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Capacity Study of Signalized Diamond Interchanges

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This paper presents a portion of the results from a research project on freeway ramps and interchanges which is presently being conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department. This study was designed to obtain traffic performance data which would have useful application in evaluating the capacity of signalized diamond interchanges. Library research and evaluation of existing design requirements indicated a great demand and need for this type of data.

The field data were gathered through the use of time-motion pictures which furnished a complete and simultaneous record of the traffic operations occurring in the intersections that were studied. The operational characteristics of over 4,000 vehicles were recorded on 16-mm film at two conventional-type diamond interchanges on the Gulf Freeway in Houston, Texas. These data were collected during the peak periods on approaches that were fully loaded.

This research was conducted to investigate the factors which could be used in developing criteria for evaluating the capacity of the various movements encountered in diamond interchange operation. Data on vehicle starting delays and time-headways at both interchanges were accurately measured to develop a basic approach to the determination of lane capacity. Special emphasis was placed on determining the capacity of a single-lane turning movement and a two-lane, or two-abreast-type turning movement.

The analysis of the study produced some significant results and provides the designer with current operational characteristics of vehicles at signalized diamond interchanges. Design procedures for diamond interchanges, based on lane capacity and signalization requirements, were developed and are presented in this report.

●IN URBAN AREAS traffic interchange between major arterials and freeways is frequently accomplished by the use of conventional-type diamond interchanges (Figs. 1 and 2). The simplicity of design, minimum right-of-way requirements, and economy of this type of interchange have greatly encouraged the use of diamond interchanges.

In contrast to simplicity of design the operation of a diamond interchange often becomes very complex. High volumes of cross-town major street traffic in combination with traffic interchanging to and from the freeway creates the need for a multiphase signal system to separate conflicting traffic streams. The signalization is further complicated by the proximity of the two at-grade intersections. These signalized intersections exert a capacity limitation which often results in the diamond interchange being incapable of handling the traffic demand.

Although the diamond interchange has many efficient applications, the operational

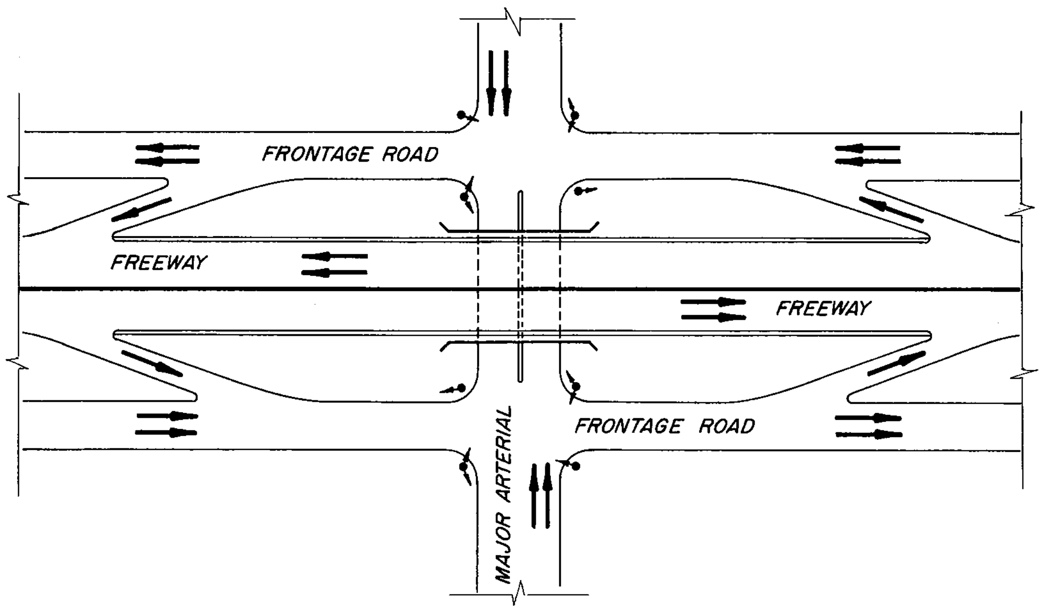

Figure 1. Diamond interchange conventional arrangement.

problems and capacity limitations of such an interchange must be recognized. The diamond interchange has been in existence for several years, but very little factual data on the operational characteristics and capacity of this type of interchange are available. The purpose of this study was to record data on the operational aspects of the diamond and to develop criteria for determining capacity, design, and signalization.

PROCEDURE

Sites

The Wayside Drive and Cullen Boulevard interchanges on the Gulf Freeway in Houston were selected for study sites. Both of these interchanges were conventional-type diamond interchanges and were known to carry large volumes of traffic (Fig. 3).

The Wayside interchange and the Cullen interchange (Figs. 4 and 5) had similar geometrics (Fig. 6). The major street approaches to both interchanges provided for two lanes of through traffic and a free right-turn lane. Each interchange was served by parallel, continuous one-way frontage roads which were 32 ft in width and operated with 3-lane flow during peak periods.

The traffic movements at each interchange were controlled by a fixed-time, multiphase signal system. The signal system used a three-dial operation with a separate dial provided to control the morning, the off-peak, and the evening peak periods of traffic flow.

At both interchanges, traffic operations at the two closely spaced intersections (300 ft center-to-center) were controlled by separate controllers which were interconnected for coordinated movement. Due to the lack of vehicle storage space between the intersections, the signal phasing (Fig. 7), as designed by engineers of the Department of Traffic and Transportation of the City of Houston, permitted vehicles to move through both intersections on receiving a green indication. With this phasing the only vehicles required to store between the intersections were a small percent of U-turning vehicles. The signal phasing also provided time separation for conflicting movements.

Figure 2. Gulf Freeway—Houston.

Collection of Field Data

Several methods of collecting data were evaluated to determine the best approach to the problem of obtaining a complete and simultaneous record of the traffic events occurring in the interchange area. The use of a 20-pen graphic recorder and manual counting methods were given consideration. However, in the final evaluation, the motion picture method was chosen as the best approach. This method required a minimum of field personnel and allowed the flexibility of enabling one to view and recreate all traffic events.

All of the traffic operational data were collected by filming traffic operations at each of the study intersections with a 16-mm motion picture camera. The filming was performed from a vantage point provided by a hydraulic platform truck similar to the one shown in Figure 8. The platform on this truck extended to a height of 35 ft and additional elevation was gained by taking advantage of the terrain. The truck was located in an inconspicious area and it is felt that the presence of the truck and photographer had little or no effect on the behavior of traffic in the intersection being filmed.

The movies at each study site were taken at a camera speed of ten frames per second which permitted accurate determination of vehicle time-headways and delay. Because it was desired to obtain data on possible capacity, the studies were conducted during both the morning and evening periods of peak flow on an average weekday.

As an aid to the determination of vehicle delays and time-headways from the motion pictures, reference lines were placed perpendicular to traffic lanes at each intersection approach. The purpose of these lines was twofold: (a) to regulate and fix the region where approaching vehicles would stop when waiting for a green indication; and (b) to aid in determining when each vehicle entered the intersection area.

Data Tabulation

Data on traffic operations were extracted from the film through the use of a specially constructed projector (Fig. 9). A special control attached to the projector permitted the film to be advanced or reversed in single-frame increments and an interconnected frame counter allowed the operator to determine the number of frames between specific events on the film. By using the constant camera speed, elapsed time between events could be determined.

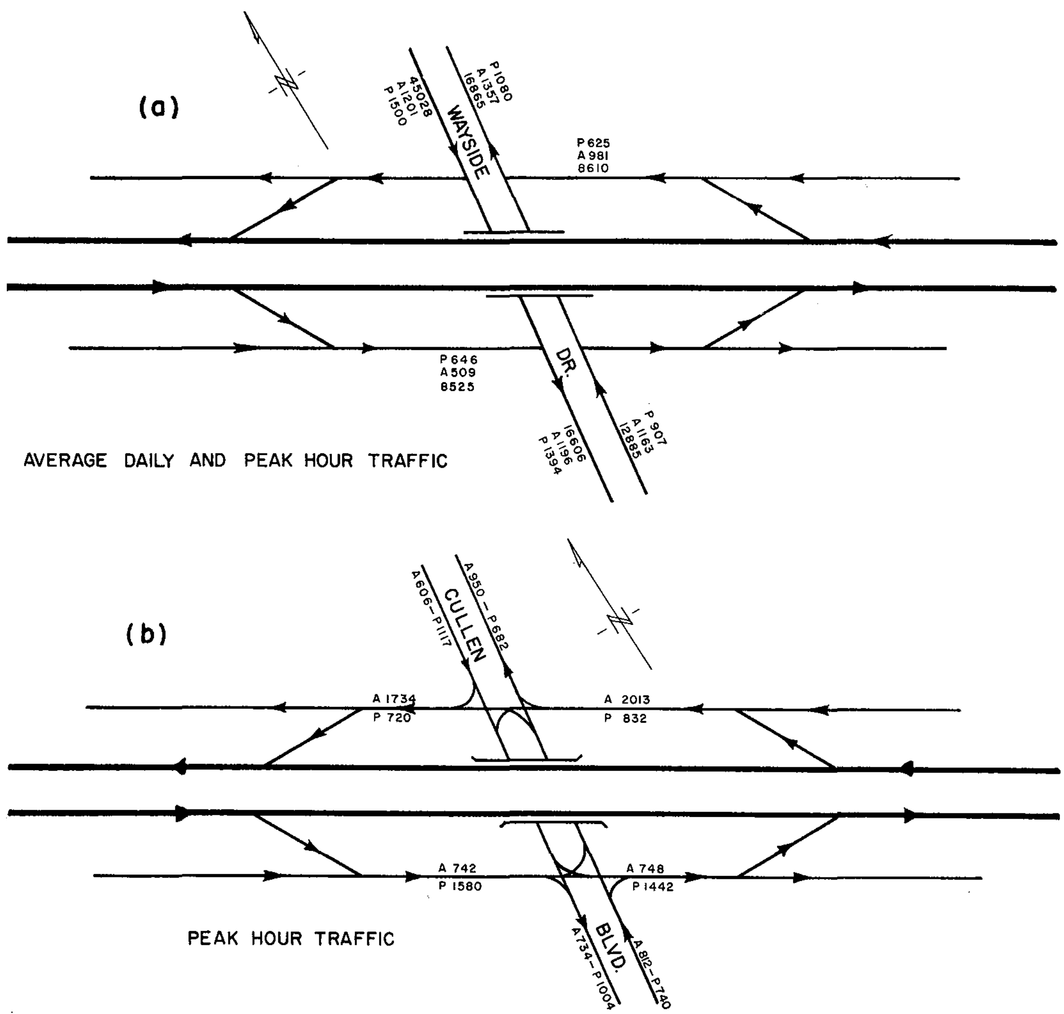

Figure 3. (a) I.H.-45 at Wayside Drive, Houston; (b) I.H.-45 at Cullen Blvd., Houston.

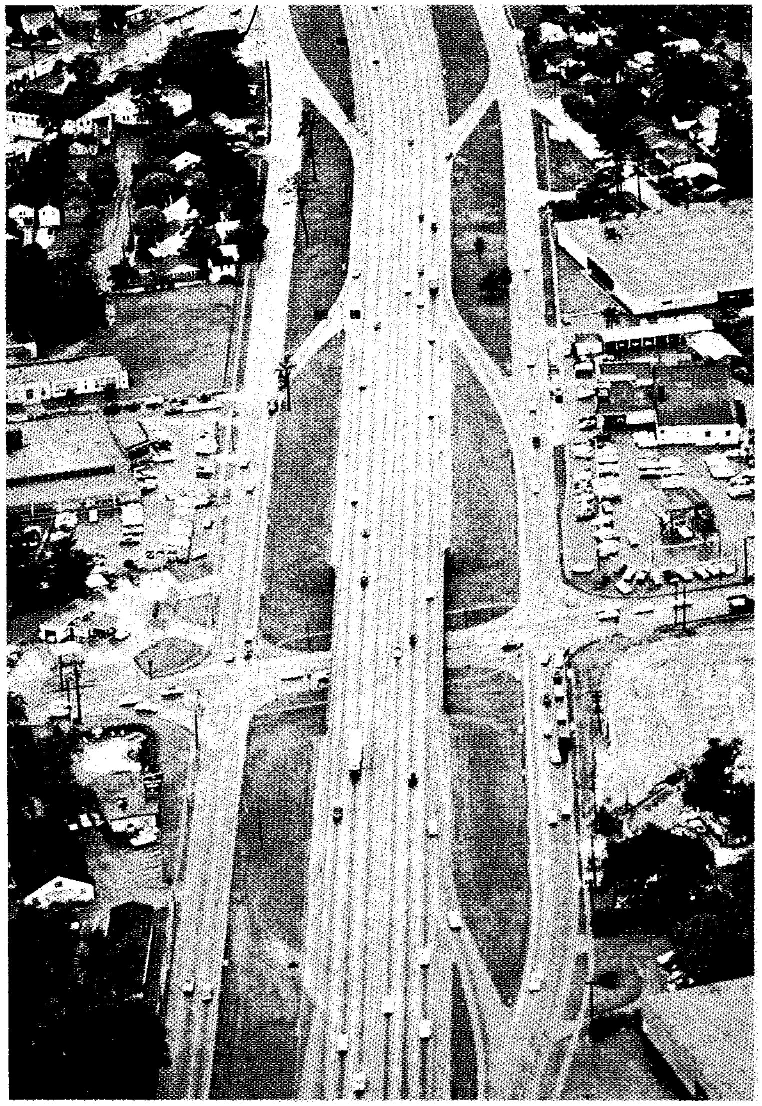

Figure 4. Wayside Drive interchange—Houston.

The traffic data were obtained on an individual lane basis and consisted of the following operational characteristics: (a) traffic volumes by composition and direction of movement; (b) starting delays after signal change to green by composition and direction of movement; and (c) time-headways between successive vehicles entering the intersection area by composition and direction of movement.

These data were recorded for each signal phase and notation was made to indicate if the approach lane was "loaded." An approach lane was considered loaded when the traffic demand was so great that vehicles were continuously entering the intersection throughout the green interval. The data on loaded intervals were then extracted and combined to provide information on peak flow conditions.

VEHICLE OPERATIONAL CHARACTERICTICS

In determining the number of vehicles that could clear a signalized diamond interchange in a given time period, a criterion was established by which vehicle operational

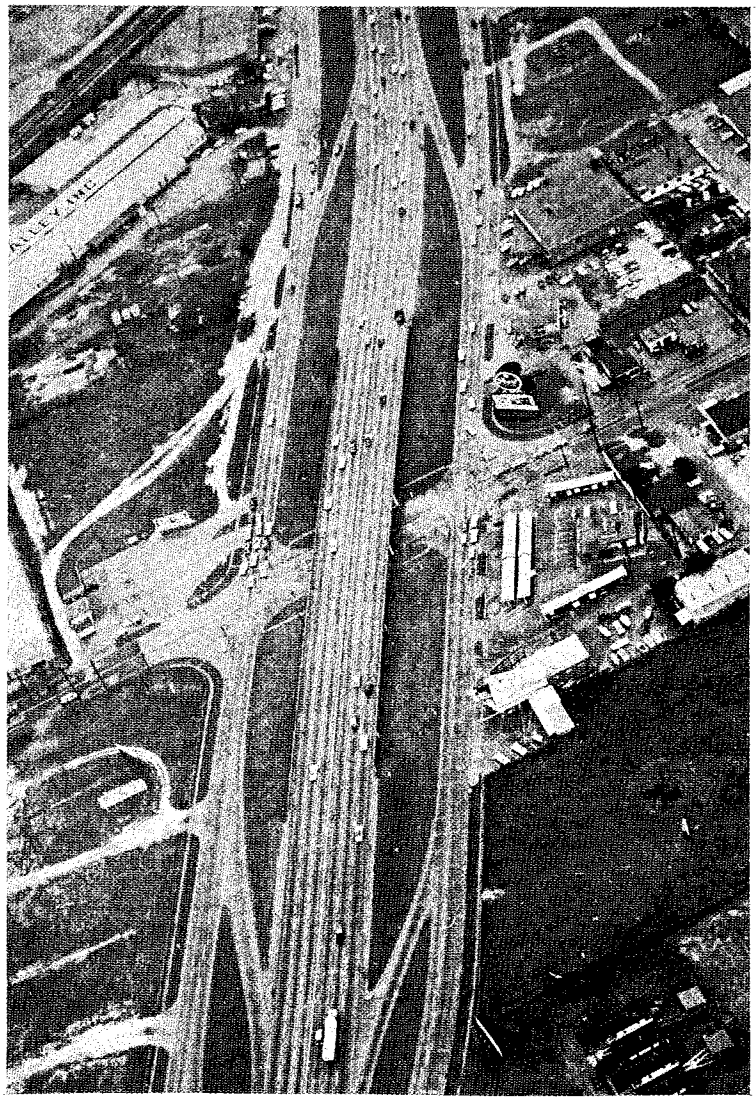

Figure 5. Cullen Blvd. interchange— Houston.

characteristics could be evaluated. This criterion involved a series of time measurements with special emphasis placed on the starting delays and time-headways of vehicles entering the interchange area. These measurements were defined as follows:

Time-Headway. —The time-headway between vehicles as they start from a stopped position one behind the other decreases progressively until they reach an average minimum (Fig. 10). Data from this study indicated that an average time-headway could best be obtained by averaging the time-headway values of the third through the last entering vehicle. It is true that this average value would be in error if an infinite number of vehicles were permitted to continue through an intersection without having to stop. However, the maximum number of vehicles that can every be expected to clear a signalized diamond interchange during one signal phase is approximately 10 to 12 vehicles per lane. This can be attributed to the fact that the signal cycle has to be proportioned to include other movements at the interchange.

Starting Delay. —When a traffic signal interrupts a flow of traffic, the vehicles stopped by a signal are delayed during the time the signal is red plus the time required for the vehicles to get started and underway again. This latter delay is commonly called starting delay. A generally accepted definition of starting delay is the time required for the first vehicle in a queue to commence motion and enter an intersection after the traffic signal displays a green indication. This time does represent a large portion

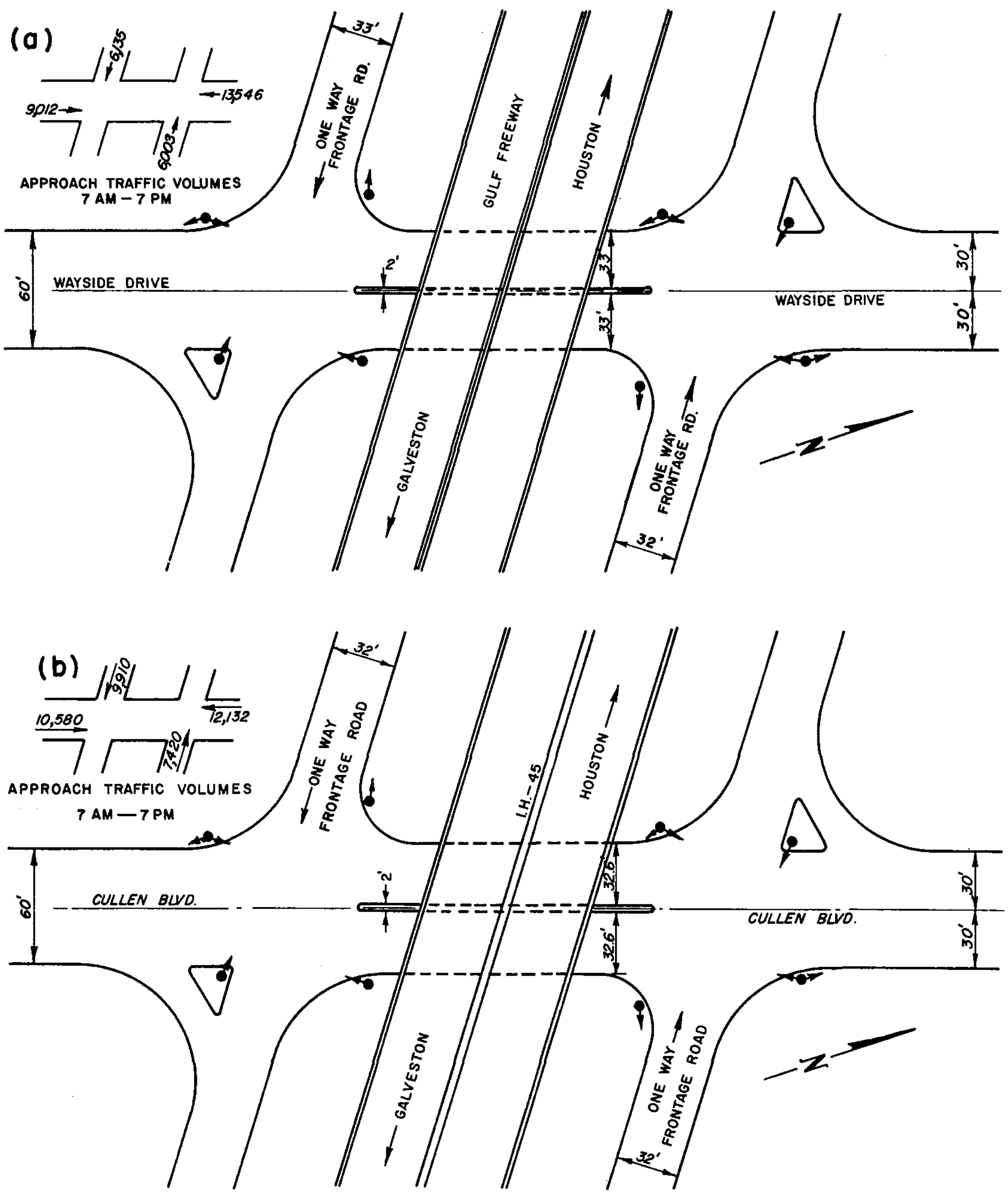

Figure 6. (a) Schematic layout—Gulf Freeway and Wayside Drive interchange Houston; (b) Schematic layout—Gulf Freeway and Cullen Blvd. interchange Houston.

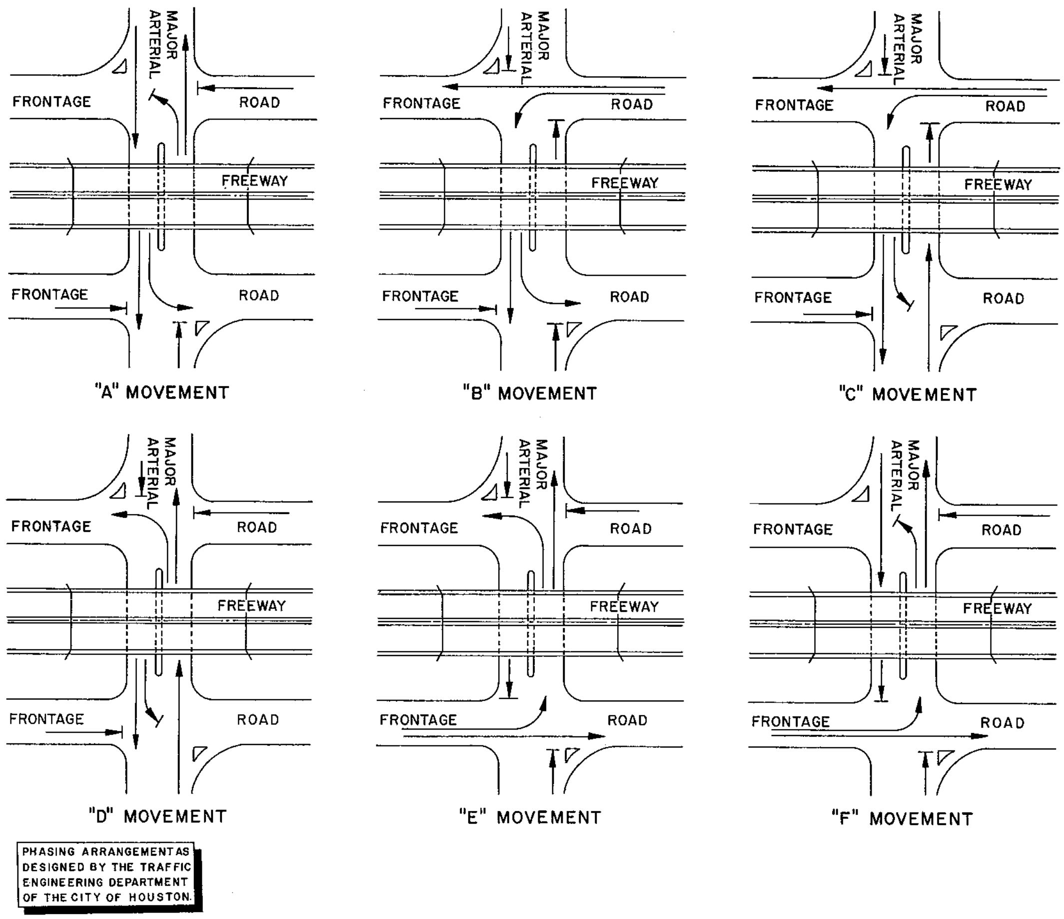

Figure 7. Phasing of traffic movements at a signalized diamond interchange.

of the starting delay experienced during each signal phase, but the operational studies showed that it does not represent the total time required for a line of vehicles to attain a reasonable degree of momentum. This is best illustrated by plotting the average time-headway values of a line of vehicles starting from a stopped position (Fig. 10). The time-headway values decrease rapidly for the first two vehicles in line with a lesser decrease for each succeeding vehicle. This indicates that the starting delay of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first two vehicles in line. In the evaluation of lane capacities for this report, the time required for the first two vehicles in a lane to enter the intersection was considered as the starting delay experienced at the beginning of each green phase.

Wayside Interchange

Starting delay and average time-headway measurements were determined for each lane approaching this interchange. The traffic on Wayside Drive consisted of two lanes of through movement with a free right-turn lane to accommodate the turning traffic. The frontage road traffic was considerably different with the predominant movements being both right and left turns onto Wayside Drive. Each of these movements was analyzed separately and are discussed as follows:

Wayside Drive Approaches. —The data gathered on each lane for the Wayside Drive approaches were combined because the traffic in each lane was similar and had no major differences in its operational characteristics. Figure 10 is a representation of the starting delay and time-headway measurements between successive passenger vehicles on these approaches. These observations yielded an average starting delay of 5.9 sec and an average time-headway of 2.2 sec.

North Frontage Road Approach. —Data were gathered on three different types of movements at this approach. These movements and their respective time-headways are shown in Figure 11a. The inside lane experienced a heavy left-turn movement, whereas the middle lane consisted of straight through movements. The outside lane had a combination of movements with 82 percent of the approach traffic turning right. The left turning movements from the inside lane and the right turning movements from the outside lane had identical operating characteristics with a 5.8-sec starting delay and a 2.1-sec average time-headway. The center lane, with a predominant straight through movement, was somewhat faster than the adjacent turning lanes with an average

Figure 8. Hydraulic platform truck.

Figure 9. Richardson film projector.

time-headway value of 1.9 sec. There was no significant difference in the starting delay for each lane.

South Frontage Road Approach. —The most significant data obtained at this approach involved the double left, or two-abreast-type turning movement. The lane signing required the inside lane traffic to turn left and gave the center lane an option of turning left or proceeding straight. At this approach, 65 percent of the center lane traffic turned left in conjunction with 100 percent of the inside lane traffic. The performance

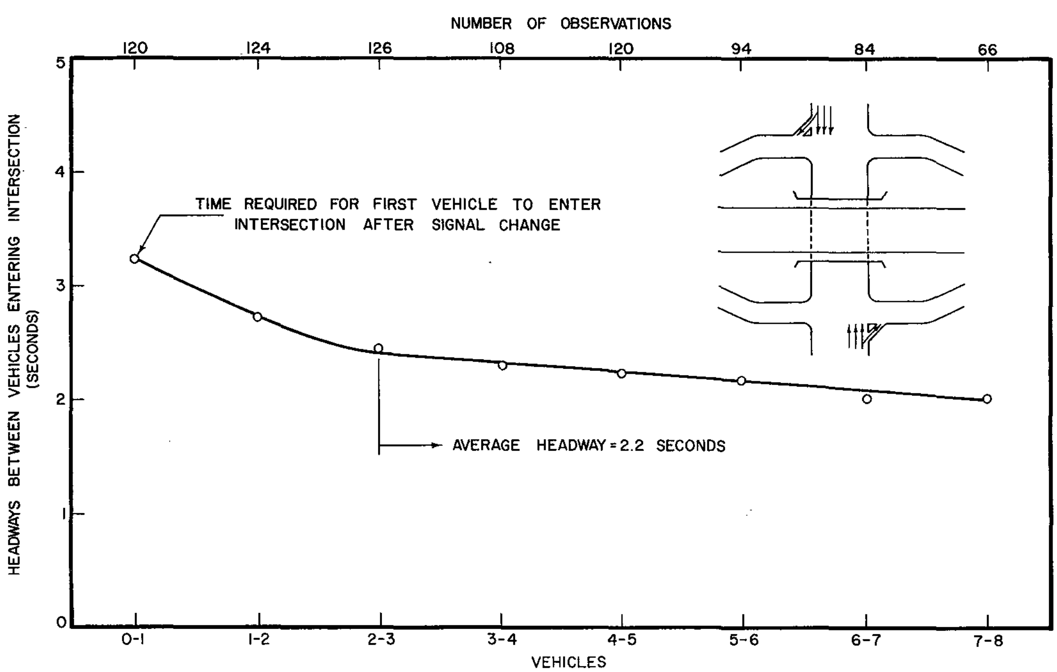

Figure 10. Time-headways between successive passenger vehicles on Wayside Drive at the Gulf Freeway and Wayside Drive interchange, Houston.

data of the traffic in these two lanes are shown in Figure 11b. From these data, it is obvious that the two-abreast turning movement had a detrimental effect on the capacity of the inside lane. The studies yielded a 6.5-sec starting delay and a 2.4-sec average time-headway for the inside lane traffic of the two-abreast turning movement as compared to a 5.8-sec starting delay and a 2.1-sec average time-headway for the traffic making a single left-turn movement on the North Frontage Road. Studies of the filmed traffic operations indicated that drivers in both lanes staggered the position of their vehicles in making the double left-turn movement. This hesitancy accounted for the inefficiency of this type of movement.

Cullen Interchange

As with the Wayside interchange, starting delay and average time-headway measurements were determined for each lane approaching the Cullen interchange. With the

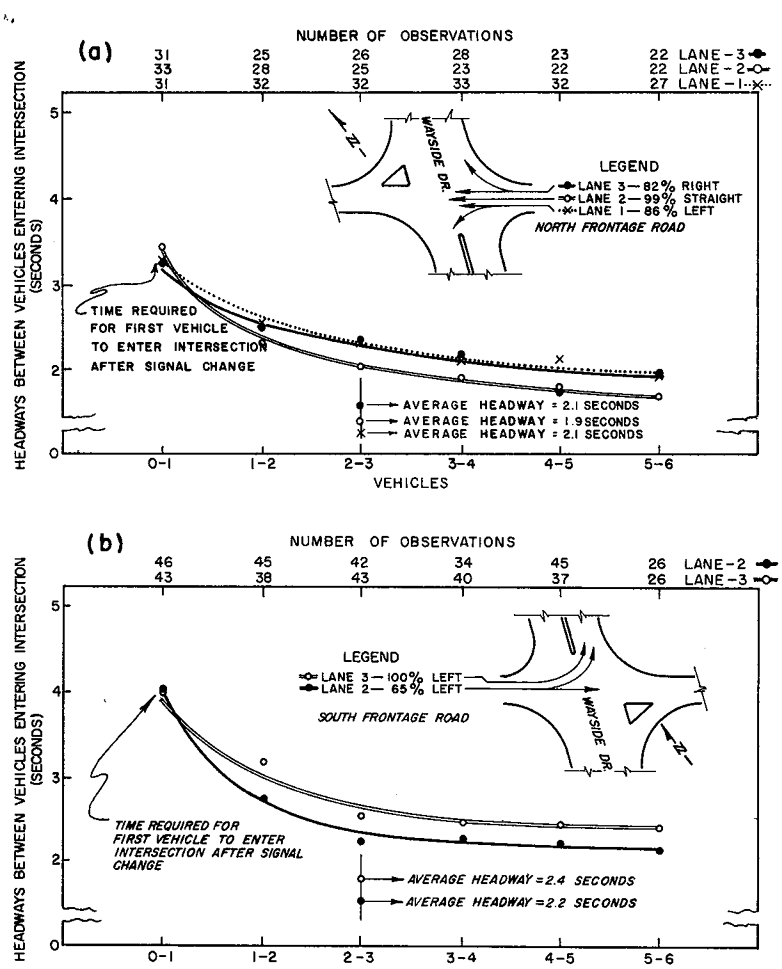

Figure 11. Time-headways between successive passenger vehicles on (a) north frontage road and (b) south frontage road at the Gulf Freeway and Wayside Drive interchange, Houston.

exception of the two-abreast-type turning movement, the movements at Cullen were similar to those observed at Wayside. The movements at Cullen were individually analyzed as follows:

Cullen Boulevard Approaches.—The traffic movement on these approaches consisted mainly of through movements with a free right-turn lane for the right turning traffic. The operational characteristics of the through traffic lanes were the only movements evaluated and their analysis showed that the through movement had a starting delay of 5.6 sec and an average time-headway of 2.1 sec. Figure 12 is a representation of the observed operations on these approaches.

North Frontage Road Approach.—This approach had three lanes of traffic. The center lane handled straight through movements, whereas the left- and right-hand lanes handled a combination of turning and through movements. The traffic in the inside lane and the center lane had similar operational characteristics with a 5.4-sec starting delay and a 2.0-sec average time-headway. The traffic in the outside lane was considerably slower with a 5.8-sec starting delay and an average time-headway of 2.1 sec. The operational characteristics of the vehicles in each lane are shown in Figure 13a.

South Frontage Road Approach.—The traffic movements on this approach were similar to those on the north frontage road. The traffic in the center lane was somewhat faster than the traffic in the adjacent lanes with a 5.4-sec starting delay and a 2.0-sec average time-headway. Traffic in the other two lanes had a large percentage of turning movements and experienced a 5.6-sec starting delay and a 2.0-sec average time-headway. These three movements with their respective time headways are shown in Figure 13b.

DATA SUMMARY

A total of seven studies was made at the study sites during which the operational characteristics of over 4,000 vehicles were recorded. Table 1A is a general summary

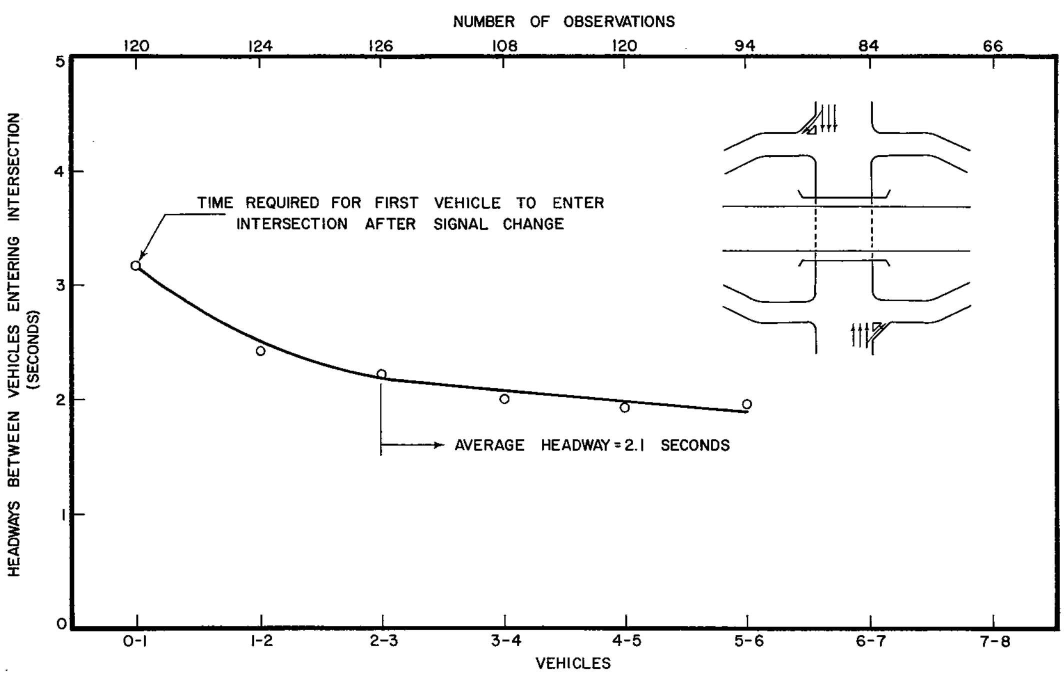

Figure 12. Time-headways between successive passenger vehicles on Cullen Blvd. at the Gulf Freeway and Cullen Blvd. interchange, Houston.

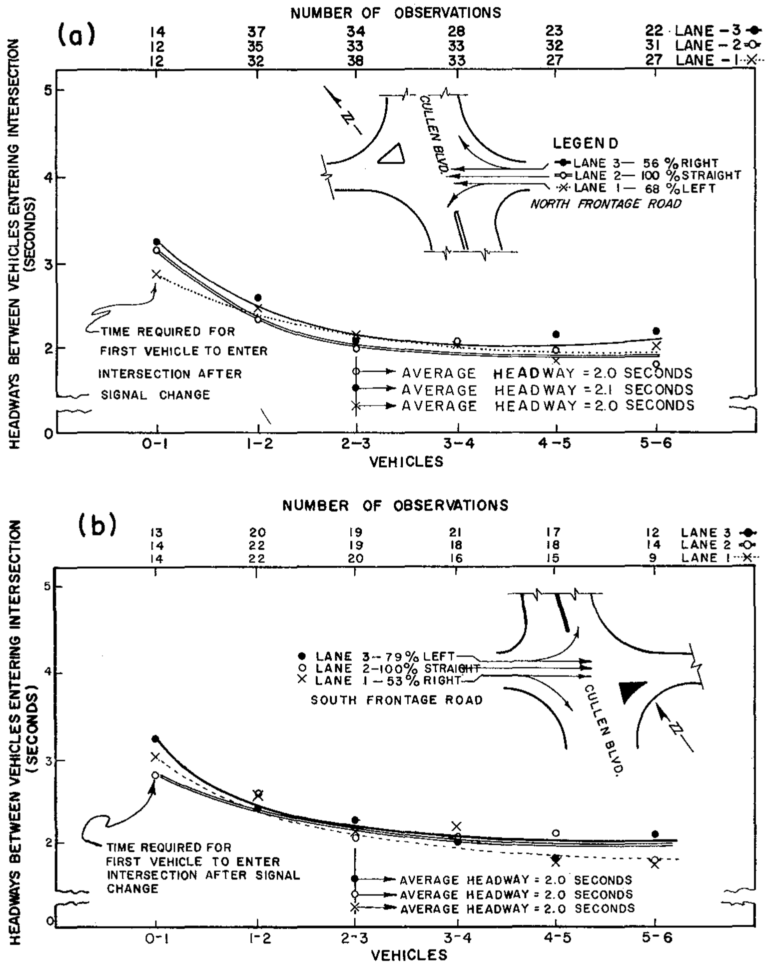

Figure 13. Time-headways between successive passenger vehicles on (a) north frontage road and (b) south frontage road at the gulf Freeway and Cullen Blvd. interchange, Houston.

of the data gathered at both the Wayside and Cullen interchanges. The operational data given in Table 1B indicated little difference in the operating characteristics of (a) straight through movements, (b) left turn movements, and (c) right turn movements. In view of this characteristic, the data on each type of movement at both the Wayside Drive and Cullen Boulevard interchange were averaged for comparison. This analogy showed that there was no significant difference in the starting delay and time-headway measurements of the straight, single left turning, and single right turning movements. However, there was a significant difference in the double left or two-abreast-type turning movement which necessitated separate consideration of these types of movements. A summary of the averaged data which are used in the capacity computations is given in Table 1B.

APPLICATION TO CAPACITY DETERMINATIONS

Capacity Formula

The problem of determining the capacity of signalized diamond intechanges is basically one of evaluating the capacity of two closely-spaced intersections. With proper

TABLE 1A

OPERATIONAL DATA

	Starting Delay (sec)	Average Time-Headway (sec)
Wayside Interchange		
Wayside Drive:		
Through movement	5.9	2.2
North frontage road:		
Lane 1 - 85% left	5.8	2.1
Lane 2 - 99% straight	5.7	1.9
Lane 3 - 82% right	5.8	2.1
South frontage road:		
Lane 1 - 57% right	6.8	2.4
Lane 2 - 65% left	6.5	2.2
Lane 3 - 100% left	6.5	2.4
Cullen Interchange		
Cullen Boulevard:		
Through movement	5.6	2.1
North frontage road:		
Lane 1 - 68% left	5.3	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 56% right	5.8	2.1
South frontage road:		
Lane 1 - 53% right	5.6	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 79% left	5.6	2.0

Interchange Studies

Study	Date	Time	Number of Vehicles Observed
Wayside Drive	June 9, 1960	A.M. Peak	
Wayside Drive	June 9, 1960	P.M. Peak	2,221
Wayside Drive	July 7, 1960	P.M. Peak	
Cullen Boulevard	April 20, 1960	A.M. Peak	
Cullen Boulevard	April 20, 1960	P.M. Peak	2,699
Cullen Boulevard	May 4, 1960	A.M. Peak	
Cullen Boulevard	May 4, 1960	P.M. Peak	

signalization, traffic may flow through both intersections on receiving the green on any approach and thus the capacity can be determined on a per-approach basis.

The signalized intersections of a diamond interchange fit into the classification of a "high-type" intersection as defined by the "Highway Capacity Manual" because the following conditions exist: (a) minimum pedestrian interference; (b) separate lanes for each traffic movement; (c) time separation of conflicting flows; (d) high standard of geometric design; and (e) no curb parking. On the basis of this classification, it was felt that the capacity of a diamond interchange should be determined on a per-lane basis for each approach with individual attention being given to the various types of movements.

The statistics on vehicle starting delay and time-headways provided the basic data for determining the number of vehicles per lane that can clear the intersections from an approach during each green interval. Hourly capacity values for all approaches can then be computed from this information.

TABLE 1B

Type Movement	Starting Delay (sec)	Average Time-Headway (sec)
Through	5.8	2.1
Single left turn	5.8	2.1
Single right turn	5.8	2.1
Two-abreast-type turning:		
Inside lane	6.5	2.4
Outside lane	6.5	2.2

In terms of starting delay, D, and average time-headway, H, the number of vehicles per lane, N'_L, that can be expected to clear the interchange from an approach during one green phase, G, is

$$N'_L = \frac{G - D}{H} + 2 \tag{1}$$

To determine the number of vehicles, N_L, that can clear per hour per lane, Eq. 1 can be multiplied by the number of cycles per hour (3,600/C), where, C, is cycle length in seconds, to give

$$N_L = \left(\frac{G - D}{H} + 2\right) (3{,}600/C) \tag{2}$$

To simplify design and evaluation procedures, capacity charts were developed from the operational data given in Table 1B and Eqs. 1 and 2. Figure 14 shows a chart with lane capacity (vehicles per hour) plotted against length of green phase for various cycle lengths (40 to 80 sec). This chart furnishes lane capacity readings for left, straight or right movements and can be used to evaluate the per-lane capacity of an approach or to determine the amount of green time required to move a specific lane volume.

Figures 15 and 16 are charts of lane capacity versus green time for the inside and outside lane, respectively, of a two-abreast left turn movement. This chart would be used for approaches signed for a double left turn.

DIAMOND INTERCHANGE CAPACITY

After studying the problem of evaluating the capacity of diamond interchanges, it was determined that it would be necessary to consider the two signalized intersections as a single unit. This is due primarily to the requirements of signalization which should perform two basic functions. These functions are as follows: (a) all high-volume conflicting movements at both intersections must be separated, and (b) storing of vehicles between the two intersections must be kept to a minimum due to limited distance between the intersections.

In considering various phasing possibilities, it was recognized that the movement sequence is dependent on existing volume conditions. However, if peak flow conditions are encountered on all approaches the movement sequence, shown in Figure 7, offers the best phasing to meet the two previously listed requirements. In addition, this sequence permits some phase overlap for maximum use of green time. This sequence was thus chosen as a basic phasing on which to base capacity considerations.

With a basic signal phasing established, the next step was to evaluate the capacity of the interchange system. For each approach to a diamond interchange, there exists a critical lane volume that must be accommodated. If the critical lane volume can be accommodated, then the adjacent lane or lanes on the same approach can accommodate less or equal volumes during the same green period. Thus it is necessary to consider only one lane (critical lane) per approach when determining the design and signalization of the interchange.

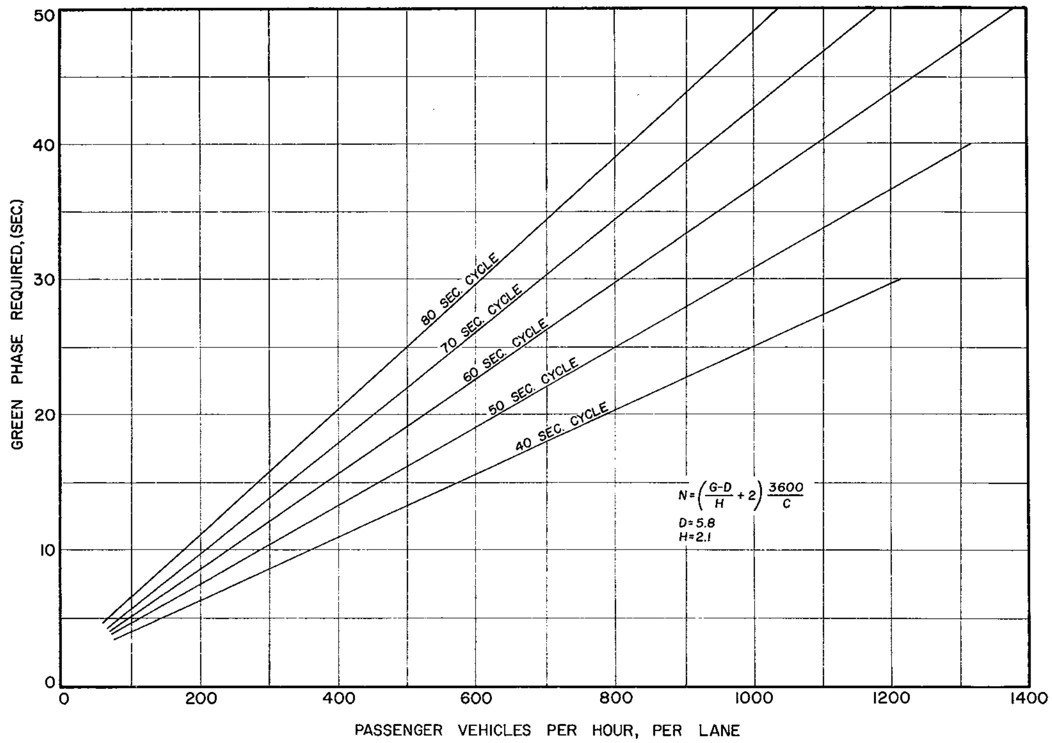

Figure 14. Design capacity of through movements and single lane turning movement.

Using this critical lane concept, an approach to the design and capacity of diamond interchanges was developed. A thorough study of conventional-type diamonds operating with a signal phasing as shown in Figure 7, indicated that the critical number of vehicles which must be accommodated is a summation of the critical lane capacities for four approaches. These critical approaches are shown in Figure 17. The interior approaches (those over or under the structure) are not critical since they receive a much greater percentage of green time than the critical approaches; for example, if the critical approach volumes can be accommodated, the interior volumes can be accommodated. This will be demonstrated in an example problem.

The critical interchange volume was determined by developing a formula for this purpose. This formula represented a summation of the single lane capacities for the four critical approaches. The formula was developed as follows:

The number of vehicles which can be accommodated from a single lane during each of the six movements (Fig. 7) is

$$\underline{\text{Movement E}} - n_1 = \frac{(G_1 - 5) - D}{H} + 2$$

$$\underline{\text{Movement F}} - n_2 = \frac{5}{H} + \frac{8 - D}{H} + 2$$

$$\underline{\text{Movement A}} - n_3 = \frac{G_2 - 8}{H}$$

$$\underline{\text{Movement B}} - n_4 = \frac{(G_3 - 5) - D}{H} + 2$$

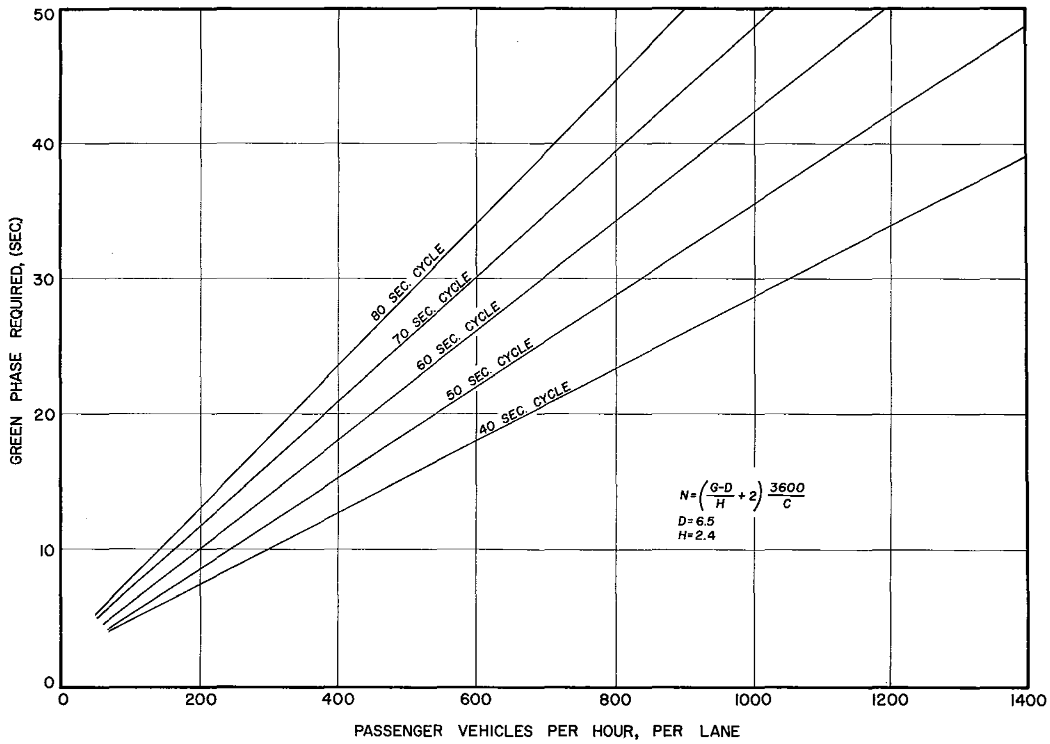

Figure 15. Design capacity of inside lane on two-abreast turning movement.

$$\underline{\text{Movement C}} - n_5 = \frac{5}{H} + \frac{8 - D}{H} + 2$$

$$\text{Movement D} - n_6 = \frac{G_4 - 8}{H}$$

(Note - An 8-sec overlap of the frontage road and major street traffic is permitted by the signalization and ambers of 3 sec were assumed.)

Total critical capacity per cycle, N_C' is equal to a summation of the vehicles accommodated during movements one through 6

$$N_C' = \frac{G - 4D}{H} + 8 \quad (G = G_1 + G_2 + G_3 + G_4) \tag{3}$$

The total cycle length, C, is equal to the following:

$$C = G + 6 - 10 = G - 4 \text{ or } G = C + 4$$

(Note - There are 6 sec of wasted amber time and 10 sec of phase overlap.)

If C + 4 is substituted for G in Eq. 3

$$N_C' = \frac{C + 4 - 4D}{H} + 8 \tag{4}$$

is obtained. If Eq. 4 is multiplied by 3,600/C or the number of cycles per hour, an

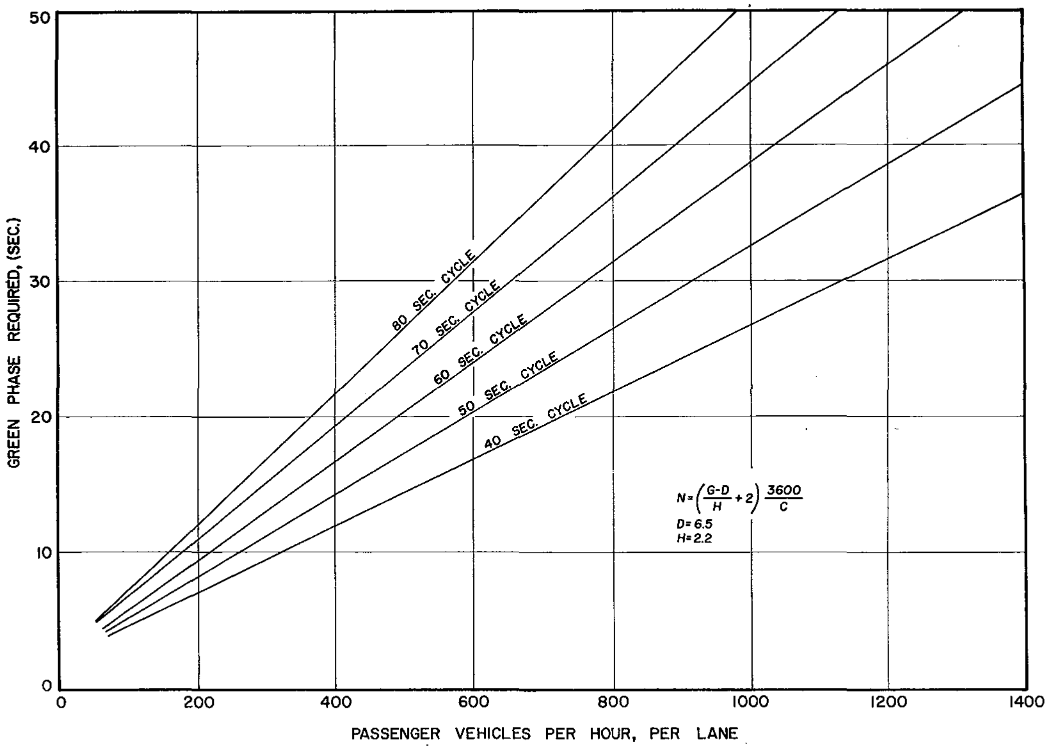

Figure 16. Design capacity of outside lane on two-abreast turning movement.

equation for the critical capacity per hour, N_H, is obtained, as follows

$$N_H = \left(\frac{C + 4 - 4D}{H} + 8\right) \frac{3,600}{C} \tag{5}$$

This critical capacity is a function of cycle length, C, starting delay, D, and time-headway, H. Eq. 5 gives the number of vehicles per hour that can be accommodated by the critical lanes on the four critical approaches. This number, N_H, has been termed "critical capacity" and represents the maximum summation of the four critical approach volumes.

If cycle lengths of 40, 50, 60, 70, 80, 100 and 180 sec and values of D = 5.8 and H = 2.1 are substituted into Eq. 5, the following values are obtained·

Cycle Length (sec)	Critical Capacity (veh/hr)
40	1,611
50	1,635
60	1,650
70	1,660
80	1,668
100	1,674
180	1,692

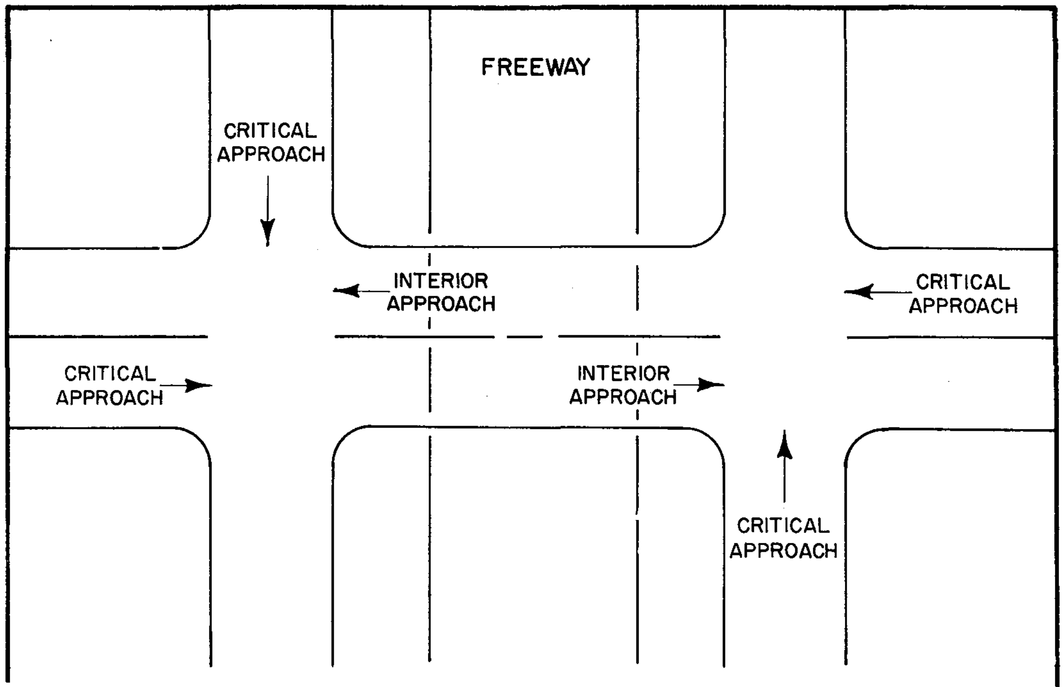

Figure 17. Critical approaches— diamond interchange.

Figure 18 shows a plot of the data in the text table. From these values it is indicated that any cycle length from 40-80 sec will yield basically the same capacity and that no significant gain in capacity is obtained by increasing the cycle length past 80 sec.

If, for example, a 60-sec cycle is assumed for design, the critical capacity is

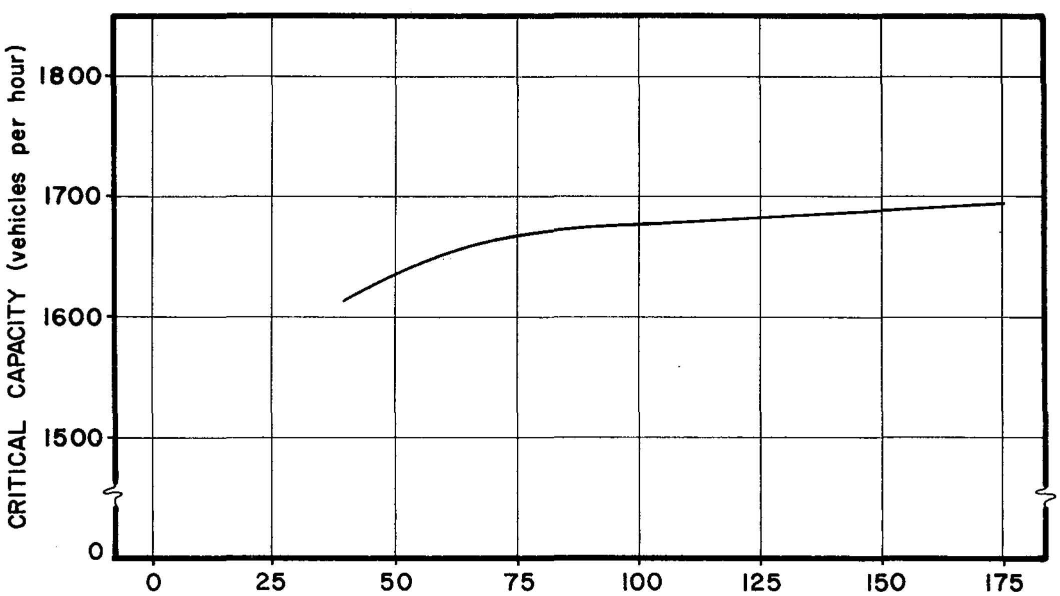

Figure 18. Cycle length (sec).

1,650 vehicles per hour. This indicates to the designer that a summation of critical lane volumes (for the four critical approaches) exceeding 1,650 veh/hr cannot be accommodated. Any combination of critical volumes with a summation less than 1,650 veh/hr can be accommodated with a 60-sec cycle or less.

Another point must be clarified at this time. The critical capacity expressed by Eq. 5 represents the total number of vehicles per hour that can be handled on the four approaches assuming that the total number is evenly distributed over the entire design hour (for example, the exact number of vehicles which can be handled per cycle is always available). Inasmuch as it is reasonable that this will not happen and that actually a great deal of fluctuation in the hourly volume will occur, special attention must be given to this factor.

Figure 19 shows a typical fluctuation of 5-min arrival volumes on an approach during the peak hour. If a design is based on the total hourly volume, it is evident that it will be inadequate to accommodate the short peaks within the hour. The question of just what volume the design should be based on warrants serious study and is presently being given detailed investigation by the Texas Transportation Institute.

On the basis of several studies, it was decided that peak hourly volumes should be increased by 20 percent to obtain a design volume which would be compatible with the indicated design procedure. This increase of 20 percent was obtained by expanding peak 30-min demands to an equivalent hourly flow and comparing this value to the total hourly demand (Fig. 19). In general, a 20 percent difference between expanded hourly demand and actual hourly demand was observed. Additional confidence in this figure was obtained from the "Highway Capacity Manual" which stipulates a 20 percent difference between Practical (or Design Capacity) and Possible Capacity. Therefore, it was determined that expected peak hourly volumes should be increased by 20 percent to obtain peak flow conditions for which Eq. 5 would be applicable.

ILLUSTRATIVE PROBLEM FOR DIAMOND INTERCHANGE

The application of the capacity formula to the design and signalization of a diamond

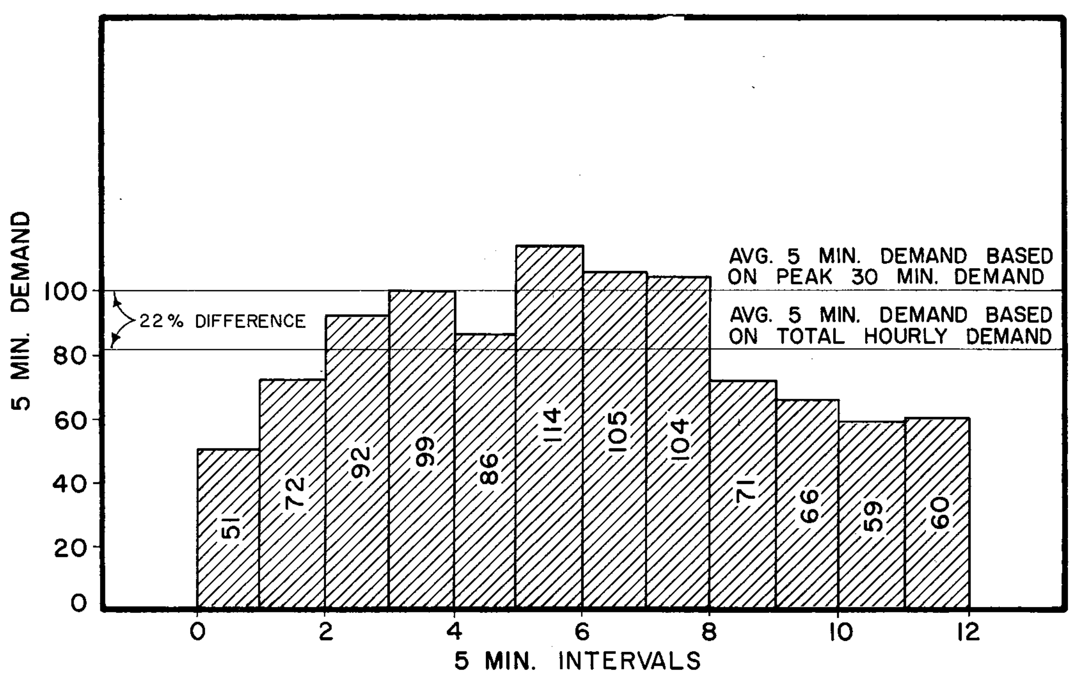

Figure 19. Hourly demand fluctuations.

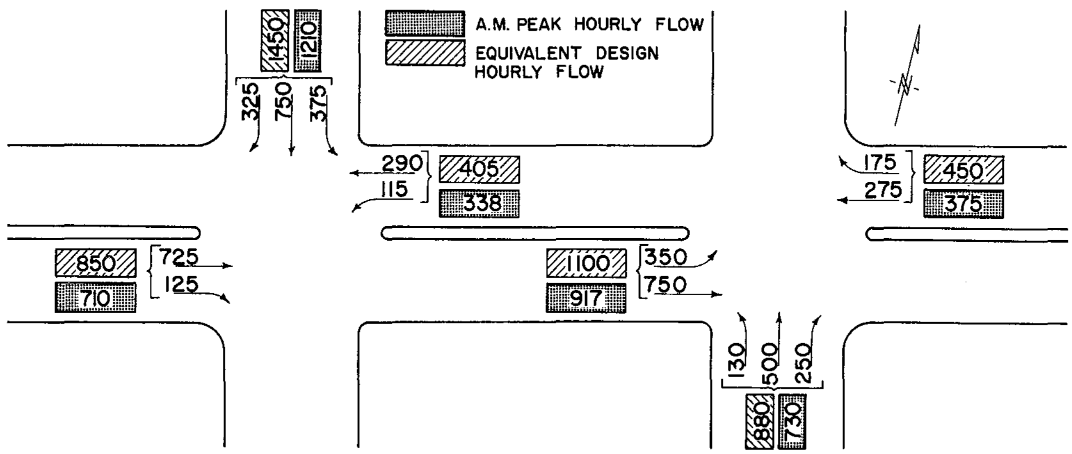

Figure 20. Design movements.

interchange can best be illustrated by an example. For this illustrative problem, traffic volumes were selected which might be representative of any large metropolitan area. These volumes are shown in Figure 20. The traffic selected is related to the intersection of a freeway and a major city arterial at which a diamond interchange is to be provided. As with all geometric design problems, both the a.m. and p.m. peaks should be considered in the design. However, for simplification, only one peak period is considered in this illustrative problem. The step-by-step design procedure is as follows:

Step I.—On the basis of the traffic movements shown in Figure 20, a reasonable geometric design is established and critical lane volumes as shown in Figure 21 are determined.

Step II.—The critical lane volumes are now examined to determine if they can be accommodated and if so, what cycle length is required.

$$\Sigma \text{ critical lane volumes} = 725 + 225 + 475 + 450$$

$$\Sigma \text{ critical lane volumes} = 1,875 \text{ veh/hr}$$

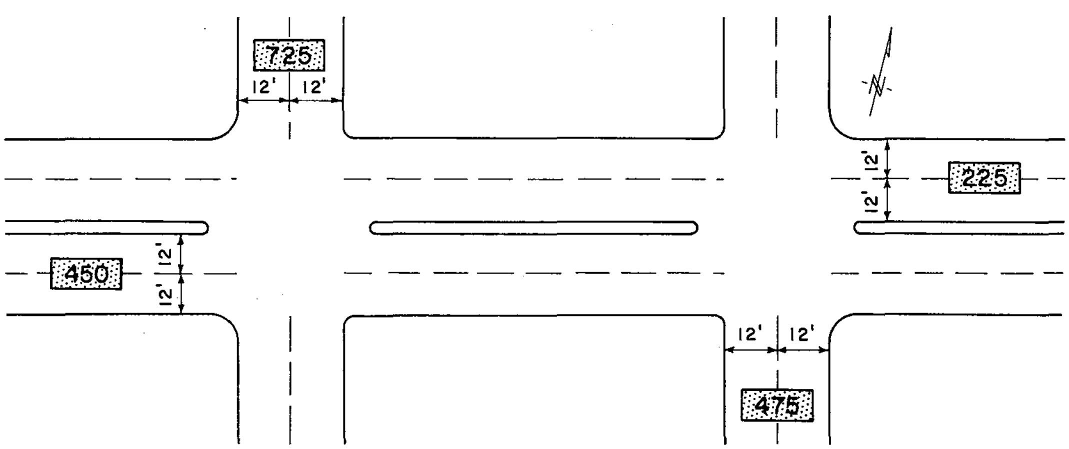

Figure 21. Critical lane volumes.

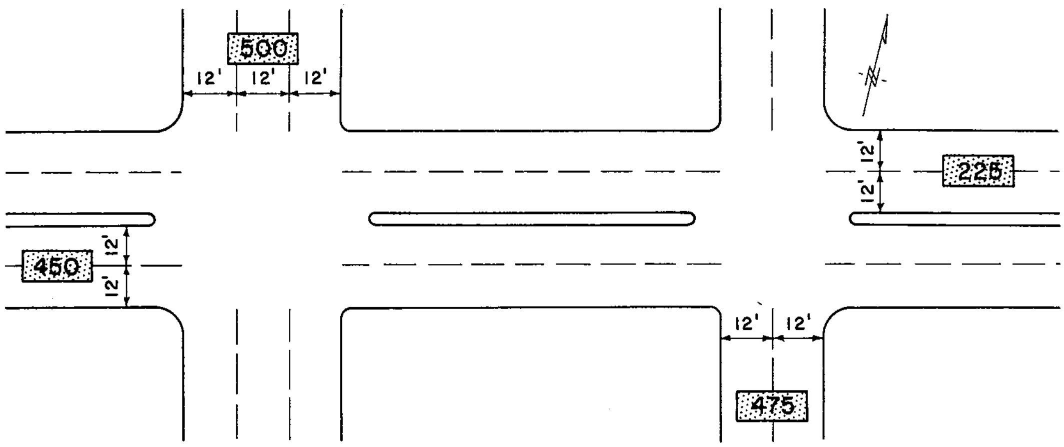

Figure 22. Critical lane volumes.

From Figure 18, it was found that this volume of vehicles cannot be accommodated with a reasonable cycle length. Therefore, the original assumed design is re-evaluated to determine what is needed to reduce the sum of the critical lane volumes. With reference to the assumed design in Figure 21, the most critical approach appears to be the west frontage road. If one additional lane is added to this approach, its critical lane volume can be reduced from 725 veh/hr to 500 veh/hr. This now gives a new design problem with critical lane volumes as shown in Figure 22.

Step III. —Considering the revised design and the new critical lane volumes, another check is made to determine if the revised volumes can be accommodated.

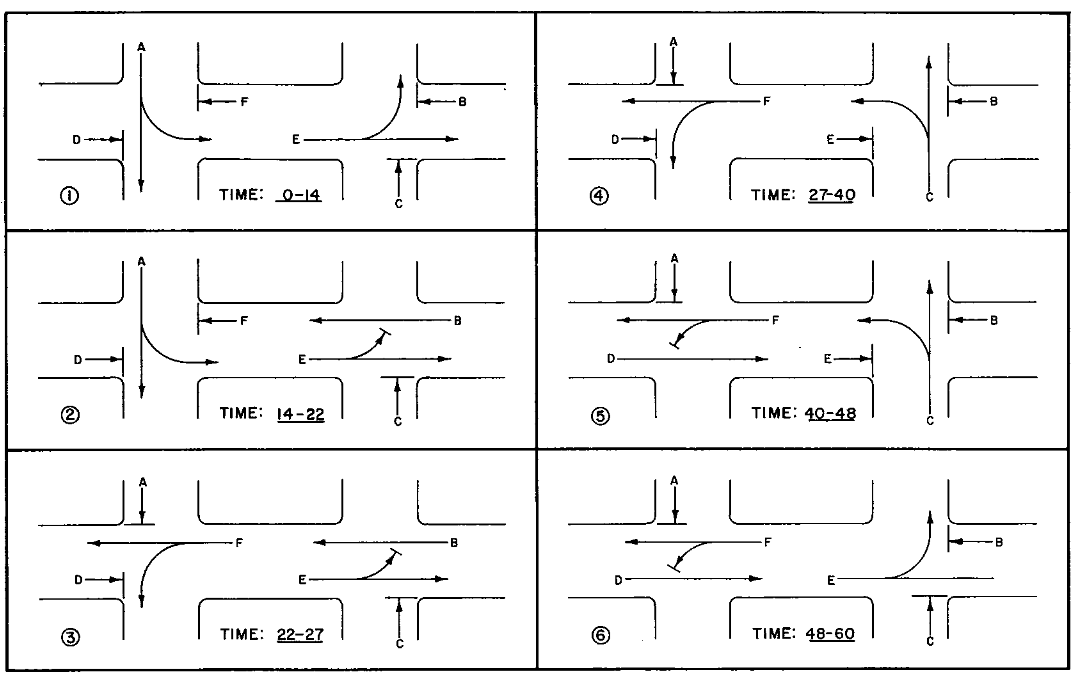

Figure 23. Phasing of traffic movements.

$$\Sigma \text{critical lane volumes} = 500 + 450 + 475 + 225$$

$$\Sigma \text{critical lane volumes} = 1,650 \text{ veh/hr}$$

Figure 18 indicates that this volume can be accommodated with a 60-sec signal cycle.

Step IV. —With the newly established lane volumes, Figure 14 is used to determine the amount of green and amber time required for each approach. The time required for each of the approaches is

Approach	Green (sec)	Amber (sec)	Total (sec)
A	19	3	22
B	10	3	13
C	18	3	21
D	17	3	20

Once the signal cycle and the length of each green plus amber phase is established, a phase arrangement for the movements is determined. A phase arrangement as shown in Figure 23 is used in this illustrative problem. With this phasing arrangement, each movement is timed so as to give each approach the required green and amber time as previously established. The timing of each movement is shown in Figure 23.

Step V. —Now that the capacity of the critical approaches has been determined, a capacity check of the interior approaches is made to insure that they are capable of accommodating their assigned volumes. The volumes to be accommodated on approach E is 350 veh/hr turning left and 750 veh/hr proceeding straight. The amount of green time allotted the straight through movement is 38 sec and will handle 980 veh/hr per lane which is greater than the 750 veh/hr demand. The time allotted the left turning movement is 23 sec and this time will accommodate 600 veh/hr per lane which is greater than the 350 veh/hr demand. Therefore, the traffic on approach E can be handled efficiently with the time which is allotted.

The traffic demand on approach F is 115 veh/hr turning left and 290 veh/hr proceeding straight. The green time allotted the straight through movement is 35 sec and will accommodate 900 veh/hr per lane. The left turning movement on this approach is allotted 16 sec which is capable of accommodating 410 veh/hr per lane. Consequently, this approach is also not critical in view of the amount of green time which is available.

It should be clarified that although the two lanes in each direction under the structure are adequate for this illustrative problem, their adequacy is dependent on the signal phasing used. If, for example, a signal phasing was used that required storage between the closely spaced intersections, then more than two lanes would probably be required.

This illustrative design problem has shown how the capacity-design procedure may be used in designing and signalizing a conventional-type diamond interchange. With modifications, this design procedure may be adapted to various intersection and interchange design problems.

DIAMOND INTERCHANGE SIGNALIZATION

The signalization of diamond interchanges is dependent on the volume conditions which may be encountered. However, of the various volume conditions which may exist, the most critical condition occurs when heavy traffic movements are experienced simultaneously on the four critical approaches (Fig. 17). This volume condition requires a signal sequence which will eliminate excessive storing of vehicles between the two closely spaced intersections. The signal sequence shown in Figure 7 provides the best phasing for this volume condition.

The recommended signal sequence shown in Figure 7 can be obtained by either fixed-time or vehicle-actuated equipment. However, because large fluctuations in traffic volumes are usually encountered on each approach of a diamond interchange, traffic-actuated equipment of the volume-density type could adjust to these fluctuations and provide more efficient operation.

SUMMARY AND CONCLUSIONS

The initial phase of this report represents the results of a study of vehicle operational characteristics at signalized intersections. This study was aimed at developing a method of determining capacity for intersection approaches on a single lane basis. The results of the study are as follows:

1. Starting delay for a queue of stopped vehicles at an intersection can best be determined by considering the time required for the first two vehicles in line to enter the intersection.
2. A time-headway value for vehicles entering an intersection can be accurately represented by an average of the time-headway values of the third through the last entering vehicle.
3. Capacity charts can be developed on the basis of starting delay and time-headways which will indicate lane capacities of an intersection approach for a given amount of green time.
4. The studies conducted on operational characteristics of vehicles at high-type signalized intersections indicated that there was no significant difference in the capacity of a straight through movement as compared to a single-lane turning movement.
5. Double left turns or two-abreast-type turning movements have a reduced capacity per lane as compared to a single left-turn movement. Capacity charts for two-abreast left turns are presented in the report.
6. A limited amount of commercial traffic was observed in the study. On the basis of the data obtained, it was found that heavy commercial vehicles had the equivalent effect (time-headway and starting delay) of approximately 1.6 passenger cars.

The second phase of the report was devoted to developing a method for determining the capacity and design of a conventional-type diamond interchange. This method and procedure is presented in the report.

It is realized that the method presented is related to a conventional-type diamond and to a basic fixed-time arrangement. It is felt that the basic method presented could be modified, adapted and used in the design of other forms of diamond interchanges or on individual high-type intersections. Any design problem requires extensive engineering judgment and the method presented is intended only as a design tool. It is further felt that the design method presented represents a good approach to the design of conventional diamonds regardless of the type of signalization which may eventually be used.

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