Handbook (7).

The method is based upon the observed or design traffic volumes, spacings between vehicles, and the observed or design speeds. The total cycle time, T, is a combination of the time necessary to accommodate the maximum traffic movements in the intersecting directions, the delay time necessary for a vehicle to accelerate from rest when the signals change, and the required yellow light clearance.

Some of the factors which must be considered in assigning green time to the intersecting streets are (1):

1. Number of traffic lanes and other physical conditions.
2. Volume of traffic in the critical lanes.

3. Requirements of commercial and public transit vehicles.

4. Vehicle headways on the intersecting streets.

5. Pedestrian crossing requirements.

6. Vehicle and pedestrian clearance requirements.

7. Turning movements.

For a simple intersection with four approaches and a two-phase signal installation, the terms used in the timing formula are as follows (7):

$$T = \frac{Y_1 + Y_2 - S_1 - S_2 + D_1 + D_2}{1 - 0.0033(N_1S_1 + N_2S_2)}$$

where

T = total cycle length, seconds

N_1 = number of vehicles on the major street entering the intersection from one approach in the peak five minutes in one hour

N_2 = same as N_1 except for minor street

S_1 = average time spacing in seconds between vehicles as they enter the intersection in the direction corresponding to N_1

S_2 = same as S_1 except in the direction corresponding to N_2

D_1 = delay time in seconds for the first vehicle in a line to move into the intersection corresponding to the direction N_1

D_2 = same as D_1 except in the direction corresponding to N_2

G_1 = the number of seconds of green light in the direction of N_1

G_2 = the number of seconds of green light in the direction of N_2

Y_1 = the number of seconds of yellow light between the green and red light in the direction of N_1

Y_2 = same as Y_1 except in the direction of N_2

The factors for determining T, G_1, and G_2 in the above list are usually not known, therefore, some studies need be made to determine values required to solve the timing equation. (See Reference 7, p. 228)

Factors N_1 and N_2 - Actual vehicle counts should be taken for each approach at the specific intersection under study. Traffic counts by 15-minute intervals, showing the movement into the intersection from each approach, should be used to determine the values of N_1 and N_2. Values of N are for all traffic in the given movement regardless of the number of lanes and includes turning vehicles. The counts should include the periods of peak traffic flow and least traffic flow to determine whether the signals should be operated on the same cycle throughout the day or changed during the day to more nearly meet the traffic demand.

Factors S and D - The values of S_1, S_2, D_1, and D_2 must correspond to the movements N_1 and N_2. The factors can be obtained by observing traffic movement and obtaining the time interval for a certain number of vehicles to pass a certain point selected by the observer. If the intersection

is already signalized, a chart (Fig. 3)
showing Stored Cars Vs. Time After Green to Enter the Intersection can be plotted. The time-spacing and initial delays can be obtained by calculation from the chart. If the intersection is not already signalized, the traffic could be controlled manually and released at regular intervals similar to signalized operation in order to obtain the required data.

Factor Y - The yellow time is dependent upon the variables as follows:

Speed - The speed of the vehicle that is approaching the intersection. The speed can be obtained by a common method of checking vehicle speeds and should be taken at a point a sufficient distance from the intersection to obtain the speed in free-moving traffic, i.e., where the speed is not influenced by acceleration or deceleration at intersections.

Reaction distance - The distance required for the vehicle operator to decide whether to stop or continue on through the intersection on the yellow light.

Total braking distance - The distance required for the vehicle to decelerate from speed Vo, to a complete stop.

Crossing distance - The distance from the stop line on the near side of the intersection to the curb line on the near side plus the intersecting street width plus the length of the vehicle.

When these factors are obtained, the total cycle time (T) and the green times for the intersecting streets can be computed. The total cycle time (T) is as follows:

$$T = G_1 + Y_1 + G_2 + Y_2 \qquad \text{(equation 1)}$$

where $$G_1 = (\frac{N_1}{300}) S_1T - S_1 + D_1 \qquad \text{(equation 2)}$$

$$G_2 = \frac{N_2}{300} S_2T - S_2 + D_2 \qquad \text{(equation 3)}$$

Now by substituting the values of G_1 and G_2 into the equation for T and solving for T we obtain

$$T = 0.0033N_1S_1T - S_1 + D_1 + Y_1 + 0.0033S_2N_2T - S_2 + D_2 + Y_2$$

$$= 0.0033(N_1S_1 + N_2S_2)\ T - S + D_1 + Y_1 - S_2 + D_2 + Y_2$$

$$T = \frac{Y_1 + Y_2 - S_1 - S_2 + D_1 + D_2}{1 - 0.0033(N_1S_1 + N_2S_2)} \qquad \text{(equation 4)}$$

After T is obtained, the values of G_1 and G_2 can be computed from equations 2 and 3 respectively.

Equation 4 will give the required cycle length for the intersection based upon the traffic volumes, spacings between vehicles, and the observed speeds. When a cycle length has been computed it is advisable to round off the cycle length to the nearest five seconds. The reason for this is that the manufacturers of signal units have standard gear boxes made up in multiples of five-second intervals and a special gear for an odd interval cycle length is quite costly.

The calculated green time for each intersection approach should be compared with the minimum green time necessary to allow a pedestrian to cross the intersection. The minimum green time required for a pedestrian to cross an intersection is as follows:

$$\text{Minimum Green} = \frac{W}{V} + 5 \text{ sec.} - \text{Yellow Time}$$

where

W = width of street, in feet

V = walking speed of pedestrian, usually 3 to 5 feet per second

The example which follows will best illustrate the use of the timing formula.

<u>Example No. 2.</u>

Determining Cycle Length and Signal Timing.

Compute the desirable minimum length of cycle, length of yellow time and length of each green time for the following traffic conditions assumed to exist at the intersections of Streets A and B (shown in Fig. 4). The values shown are for $3N_1$, $3N_2$, D_1, D_2, S_1, and S_2.

Assume the 85th percentile speeds of approach were 22 m.p.h. "A" Street is 54 feet curb to curb; "B" Street is 40 feet curb to curb. Property lines are 15 feet back of curb lines. The vehicle volumes shown are in equivalent vehicles, i.e., the volumes have been corrected for turning movements, etc.

Solution:

The timing equations to be used are:

$$T = G_1 + Y_1 + G_2 + Y_2$$

$$G_1 = \left(\frac{3N_1}{900}\right)S_1T - S_1 + D_2$$

$$G_2 = (\frac{3N_2}{900})S_2T - S_2 + D_2$$

therefore

$$T = \frac{Y_1 + Y_2 - S_1 - S_2 + D_1 + D_2}{1 - 0.00111(3N_1S_1 + 3N_2S_2)}$$

The initial delays D_1 and D_2 and the average time spacing between vehicles S_1 and S_2 are given, therefore Y_1 and Y_2 must be computed.

The yellow time is given as:

Yellow Time "Y" =

$$\frac{\text{Reaction Distance + Total Braking Distance + Crossing Distance}}{\text{Speed}}$$

Braking reaction time - assume 0.8 seconds (7)

Reaction distance = 0.8(22) (1.47) = 25.9 feet

Braking Distance - a coefficient of friction for deceleration of 0.5 is assumed

$$Wf = \frac{Wa}{g}$$

Solve for a: $a = fg = 0.5(32.2 \text{ ft/sec}^2) = 16.1 \text{ ft/sec}^2$

Braking Distance $S = \frac{Vo^2}{2a}$

$$S = \frac{(22\text{mph})^2(1.47)^2}{2(16.1)\text{ft/sec}^2} = 32.5 \text{ feet}$$

Crossing Distance - assume length of vehicle is 18.0 feet (12)

Street "A" crossing distance = 15' + 40' + 18' = 73 feet

Street "B" crossing distance = 15' + 54' + 18' = 87 feet

therefore

$$Y_1 \text{ (Street A)} = \frac{25.9' + 32.5' + 73.0'}{22(1.47)} = 4.1 \text{ sec.}$$

$$Y_2 \text{ (Street B)} = \frac{25.9' + 32.5' + 87.0'}{22(1.47)} = 4.5 \text{ sec.}$$

Now

$$T = \frac{4.1 + 4.5 - 1.7 - 2.4 + 4.0 + 5.0}{1 - 0.00111\ ((275)(1.7) + (100)(2.4))} = \frac{14.5}{.2147} = 67.54 \text{ sec.}$$

Total cycle (rounded to nearest 5 sec.) = 70 seconds

To obtain the green times, substitute the value of T into the respective green time equations.

$$G_1 = \frac{3N_1}{900}S_1T - S_1 + D_1 = \frac{275}{900}(1.7)(70) - 1.7 + 4.0 = 38.7 \text{ sec.}$$

G_1 = 40 seconds - Green time for Street "A"

$$G_2 = \frac{3N_2}{900}S_2T - S_2 + D_2 = \frac{100}{900}(2.4)(70) - 2.4 + 5.0 = 21.3 \text{ sec.}$$

G_2 = 20 seconds - Green time for Street "B"

The timing for the intersection would be as follows:

T = 70 seconds

G_1 = 40 seconds

G_2 = 20 seconds

Y_1 = 5 seconds

Y_2 = 5 seconds

Now the minimum green time required for pedestrian crossing is calculated:

Minimum Green Time Street "A" =

$$\frac{54 \text{ ft.}}{4 \text{ ft/sec}} + 5 \text{ sec} - 5.0 \text{ sec} = 13.5 \text{ seconds}$$

Minimum Green time Street "B" =

$$\frac{40 \text{ ft.}}{4 \text{ ft/sec}} + 5 \text{ sec} - 5 \text{ sec} = 10.0 \text{ seconds}$$

It is seen that the computed green times are adequate for pedestrian traffic.

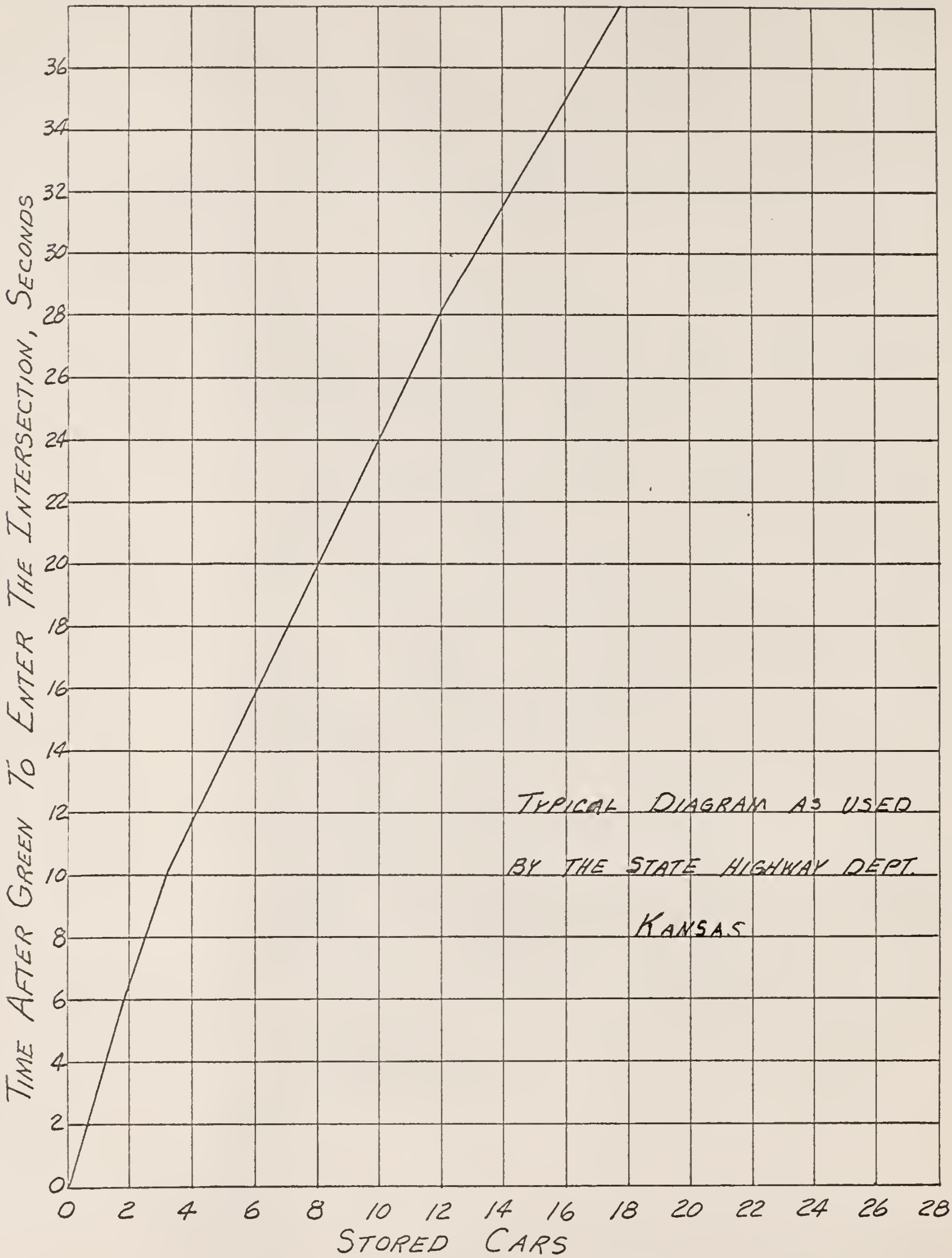
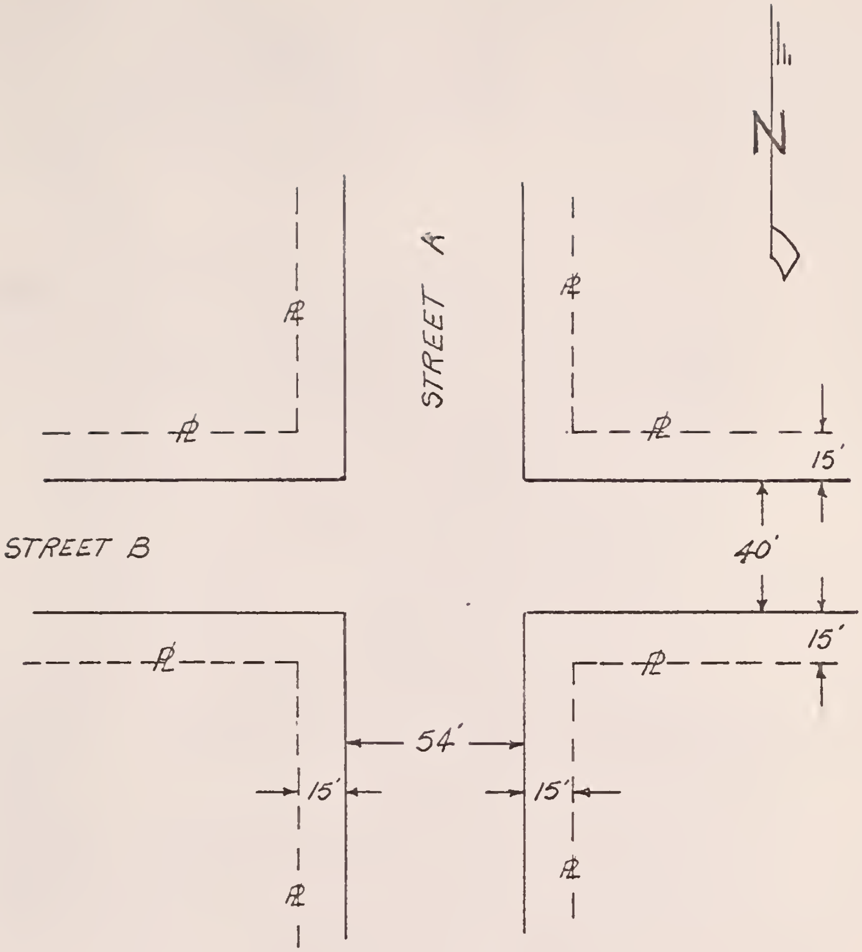


FIGURE 3 STORED CARS VS. TIME AFTER GREEN TO ENTER THE INTERSECTION.



PEAK 15 MINUTE ENTERING VOLUMES:

N.B. ON "A" STREET	275 VEHICLES = $3N_1$
E.B. ON "B" STREET	100 VEHICLES = $3N_2$
D_1 = 4 SEC.	S_1 = 1.7 SEC.
D_2 = 5 SEC.	S_2 = 2.4 SEC.

FIGURE - 4 INTERSECTION DETAILS FOR EXAMPLE PROBLEM.

SIGNAL TIMING WITH PROGRESSION IN TWO-WAY SYSTEMS

In general, all pretimed signals within one quarter of a mile of one another and controlling the same traffic should be operated in coordination (1). At times, coordination may be desirable at even greater distances under certain conditions.

When traffic movement is permitted to flow on a street in both directions, it becomes necessary to provide a signal timing system that will provide essentially continuous flow. Great inconvenience and undue delay is burdened upon the traffic stream if independent, non-interrelated operation of closely adjacent signals is allowed. Coordination of signals may be achieved by the operation of two or more signalized intersections as a system in which there exists a desirable and definite time relationship among the diverse phases at all intersections. In order for the system to operate efficiently, the flow of traffic on a given phase of movement at one intersection must be accommodated by a green light on its arrival at the next signalized intersection. The efficiency of any signal system will depend to a great degree on varying block lengths, conditions of traffic flow, and physical aspects of the street and intersections.

Signal Systems

The most useful classification of traffic control signal systems is based on their method of coordination. On the basis of classification, there are four general types of coordination at pretimed signals. These are defined as the simultaneous system, the alternate system, the limited progressive system, and the flexible progressive system (1).

The method (7, 9, 10) to be illustrated is a graphical method of timing progressive systems. This method is useful for determining timing offsets for equal speed and band widths in both directions and it also assures optimum timing conditions by adjusting deviations of "lead" and "lag" time so they are minimized. The graphical method is based on the preparation of a master chart (7) showing uniform signal spacing for identical two-way speed progression. The actual signal spacing is superimposed on the master chart and the optimum speed and band width is obtained. This method is trial and error only in the sense that different cycle lengths should be tried to obtain the optimum timing sequence for the signal system. However, for any cycle length selected, the solution is obtained directly and without the trial and error calculations that are required in mathematical solutions.

First, a master chart must be constructed as shown in Fig. 5. The vertical scale in miles per hour is given for three cycle lengths which may be extended as desired or intermediate cycle periods may be inserted by interpolation. The basic horizontal scale is distance in feet. (A suitable scale in practice is 1" = 1000'.) Now, upon this system of coordinates is plotted a series of "½-cycle" lines.

For the vertical scale in Fig. 5, one can use any cycle length to obtain the half-cycle lines, then the other cycle times which are to be considered can be plotted using appropriate ratios. For example, a 20 m.p.h. speed for a 60-second cycle would be equivalent in terms of linear distance to a 24 m.p.h. speed for a 40-second cycle. Table 1 shows the half-cycle relationship between the various cycle times and emphasizes that the distance traversed in each case is equal.

Table 1. Speed vs. cycle time for fixed distances.

Cycle time (sec)	Half-cycle time (sec)	Speed mph	Speed ft/sec	Distance (ft)
60	30	20.0	29.3	880
50	25	24.0	35.2	880
40	20	30.0	44.0	880

The horizontal scale in Fig. 5 is in feet. The points along each half-cycle line are plotted at a distance traveled at the corresponding speed during one-half of the period of the corresponding cycle. Thus along the top line of Fig. 5, the distance between half-cycle lines is 1100 feet which is the distance traveled in 30 seconds (one-half of the 60-second cycle) at 25 m.p.h., or the distance traveled in 25 seconds (one-half of the 50-second cycle) at 30 m.p.h., or the distance traveled in 20 seconds (one-half of the 40-second cycle) at 37.5 m.p.h. The distance between half-cycle lines along the bottom of Fig. 5 is 550 feet.

Further explanation of the chart will be illustrated by application to an example problem.

Example No. 3. Progression in Two-way Systems.

The example to be illustrated is a progressive timing problem for the street system as shown in Fig. 6. The proposed signal locations are plotted on the progressive timing chart with the "A" intersection located on the first half-cycle line. The first half-cycle line is considered as an index for all vertical movements. It is usually advised to plot the street intersections upon a straight edge. The straight edge is then placed upon a horizontal line of the chart somewhere within the desired range of speed on the preferred cycle. The desired range of speed may be obtained from field studies or it may be the speed deemed most desirable for the conditions of the street system. In an initial trial toward solving the example problem, assume the desired speed is 19 m.p.h. on the 50-second cycle scale. The straight edge is then set on the horizontal line corresponding to 19 m.p.h. This setting is shown in Fig. 6.

Along the 19 m.p.h. line in Fig. 6, the distance between each of proposed signal location points and the nearest half-cycle line measures, in feet, the distance between the signal location and the point where opposing flow bands of traffic will intersect. The distance between half-cycle lines is 25 seconds because a 50-second cycle was chosen. Now, the distance between signal location points and the half-cycle lines will be one-half the time interval between corresponding lines of opposing flow bands at the signal locations. This is all the data that is needed to construct the Time-Space Diagram for this signal system. However, due to the presence of unequal block lengths, it is noted that the signal locations do not fall on half-cycle lines. Therefore, the flow bands in opposite directions will not be of equal width which is desired for equal flow in opposite directions. The flow bands can be made equal by adjusting the deviations from the half-cycle lines so that they are equal. This is done by moving the straight edge to the left or right, whichever is required. In this case, it is noted that the greatest deviation of a signal location from the half-cycle line is at signal location "H" and this deviation is to the right. The greatest deviation to the left is at "E". Therefore, to balance the deviations, the straight edge should be shifted to the right two and one-half seconds. This shifting, in effect, moves the proposed signal locations to the right the distance which is equivalent to a time interval of two and one-half seconds. Since the maximum and minimum deviations from half-cycle lines are now equal, we have equal flow bands in opposing directions. The Time-Space Diagram has been constructed as shown in Fig. 7, with the deviations made equal at signal locations E and H, (i.e., the signal locations shifted to the right).

To construct the Time-Space Diagram, the points at which the half-cycle lines intersect the 19 m.p.h. line on the Progressive Timing Chart are projected downward to locate the points of intersection of the opposing flow bands. The parallelograms of flow bands are properly established because the points of intersection occur at intervals of half-cycle periods. The width of the flow band depends upon the duration of green interval available for a progressively timed street.

The maximum time difference between corresponding lines of opposing flow occurs where the greatest irreducible deviation remains after the system has been balanced laterally. In the case illustrated, the maximum deviation is at location E and H and the time difference is 15 seconds. The difference between the half-cycle time and the maximum deviation time difference is the flow band width. This system has a flow band width of 10 seconds, or 20 per cent of the time cycle. This represents a system of low efficiency and indicates that the 19 m.p.h. speed chosen may not be the optimum speed.

The optimum arrangement occurs when the sum of two critical deviations is at a minimum. This optimum condition can be obtained by moving the straight edge vertically in either direction until the sum of the largest deviation to the left and the largest deviation to the right of the half-cycle lines is a minimum. When the optimum speed is found, the Time-Space Diagram is constructed in the same manner as before.

To illustrate the procedure for obtaining the optimum speed which gives the greater band width, Fig. 8 and Fig. 9 have been constructed. In the Progressive Timing Chart each successive one-half cycle line to the right slopes more acutely than the one to the left. Therefore, as the system of signal locations is moved upward or downward on the chart, the deviations farthest to the right will change more rapidly than the ones to the left of the chart.

In Fig. 6 it is noted to minimize the deviations of locations E and H, the system of signal locations should be moved downward. As the system is moved downward the deviation at E increases. At 17.5 m.p.h. the deviation at E is to the right and the deviations at F, G, and H, which were to the left, are now zero. Therefore, the critical deviation is at E and this is the sum total of the critical deviations as there are no deviations to the left.

In Fig. 8, to equalize the deviation, the signal is shifted to the left a distance of one-half of the deviation at E, as shown in Fig. 9. Now, the deviations at A, E, and F are equal and it is seen that a zero deviation can be a "critical" deviation. In the same manner as in Fig. 7, the standard type of time-space diagram is constructed from the Progressive Timing Chart. In Fig. 9, the band width is shown as 14 seconds, which gives an increase of eight per cent over the timing at 19 m.p.h.

The signal system can be timed using any cycle length desired since interpolation between cycle time lines produces equally good results. However, most signal cycles are in multiples of five seconds.

There may be more than one optimum condition for a proposed plan of signal spacing within the range of a prepared master chart. It is usually up to the traffic engineer to decide which optimum condition best suits the traffic flow characteristics of the system. However, after a signal cycle

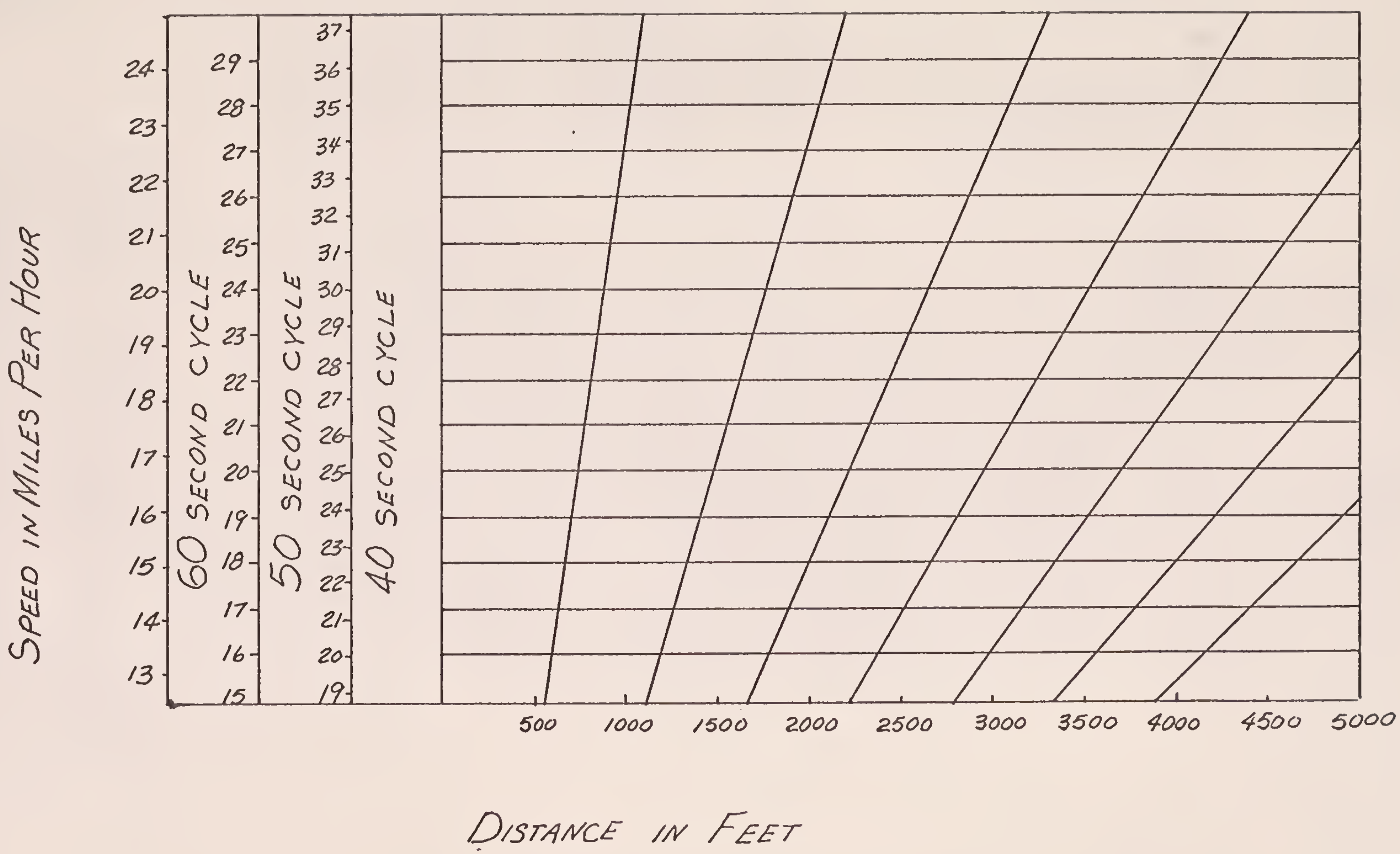


Figure 5 Master Chart for Graphical Determinations of Timing Diagrams

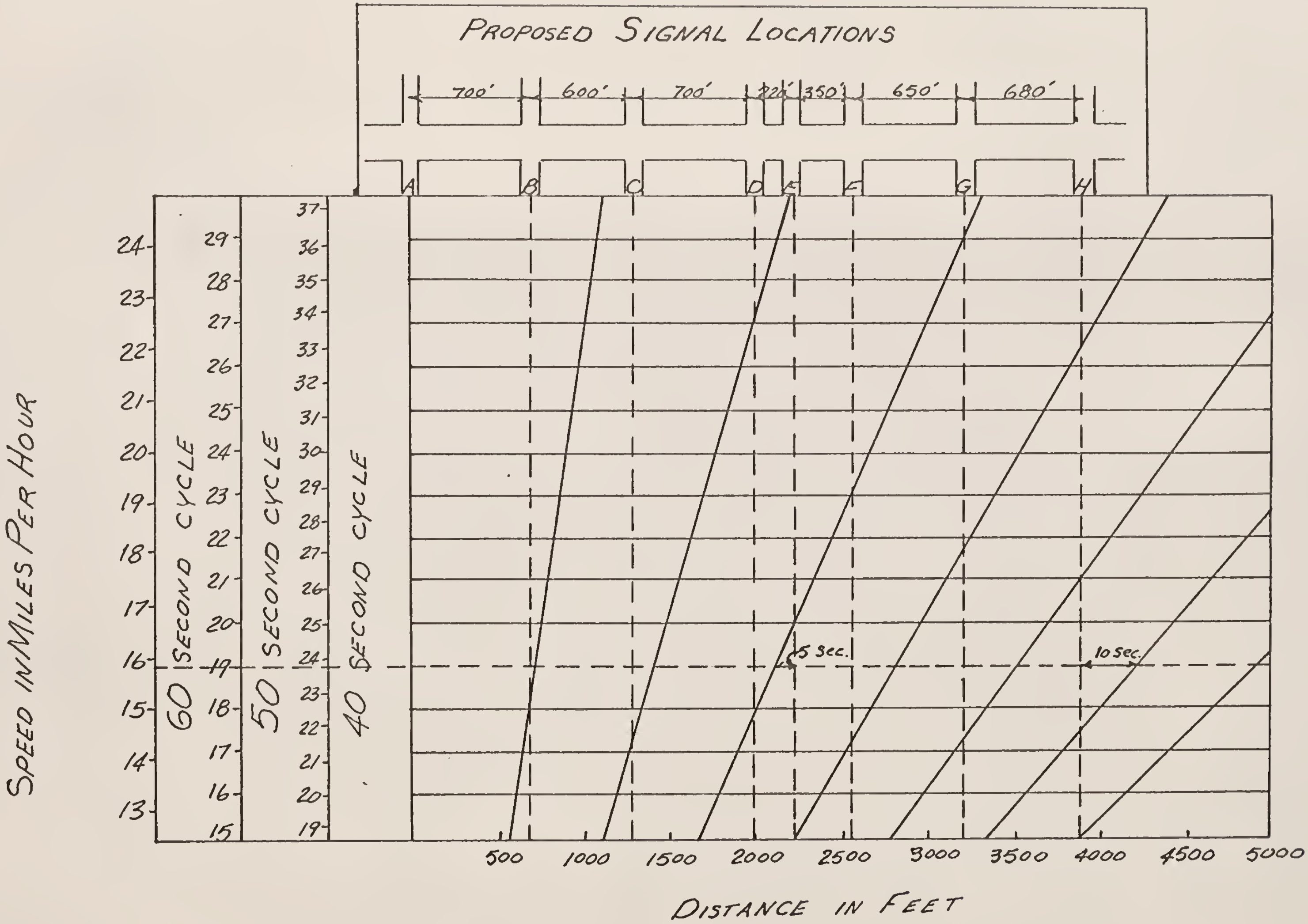
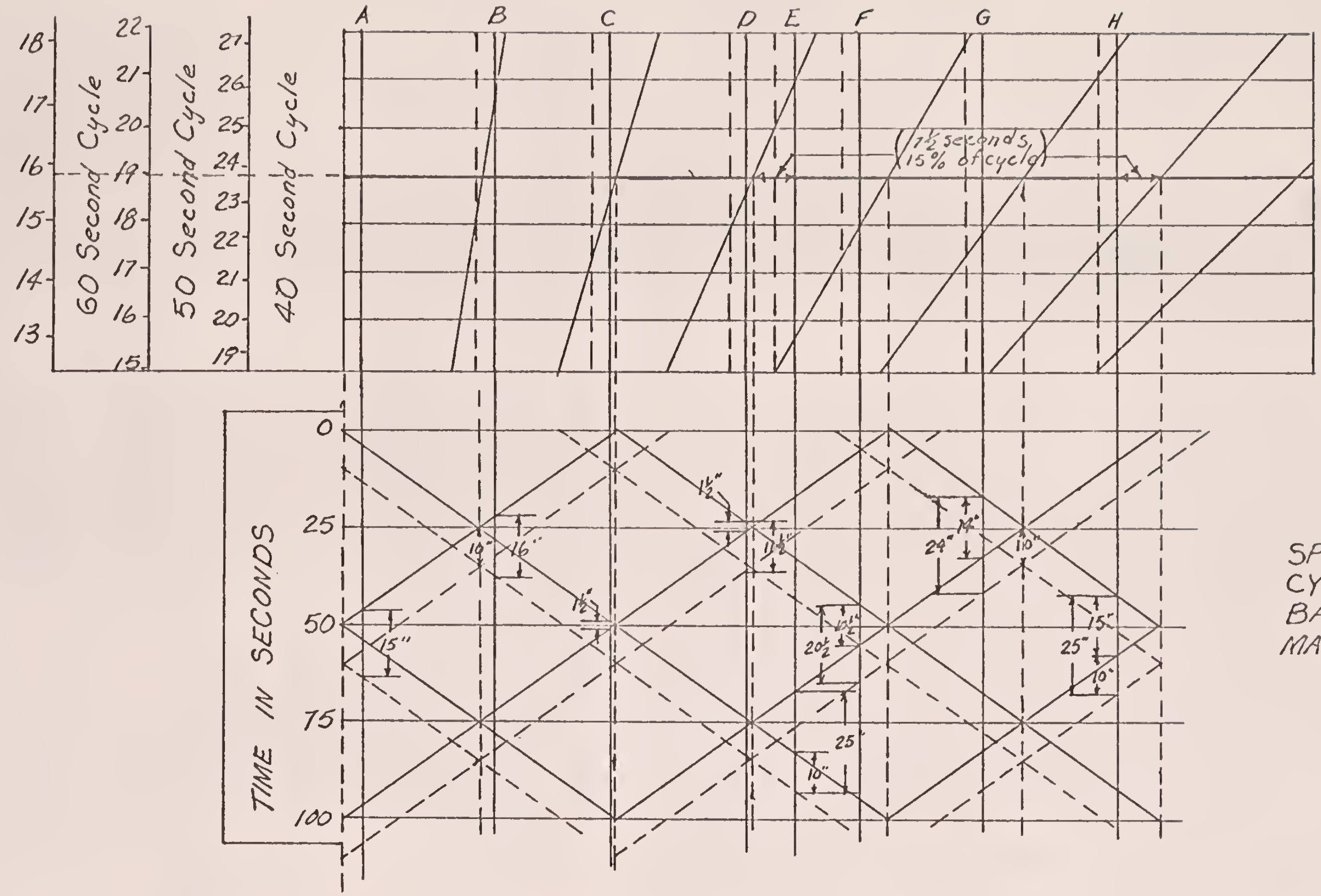


FIGURE 6 TRAFFIC SIGNAL - PROGRESSIVE TIMING CHART



SPEED	19 M.P.H.
CYCLE	50 SEC.
BAND	10 SEC.
MAX. GREEN	25 SEC.

THE SYSTEM OF SIGNALS HAS BEEN SHIFTED LATERALLY TO MAKE THE DISTANCE BETWEEN SIGNAL LOCATIONS E AND H AND THE HALF CYCLE LINES EQUAL.
THE PROGRESSIVE TIMING CHART WAS THEN PROJECTED DOWNWARD TO DEVELOP THE TIME-SPACE DIAGRAM.

FIGURE 7 TIME-SPACE DIAGRAM

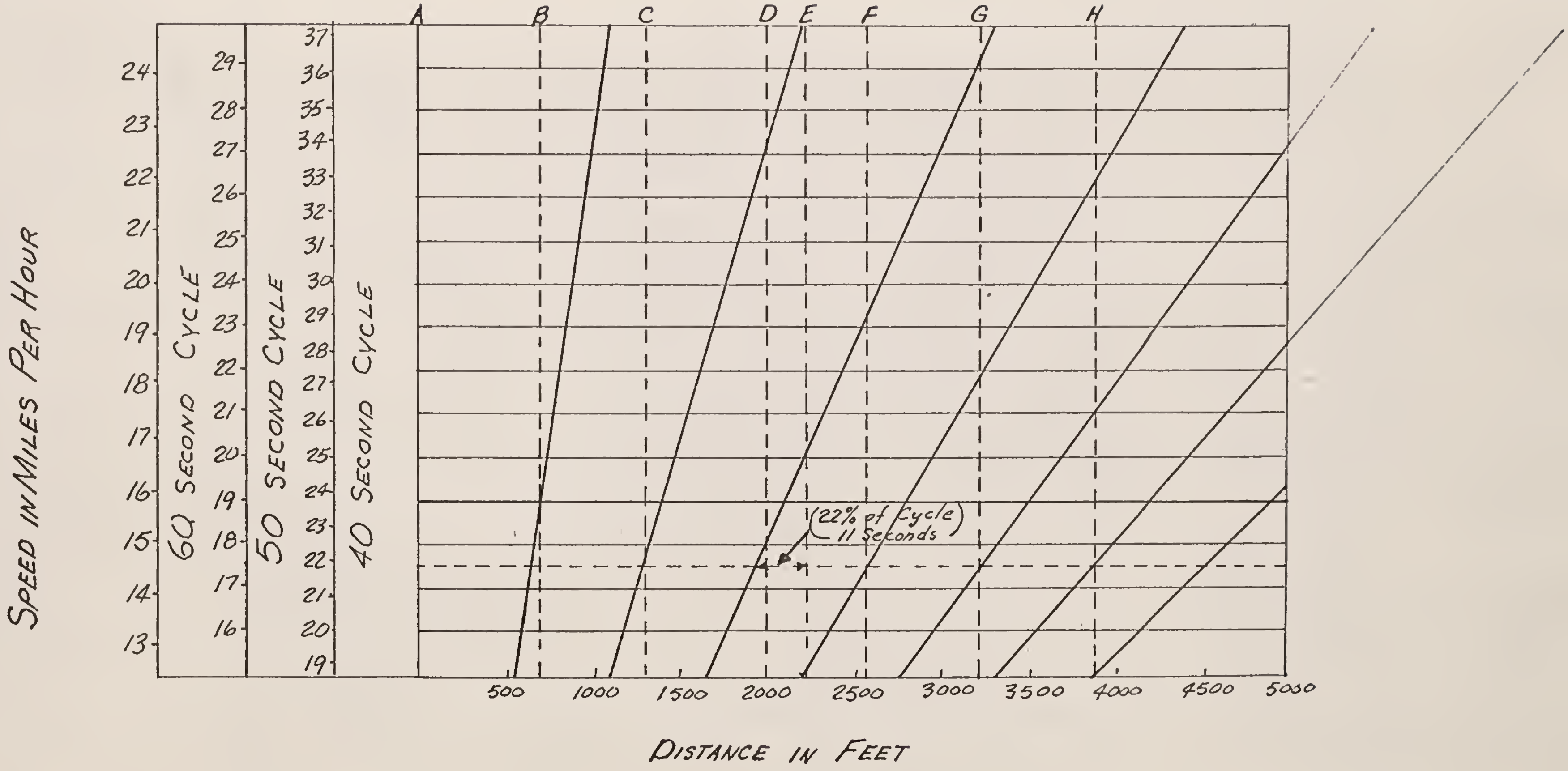


Figure 8 Traffic Signal - Progressive Timing Chart

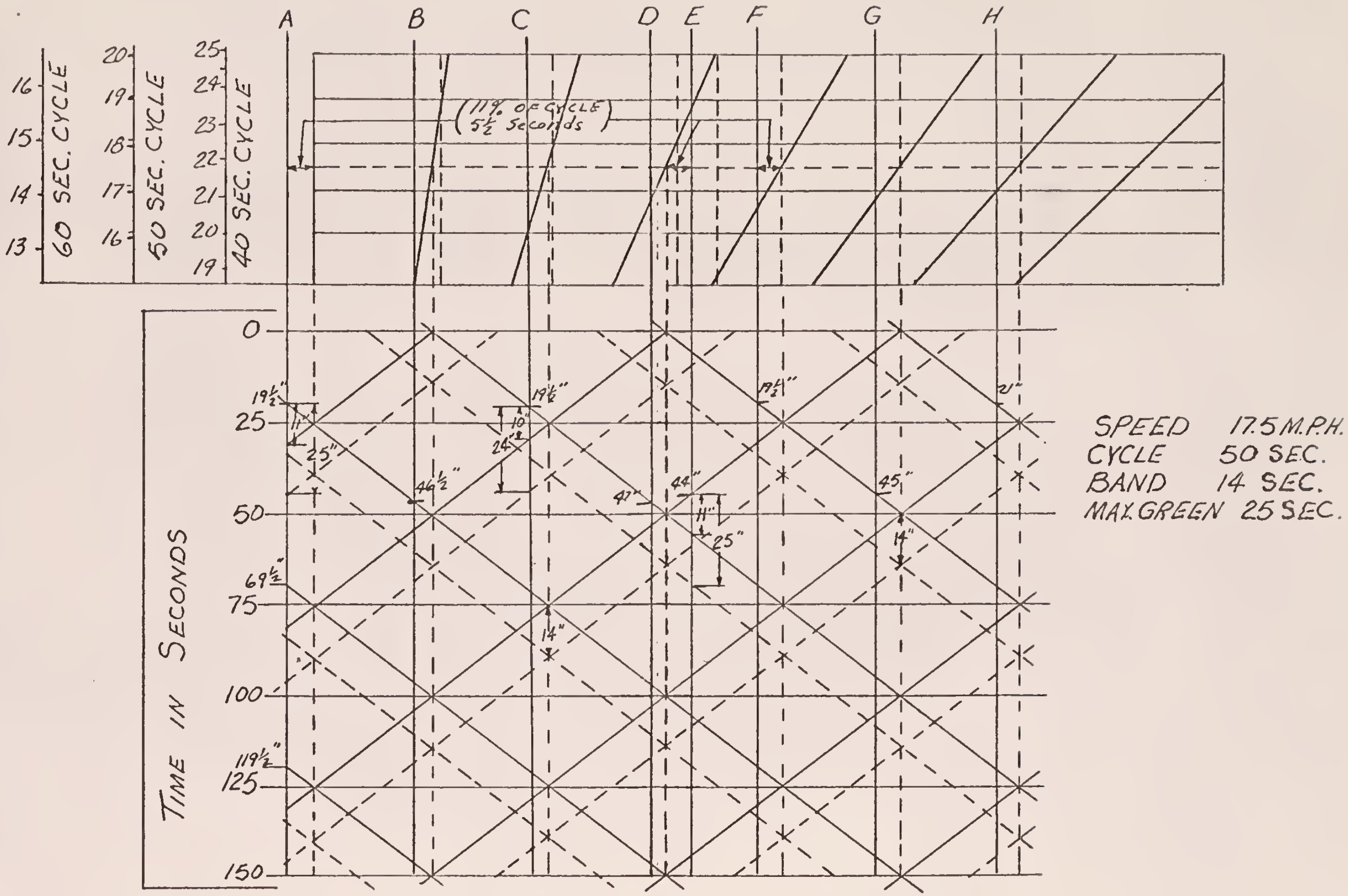


FIGURE 9 TIME-SPACE DIAGRAM

length has been chosen and the system put into operation, the traffic engineer should observe the operation to make certain the timing sequence handles the traffic. This is a requirement of all traffic engineers because a timing system that doesn't accommodate the demand of the traffic isn't worth a "hoot".

THE VALUE OF SIGNAL PHASE OVERLAP IN SIGNALIZED INTERSECTION CAPACITY

This section is intended to serve only as a reference to research and investigation which was done by the Texas Highway Department and the Texas Transportation Institute (11). The investigation was made in the interest of increasing signalized intersection capacity by the use of signal phase overlap.

The method of signal timing using signal phase overlap applies itself well to the multiple type intersections, such as signalization for the traffic from a shopping center to provide access to a major arterial or the conventional two-level diamond interchange.

The signalized intersection is a vital element in the street system of all cities and methods to increase the capacity of the system is the objective of all traffic engineers.

SUMMARY

This report has been presented to illustrate methods of cycle length selection and signal timing. The examples shown do not represent any specific type of intersection as each intersection to be signalized represents an individual study in itself.

The selection of signal cycle lengths using Method I in this report is a relatively new method. It is based on "levels of service" to the user rather than arbitrary values for capacity. The application of Method I is questionable for all instances of signal timing such as isolated intersections where the warrants for signalization may be at a minimum. It seems that to use this method of cycle length selection, a great amount of professional experience would be necessary in order for signalization to be accomplished properly.

The use of Method II, or variations of this method, is in wide use for signal timing. This may be due to the fact that it has been used widely in the past. The method is such that, analytically, an engineer or layman with little experience can solve a signal timing problem if the data for the location are furnished. The terms of capacity as defined in the Highway Capacity Manual (5) are somewhat arbitrary and hard to "pin point". Therefore, the needed change in methods and procedures has been recognized and revisions are in the process.

The graphical method of signal timing is a relatively easy and rapid solution to progressive timing of signal systems. This timing solution can be solved by inexperienced engineers as the sequence of procedures is straight forward and analysis of the Time-Space Diagram is readily apparent. Solutions to timing problems can be obtained much faster using the graphical solution rather than a mathematical solution. The mathematical solution requires a considerable amount of "trial and error" calculations. The example problem shown gives equal flow in both directions. In most cases traffic is found to be heavier in one direction than the other at certain hours and at other hours the situation is found to be reversed. Timing signals for unbalanced flow conditions can be solved quite easily with a graphical analysis.

ACKNOWLEDGMENTS

The author wishes to express his sincerest appreciation to his major instructor, Professor Bob L. Smith, for his assistance, guidance, and suggestions during the writing of this paper.

To the City Traffic Engineers of Wichita, Topeka, and Lawrence, Kansas, he offers his gratitude for their suggestions offered for this paper.

To Mr. Gerald Brickell of the Traffic Department of the State Highway Commission of Kansas, he offers a sincere thanks for advice and assistance in preparation of data.

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METHODS OF CYCLE LENGTH SELECTION
AND
SIGNAL TIMING

by

KURT ALLEN BOCE

B. S., Kansas State University, 1960

AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1964

The traffic signal is a valuable device for the control and safe facilitation of vehicle and pedestrian traffic. Because of its arbitrary or traffic-induced assignment of right of way to the various movements at intersections and at other street and highway locations, the traffic signal exerts a strong influence on traffic flow. It is of utmost importance that the selection and use of such a control device be preceded by a thorough study of roadway and traffic conditions by an experienced engineer. Equally important is the need for checking the efficiency of a signal or signal system, once in operation, to ascertain the degree to which the type of installation and the timing program meet the requirements of traffic, and to permit intelligent operating adjustments of the controls.

This paper deals with the phase of traffic engineering concerned with methods of selection of signal cycle length and timing of signals for individual intersections and street systems.

Some of the requirements of traffic signals are presented along with some advantages and disadvantages of signalized operation of street intersections. Also, some preliminary considerations for signal timing are given to introduce the methods of signal timing.

Two methods are given for cycle length selection for fixed-time signals at individual intersections. The first method is relatively recent and is based on three criteria of levels of service to the user. These three criteria are: delay per vehicle, expected queue length, and the probability of entering the intersection during the first green phase. In this method it is required that calculations be made for the desirad range of cycle lengths using amber as lost time and amber as green time. Then the cycle length is selected based on the judgement of an experienced traffic engineer who considers all the physical and operational conditions for the intersection under study.

The second method of cycle length selection is based upon the observed or design traffic volumes, spacings between vehicles, and the observed or design speeds. This method or some variation is in wide use by experienced traffic engineers. The total cycle time is computed and then apportioned to the intersecting traffic flows according to traffic demand. The calculations for this method are straight forward and easily applied to collected data.

The third method of cycle length determination is for the progressive timing of a signal system. The signal system is plotted on a Master Chart for Progressive Timing and graphically solved to provide a timing sequence im the desired range of signal cycle length. By selecting different cycle lengths and solving for a timing sequence, the optimum cycle length can be selected which fits the specific street system and traffic flow conditions.