Some Traffic Parameters for the Evaluation of the Single-Point Diamond Interchange

MARK JAMES POPPE, A. ESSAM RADWAN, AND JUDSON S. MATTHIAS

The single-point diamond interchange (SPDI) has received a great deal of attention in recent years. Various analyses have been performed to evaluate the operation of the SPDI, but little field data has been collected to substantiate the input parameters used in these analyses. Research was performed to determine the saturation flow rate for the through and left-turn movements at the SPDI, and to determine the lost time per phase for the SPDI. Data were collected on 10 approaches at three interchanges in the Phoenix metropolitan area. More than 3,500 headways were measured. Mean saturation flow rates of approximately 2,000 passenger cars per hour green per lane were observed for both the through and the left-turn movements at the three SPDIs studied. Mean start-up lost times between 1.5 and 2 sec per phase were measured. Clearance lost time tended to be approximately 2.5 to 3 sec per phase less than the length of the clearance interval. Total lost time varied from 20 to 24 sec per cycle. The data indicate that the large turning radii found at the SPDI tend to cause the left-turn movement to operate much like a through movement in terms of capacity. The study also indicates that long clearance intervals translate directly into increased lost time per cycle.

Recent innovations in the design and operation of signalized diamond interchanges have created some uncertainty and controversy regarding the selection of the most appropriate interchange design. This uncertainty is mostly due to the advent of the diamond interchange configuration first introduced (by Greiner Engineering) in the early 1970s. This configuration has come to be known by a variety of names: the urban interchange, the single-signal interchange, the single-point urban interchange, and the single-point diamond interchange. The name used in this research is single-point diamond interchange (SPDI).

The SPDI (Figure 1) has received a great deal of attention in recent years (*1–3*) as a workable and even superior alternative to the conventional diamond interchange (CDI, Figure 1). The operational and geometric characteristics of the two interchange forms are essentially identical with respect to the freeway. The two forms differ considerably, however, with respect to operation on the cross street. The CDI is characterized by two closely spaced intersections, three-phase control at each intersection, and tight turning radii. The SPDI operates as one large three-phase controlled intersection with large turning radii. These major differences between the two configurations affect operation of the interchange on the cross street. Other differences with respect to geometrics, bridge design, right-of-way requirements, construction costs, and flexibility for future reconstruction further cloud the decision as to which design is the most appropriate for any given situation (*4*). The greatest confusion, however, lies in the area of the relative operational efficiency of the two competing configurations.

M. J. Poppe, Lee Engineering, 2701 E. Camelback Ave., Phoenix, Ariz. 85016. A. E. Radwan, Department of Civil and Environmental Engineering, University of Central Florida, Orlando, Fla. 32816-0450. J. S. Matthias, Department of Civil Engineering, Arizona State University, Tempe, Ariz. 85287-5306.

STUDY SCOPE AND OBJECTIVES

The Problem

The following statements provide some indication as to the range of opinions regarding the relative merits of the two configurations. Brown and Walters (*1*) write:

> By reducing the number of conflicts, and thus reducing the number of phases, the single-signal interchange can offer at least 10% more capacity than a diamond interchange. The advantage can be as much as 50% in cases where the left-turn volumes on the off-ramps are balanced and relatively high compared to the approach volumes on the minor street.

Leisch et al. (*4*) state:

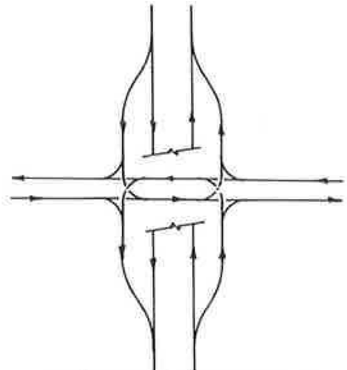
> The analyses presented make it evident that applications are limited for the single-point diamond. Generally speaking, the compressed diamond is less costly, has similar right-of-way requirements, and is more efficient.

Warner (*5*) finds that

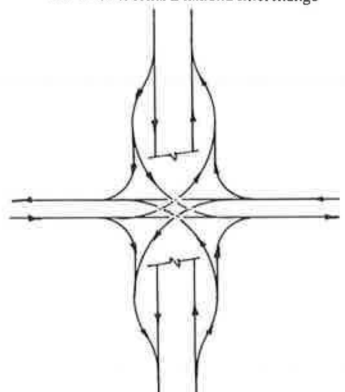
> the urban interchange is more efficient in distributing traffic between the freeway and the arterial, and vice-versa, whereas the diamond interchange is more efficient for through traffic on the arterial under the conditions studied.

A great deal of the confusion and lack of consensus among traffic engineers concerning the operational efficiency of the SPDI is due to the fact that very little is known about the operational characteristics of the SPDI. Because the SPDI operates as a single intersection, the methodology found in Chapter 9 of the *Highway Capacity Manual* (*6*) has been applied to evaluate the operation of the SPDI.

The two characteristics of the SPDI that distinguish it from the high type intersection are the large turning radii for the



The Conventional Diamond Interchange



The Single-Point Diamond Interchange

FIGURE 1 Interchange types.

left-turn movements and the large distance between stop bars. These two characteristics should be acknowledged in the analysis.

The large turning radii provided for the left-turn movements constitute one of the positive aspects of the SPDI configuration. However, there are currently very little data that quantify this advantage. Do the large turning radii provided by the SPDI configuration substantially affect the saturation flow rate for the left-turn movements?

The large land area required for the SPDI (and the long clearance intervals required as a consequence of the large area) is identified as one of the negative aspects of the SPDI. The hypothesis is that the lost time is directly proportional to the length of the clearance interval. There are no data, however, that quantify the lost time associated with long clearance intervals like those found at the SPDI. How much lost time per phase is generated by the long clearance intervals required by the SPDI?

Research Objectives

The focus of this research is the collection of data on various key parameters for the analysis of the SPDI. Current techniques for the operational analysis of the SPDI are based on the operational characteristics of the typical high type signalized intersection. The traffic parameters used in these analyses are based on data collected at typical high type intersections.

The primary objectives of the research are as follows:

1. To determine the saturation flow rates for the through and left-turn movements at the SPDI, and
2. To determine the lost time per phase for the SPDI.

Completion of the first objective will provide information that may be used to evaluate the influence of the SPDI's large turning radii. Completion of the second objective will provide information that may be used to evaluate the influence of the SPDI's long clearance intervals.

BACKGROUND INFORMATION AND LITERATURE REVIEW

Interrupted Traffic Flow Theory

The number of vehicles that may pass through a signalized intersection during one green indication is controlled by two factors: (a) the effective length of the green indication, and (b) the rate at which vehicles pass through the intersection, or the flow rate. The operation of a traffic signal also entails the periodic stoppage of traffic, at which time a queue of vehicles is formed. Therefore, the dynamics of starting a standing queue and stopping a steady flow of vehicles must also be considered.

When a traffic signal changes to green, time will elapse before the driver reacts and accelerates the vehicle into the intersection. This elapsed time, from beginning of green to the time the first queued vehicle enters the intersection, is called the first headway (h_1), measured in seconds per vehicle. The second headway (h_2) is elapsed time between entry of the first vehicle into the intersection and the entry of the second vehicle. The second driver must also react to the green indication and accelerate into the intersection; however, part of this reaction and acceleration time will occur during the first headway. Therefore, the second headway will be less than the first. The third, fourth, and fifth vehicles proceed through the intersection in a similar fashion, each with a slightly shorter headway than the previous vehicle. At some vehicle position, the headway between vehicles stabilizes at some relatively constant headway (h).

The saturation flow rate is defined as the rate of flow per lane at which vehicles pass through the intersection under the condition of stable headways (*6*, p. 1–8). The saturation flow rate is computed as

$$s = 3600/h, \qquad (1)$$

where s is the saturation flow rate, in vehicles per hour green per lane (vphgpl), and h is the saturation headway (sec).

The start-up lost time associated with the first vehicle (l_{s1}) is computed as

$$l_{s1} = h_1 - h \qquad (2)$$

where l_{s1} is the first vehicle start-up lost time (sec), and h_1 is the first vehicle headway (sec).

If the nth + 1 vehicle is the first in the queue with a headway equal to h, then the total start-up lost time (l_s) is computed as

$$l_s = \Sigma_{i=1 \text{ to } n} (h_i - h) \qquad (3)$$

The start-up lost time (l_s) must be subtracted from the green time in order to account for the dynamics of starting a standing queue.

The dynamics of stopping a steady stream of vehicles is another source of lost time at signalized intersections. When the movement of vehicles on a particular approach is terminated, some time must be provided to allow all vehicles to safely clear the intersection. Immediately following a green indication, some combination of yellow and all-red indication is provided as a clearance interval. Therefore, the time actually allocated to the movement is the green time (G) and the clearance interval ($Y + R$). Experience attests to the fact that a portion of the clearance interval is used by motorists. This portion of the clearance interval (the time from beginning of yellow to the time the last vehicle enters the intersection) is, essentially, an extension of the green time. The clearance lost time (l_c) is the portion of the clearance interval that is not used by the motorists. It is measured as the time elapsed from when the last vehicle entered the intersection to the end of the clearance interval (i.e., the beginning of the green indication for the next conflicting movement).

The number of vehicles that may pass through a signalized intersection during one green indication is controlled by two factors: (a) the effective length of the green indication, and (b) the saturation flow rate. However, as discussed, the effective green time is not necessarily the length of the green indication. The effective green time available to any given movement is the length of the green indication (G) plus the length of the clearance interval ($Y + R$) minus the start-up and clearance lost times.

With respect to this research, two questions must be answered through a review of the literature:

1. What are the key factors that should be analyzed in establishing values for saturation flow rates and lost times per phase at signalized facilities?
2. What is the best method for the collection of field data on the parameters of saturation flow rate and lost time at signalized facilities?

Saturation Flow Rate

The saturation flow rate at a signalized intersection is related to a number of geometric, traffic, and signalization conditions. The *Highway Capacity Manual* established a default value of 1,800 passenger cars per hour green per lane (pcphgpl) as the ideal or maximum rate of flow for vehicles passing through a signalized intersection (*6*, pp. 2–27). This value is called the ideal saturation flow rate because it represents the rate of flow expected under ideal traffic, geometric, and operational conditions. This value of 1,800 pcphgpl is based on research by a number of individuals and the observation of many intersections throughout the United States. It is an average value, not necessarily an absolute maximum.

The *Highway Capacity Manual* (*6*) notes that, irrespective of improved vehicle design or driver response, headways have remained relatively constant over time. Research performed by Johnsen and Matthias (*7*), however, indicates possible regional variations in saturation flow rates.

A number of factors affect the saturation flow rate at a signalized intersection. Tables and equations are given in the *Highway Capacity Manual* (*6*) to determine the value for each factor given the prevailing conditions. The *Highway Capacity Manual* identifies eight factors that influence the saturation flow rates at signalized intersections:

- Lane width,
- Vehicle type,
- Grade,
- Parking conditions,
- Bus blockage,
- Area type,
- Right turn, and
- Left turn.

The left-turn factor (f_{lt}) is of particular importance to this research. All existing SPDIs in the Phoenix metropolitan area operate under protected left-turn phasing, with double exclusive left-turn lanes on all approaches. The *Highway Capacity Manual* (*6*) adjustment factor for double exclusive left-turn lanes under protected phasing is 0.92. This adjustment factor converts to a saturation flow rate of 1,656 vphgpl. The *Highway Capacity Manual* adjustment factor for single exclusive left-turn lanes under protected phasing is 0.95, or a saturation flow rate of 1,710 vphgpl.

This smaller f_{lt} for dual left-turn approaches is partially attributed to research by Capelle and Pinnell (*8*) on the operational characteristics of the CDI. Capelle and Pinnell concluded from the data that the dual left-turn configuration reduced the carrying capacity of the inside lane. They attributed this reduction to a tendency by motorists in both lanes to stagger their position in making the double left-turn movement.

Lost Time

The lost time at signalized intersections is related to a number of geometric, traffic, and signalization conditions. The *Highway Capacity Manual* (*6*) establishes a default value of 4 sec lost time per phase, 2 sec each for start-up and clearance interval lost times. The *Highway Capacity Manual* does not provide a procedure for calculating the lost time as a function of the factors that may influence lost time at signalized intersections.

Agent and Crabtree (*9*) identified a number of factors that influence lost time at signalized intersections:

- City size,
- Location in city,
- Cycle length and length of green time,
- Speed limit,
- Gradient,
- Vehicle type and turning maneuver,
- Turning radius,
- Length of yellow, and
- Lane type.

Agent and Crabtree (*9*) observed a strong relationship between start-up lost time and cycle length. Start-up lost time tended to be lower for longer cycle lengths. They also observed lower start-up lost times for right-turn maneuvers over left-turn maneuvers. This is attributed more to the shorter saturation flow headways for the left-turn movement than to any actual advantage for the right-turn movement. An investigation of the effect of turning radius on the start-up lost time for the right-turn maneuver indicates lost time is less for shorter turning radii. Again, this is more a function of the saturation flow headways than any difference in the start-up time headways.

The effect of shorter saturation flow headways on start-up lost time is illustrated using the data collected by Capelle and Pinnell (*8*) as shown in Table 1. Capelle and Pinnell identified the first two vehicles in the queue as those contributing to the total start-up lost time at a signalized intersection. The starting delay values shown in Table 1 are the sum of the headways for the first two vehicles. The starting delay defined by Capelle and Pinnell should not be confused with the start-up lost time previously defined. The starting delay is actually the time required for the first two vehicles to enter the intersection and should, more properly, be called the start-up time.

The start-up lost time is calculated as

$$l_s = h_1 + h_2 - 2h \quad (4)$$

where

l_s = total start-up lost time (sec),
h_2 = second vehicle headway (sec), and
h = saturation flow headway (sec).

The inside and outside lanes, for two abreast type turns, have identical start-up times. However, the smaller saturation headway value for the outside lane increases the amount of lost time calculated for the outside lane. This relationship between start-up lost time and saturation flow rate should not be taken lightly. The general (though not universal) relationship exhibited here is that the higher the saturation flow rate the larger the start-up lost time.

Data Collection

The main issue with regard to data collection for this research was the selection of an appropriate intersection screen line and vehicle reference point for measuring time headways. The *Highway Capacity Manual* describes a procedure for the direct measurement of prevailing saturation flow rates (*6, pp. 9–74*). The selection of the reference screen line is largely left to the observer: "Choose a reference point, usually the crosswalk or stop line." The prescribed vehicle reference is the rear axle.

The curb line (extension of the cross street face of curb) is the screen line recommended by Berry (*10*), particularly when measuring headways for calculating lost time. Berry also recommends using the front of the vehicles, rather than the rear axle, as the vehicle reference point.

Agent and Crabtree (*11*) used the rear wheels of the vehicle as the vehicle reference point and the stop bar as the roadway screen line. It was believed this method would provide the best and most consistent results because of cross road offsets and angled intersections.

METHOD OF STUDY

For this study, each movement was classified by number, as shown in Figure 2. This classification of movements is in basic conformance with the classification of movements used in PASSER III-88 (*12*) for the numbering of movements at the CDI. PASSER III is a macroscopic computer model used to evaluate and optimize signal timing at CDIs. Movements 5 and 12, not shown in Figure 2, are the ramp or frontage road through movements in PASSER III. This particular SPDI configuration does not provide for these movements. PASSER III also classifies movements 15, 16, 17, and 18, which are the interior movements at the CDI. These movements are not applicable to the SPDI.

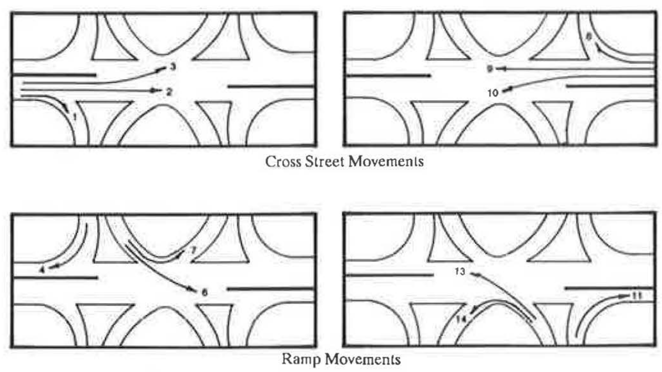


FIGURE 2 Classification of movements.

TABLE 1 CAPELLE AND PINNELL RESULTS FOR START-UP LOST TIME

Type of Movement	Starting Delay: h_1+h_2 (sec)	Average Headway: h (sec)	Start-up Lost Time (sec)
Through	5.8	2.1	1.6
Single left turn	5.8	2.1	1.6
Single right turn	5.8	2.1	1.6
Two-abreast turns:			
Inside lane	6.5	2.4	1.7
Outside lane	6.5	2.2	2.1

Note: Start-up lost time was not a reported parameter in the original source, but calculated by the authors of this research for illustrative purposes.

Site Selection

Five SPDI locations in the Phoenix metropolitan area could be used for data collection at this time:

- Hohokam Expressway (SR-143) and University Drive,
- Squaw Peak Parkway (SR-51) and Indian School Road,
- Squaw Peak Parkway and McDowell Road,
- Papago Inner Loop (I-10) and 7th Street, and
- Papago Inner Loop and 7th Avenue.

Based on the field observations made at the five sites the following three interchanges were selected for this study:

- Interchange A: Hohokam and University Drive (Figure 3),
- Interchange B: Squaw Peak and Indian School Road (Figure 4), and
- Interchange C: Papago Inner Loop and 7th Avenue (Figure 5).

Three criteria were used to select these three interchanges and the movements to be observed at each interchange. The first and most important criterion was length of queue. Long queues are important for collection of valid saturation flow rates and the evaluation of clearance lost time. The second criterion was variety of roadway geometry and signalization. The interchanges at 7th Street and 7th Avenue are nearly identical with respect to geometry and signalization; therefore, the location with the most favorable traffic conditions was selected. The interchanges at Indian School Road and at McDowell Road are also quite similar; therefore, only one of these two interchanges was selected. The third criterion used to select the most appropriate interchanges for this study was location of adjacent intersections and the influence they could have on traffic flow through the interchange. Four movements were selected at Interchange A for data collection: movements 2, 6, 10, and 13. Three movements were selected at Interchange B: movements 2, 10, and 13. Three movements were selected at Interchange C: movements 3, 6, and 9.

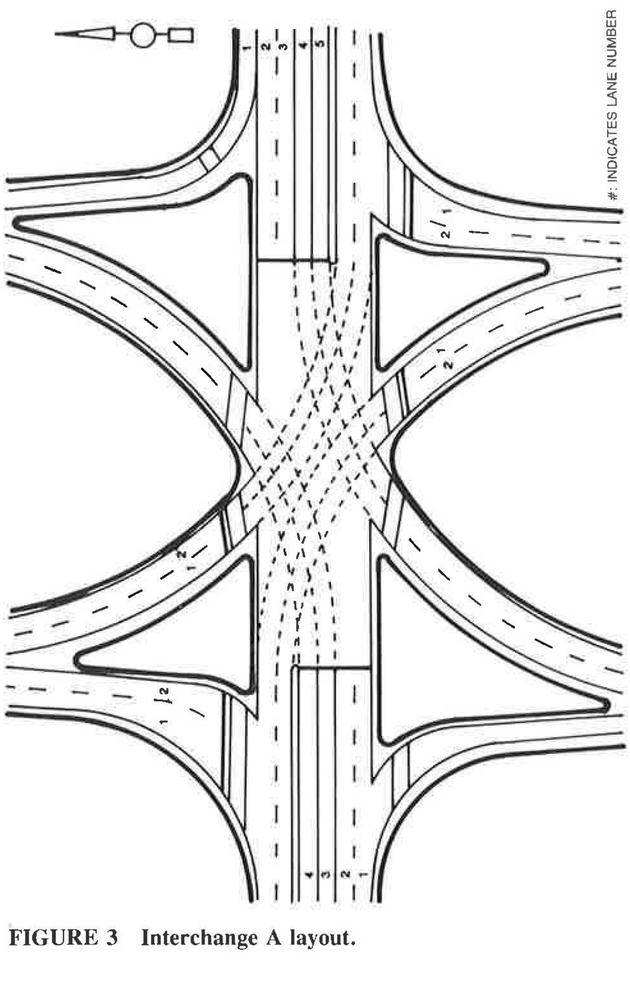


FIGURE 3 Interchange A layout.

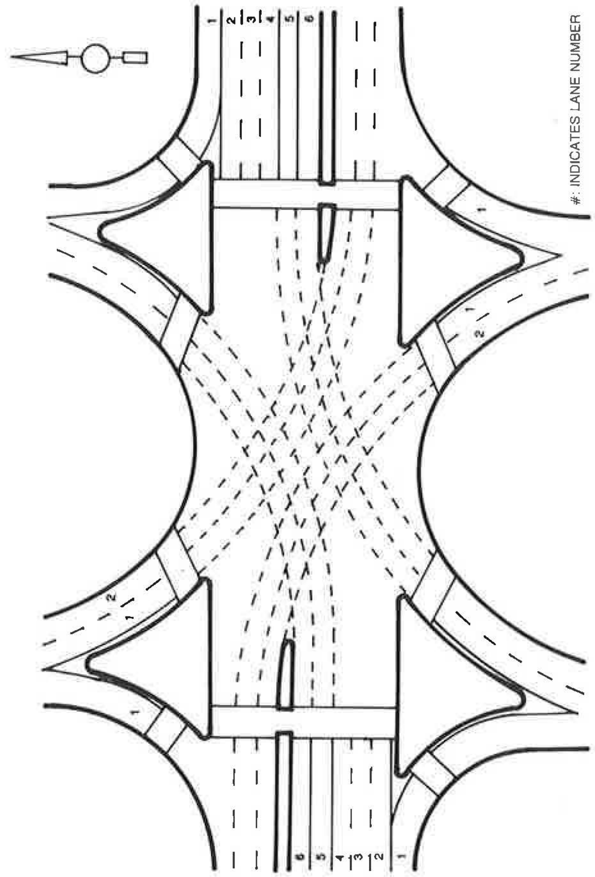


FIGURE 4 Interchange B layout.

Geometric, Signalization, and Traffic Conditions

Information concerning geometric and signalization conditions at the three sites was obtained from the Arizona Department of Transportation (ADOT) and the city of Phoenix. Geometric conditions at the three interchanges were determined by means of as-built drawings and field inspections. The geometric conditions recorded include the number of lanes per movement, lane widths, approach grades, lane configuration, length of turning lane storage bays, left-turn movement radii, parking conditions, stop line separation, and distance to adjacent signalized intersections. Stop line separation is defined as the distance between the stop line and the point

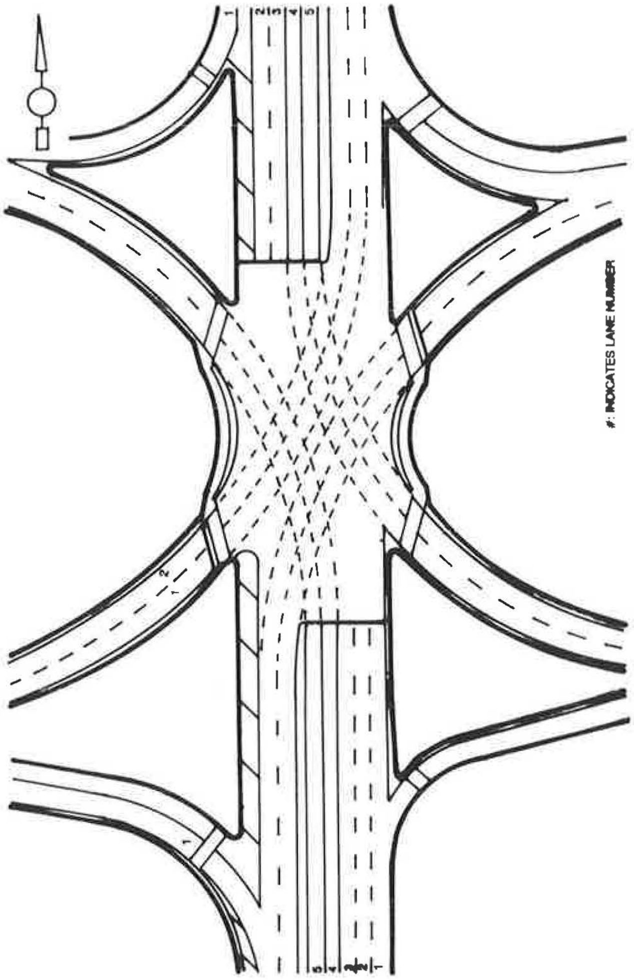


FIGURE 5 Interchange C layout.

beyond the path of conflicting movements. A summary of the geometric conditions for the three interchanges is presented in Table 2. Signalization information was obtained from ADOT documentation and through interviews with ADOT and city of Phoenix personnel. Signal timing was verified in the field using a stop watch. The signalization information recorded includes the cycle length, maximum and minimum green times, yellow and all red clearance interval times, type of operation (actuated or pretimed), pedestrian push-button actuation, minimum pedestrian green times, and the phasing plan. The phasing patterns used at each interchange are shown in Figure 6. Traffic conditions at the three interchanges were evaluated by means of field inspection. The traffic conditions recorded include a qualitative evaluation of the volume for each movement, the length of the queue for each movement with heavy traffic volumes, and the extent to which drivers tend to stop with the front wheels of the vehicle behind the stop line.

Data Collection

Data collection for this research involved the direct measurement of headways relative to changes in the signal head indication. It was important to measure headways relative to the changes in signal head indications in order to evaluate starting and clearance lost times. Data were collected on a per-lane basis for the ten lane groups studied. For this study, the rear wheels of the vehicle were considered the vehicle reference point and the stop line was the roadway screen line. Given the unusual geometry of the SPDI, it was believed that this method would provide the best and most consistent results.

The data collection method selected for this research involved the use of a portable computer and a Turbo Basic program. The computer's real time internal clock measured elapsed time. Keys were programmed to record, when pressed by an observer, the times for changes in signal aspect and the times as each vehicle crossed the stop line. Two observers were used in the data collection process, both working from the same keyboard. The first observer recorded the changes in signal aspect. The second observer recorded the passage of vehicles across the stop line. The second observer was also responsible for identifying the last vehicle in queue at the time the signal changed to green.

ANALYSIS

The first step in data reduction was to determine how average headways varied with position in queue. All data for a particular lane were pooled into a single spreadsheet. All vehicles not in queue at the onset of green were eliminated from the data base. All cycles with oversized vehicles were eliminated from this portion of the data reduction. A mean value for each position in queue was then calculated. A series of plots, like the one shown in Figure 7, were generated to determine the number of queued vehicles that must be counted before the beginning of saturation flow. The plots indicate that headways become fairly constant after the third vehicle. Therefore, the fourth through the last queued vehicle were those used to calculate the mean saturation flow rate. This is consistent with the procedure outlined in the *Highway Capacity Manual* (*6*) for the measurement of prevailing saturation flow rates.

Saturation Flow Rate

The data were analyzed on a per-lane basis in order to evaluate any difference between lanes on a multilane approach. The cross street through movement data do seem to indicate that saturation flow rates tend to be higher for the inside lanes and lower for the curb lane. The data are mixed with regard to changes in saturation flow rates with changes in lane position for the left-turn movements. Five of the seven left-turn lane groups examined recorded higher saturation flow rates in the inside lane. There does not appear to be a strong relationship between lane position and saturation flow rate for the left-turn movements at the SPDI.

A mean saturation headway value was calculated for each lane group studied. A sample standard deviation for the mean saturation headway was also calculated. Only those vehicles in queue at the onset of green were used in the analysis. All oversized vehicles were eliminated from the analysis. The results are shown in Table 3. A mean and a range for the saturation flow rate were calculated using Equation 1. The range is based on the values that represent a 95-percent con-

TABLE 2 INTERCHANGE GEOMETRY

Movement Number	Turning Radius (ft)	Stop Line Separation (ft)	Number of Lanes	Lane Number	Lane Width (ft)
Interchange A:					
2	---a	260	2	1	12
				2	12
6	270	190	2	1	12
				2	12
10	280	165	2	4	12
				5	12
13	270	190	2	1	12
				2	12
Interchange B:					
2	---a	310	3	2	11
				3	11
				4	11
10	280	235	2	5	10
				6	12
13	280	235	2	1	12
				2	12
Interchange C:					
3	310	200	2	5	11
				6	11
6	360	210	2	1	14
				2	14
9	---a	240	2	2	11
				3	11

a. Not applicable

fidence interval for the mean headway. The results of this calculation are shown in Table 4.

The mean saturation flow rate for the combined movements 6 and 13 at Interchange A is approximately 2,050 pcphgpl. The left-turn radii for movements 6 and 13 at Interchange A are 270 ft. The mean saturation flow rate recorded for movement 13 at Interchange B is also approximately 2,050 pcphgpl. The radius for this movement is 280 ft. Interchange C produced a substantially higher saturation flow rate for the ramp to cross street left-turn movement. A saturation flow rate of approximately 2,170 pcphgpl was recorded at Interchange C for movement 6. The left-turn radius for movement 6 at Interchange C is 360 ft. The data for this movement were collected under generally saturated conditions. The data for the ramp to cross street left-turn maneuver at Interchange C were collected under slightly different conditions than that found at the other two interchanges. Due to the closure of the west approach ramps at Interchange C there was no side friction with the opposing left-turn movement.

The saturation flow rate for the cross street to ramp left-turn movement 10 at Interchange A is approximately 2,025 pcphgpl. The turning radius for movement 10 at Interchange A is 280 ft. The saturation flow rate for movement 10 at Interchange B is substantially lower at 1,835 pcphgpl. But the combination of a small sample size and a large sample standard deviation raises a question as to the statistical significance of this mean value. This is reflected in the large range calculated for the mean saturation flow rate for movement 10 at Interchange B. The turning radius for this movement is also 280 ft. The saturation flow rate for the cross street left-turn movement at Interchange C is the highest flow rate recorded for any lane group in this study. The saturation flow rate for movement 3 at Interchange C is 2,225 pcphgpl. Virtually every cycle observed for this movement was operating under saturated conditions. The left-turn radius for movement 3 at Interchange C is 310 ft. However, there was, again, no side friction with opposing left turns for this movement.

Start-up Lost Time

A mean start-up lost time was calculated for each lane group studied. A sample standard deviation for the mean start-up lost time was also calculated. Start-up lost time was calculated using Equation 3 with n equal to 3 and h equal to the mean headway values shown in Table 3 for the lane group of interest. Only those cycles with three or more vehicles in queue at the onset of green were used in the analysis. All cycles with an oversized vehicle in position 1, 2, or 3 in the queue were eliminated from the analysis. The results of this calculation are shown in Table 5.

The mean start-up lost times as calculated on a per-lane group basis generally fall between 1.5 and 2 sec per phase.

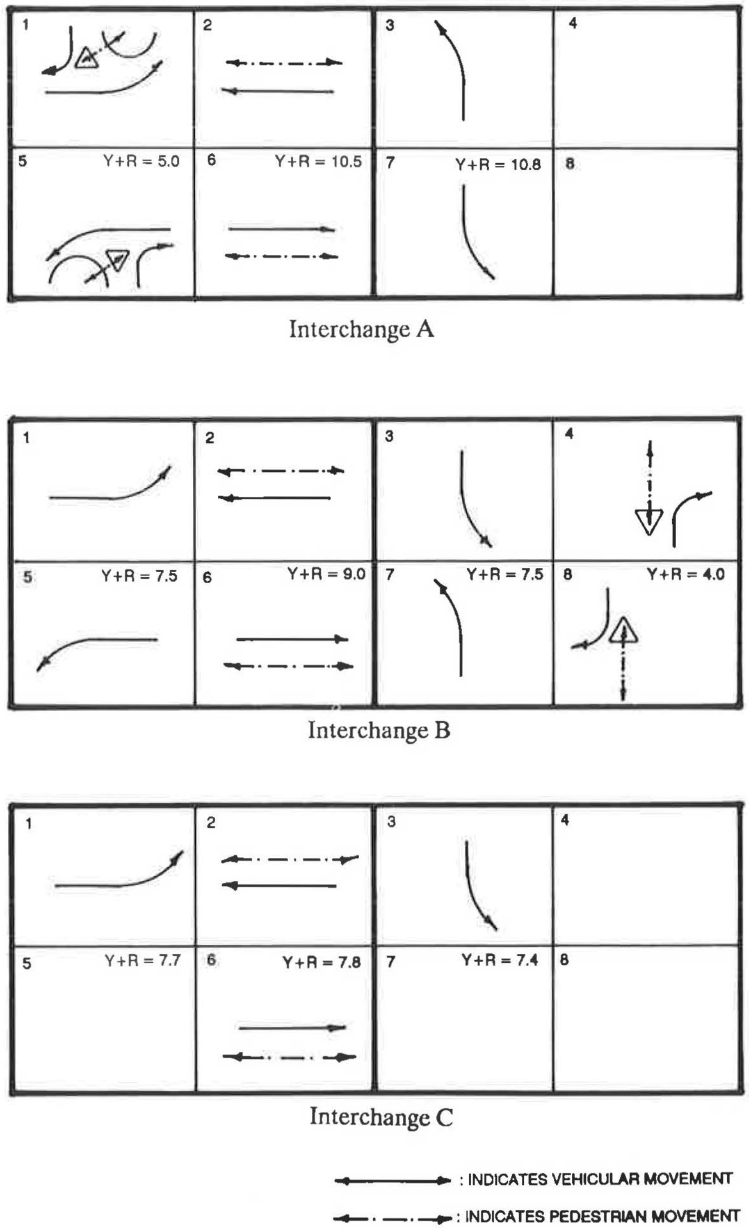


FIGURE 6 NEMA phasing.

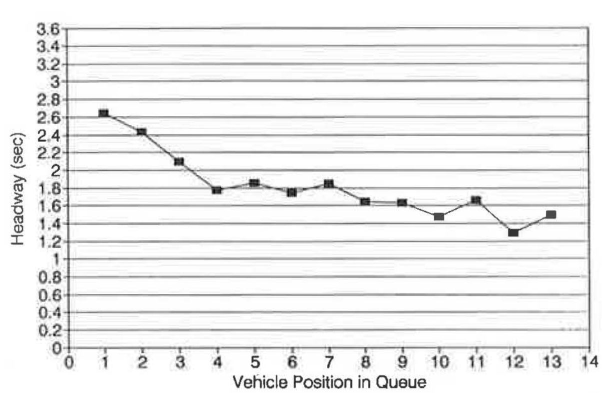


FIGURE 7 Mean headway versus vehicle position in queue.

TABLE 3 STUDY LANE GROUP WEIGHTED MEAN SATURATION HEADWAYS

Movement	Sample Size	Mean Headway (sec)	Sample Standard Deviation	95% Confidence Interval
Interchange A:				
2	229	1.81	0.68	1.72-1.90
6	216	1.79	0.76	1.69-1.89
10	169	1.78	0.76	1.66-1.89
13	298	1.73	0.45	1.68-1.78
6 & 13	514	1.75	0.60	1.70-1.81
Interchange B:				
2	505	1.76	0.50	1.72-1.80
10	76	1.96	0.81	1.78-2.14
13	74	1.76	0.44	1.66-1.86
Interchange C:				
3	354	1.62	0.48	1.57-1.67
6	255	1.66	0.44	1.60-1.71
9	226	1.94	0.55	1.87-2.01

TABLE 4 STUDY LANE GROUP WEIGHTED MEAN SATURATION FLOW RATE

Movement	Sample Size	Saturation Flow Rate (pcphgpl)	Range
Interchange A:			
2	229	1986	1894-2088
6	216	2009	1902-2129
10	169	2026	1904-2164
13	298	2085	2025-2148
6 & 13	514	2052	1994-2115
Interchange B:			
2	505	2044	1995-2096
10	76	1835	1679-2023
13	74	2047	1937-2171
Interchange C:			
3	354	2225	2159-2296
6	255	2172	2103-2246
9	226	1859	1793-1930

TABLE 5 STUDY LANE GROUP MEAN START-UP LOST TIME

Movement	Sample Size	Start-up Lost Time (sec)	Sample Standard Deviation	95% Confidence Interval
Interchange A:				
2	19	1.69	1.09	1.16-2.21
6	32	1.95	1.06	1.58-2.31
10	18	1.58	0.75	1.21-1.96
13	34	1.75	0.91	1.45-2.06
6 & 13	66	1.86	1.00	1.62-2.10
Interchange B:				
2	33	1.69	0.92	1.38-2.01
10	47	0.98	1.06	0.68-1.28
13	16	1.83	1.04	1.28-2.39
Interchange C:				
3	23	1.72	1.00	1.28-2.15
6	16	2.07	0.75	1.67-2.47
9	26	1.56	0.78	1.25-1.88

Two study lane groups were exceptions to this rule. Movement 10 at Interchange B recorded a mean start-up lost time of 0.98 sec. The mean saturation headway for this movement is 1.96 sec (Table 3), which is the largest saturation headway value recorded for the ten lane groups studied. Movement 6 at Interchange C recorded a mean start-up lost time of 2.07 sec. The mean saturation headway value recorded for this movement is 1.66 sec (Table 3), which is the second smallest mean saturation headway value recorded for the ten lane groups studied.

Clearance Lost Time

Clearance lost time was calculated as the difference between the clearance interval time ($Y + R$) and the green extension time. The green extension time was calculated as the elapsed time from the onset of the yellow indication to the time the last vehicle crossed the stop line. A mean green extension time was calculated for each lane group studied and then used to calculate the clearance lost time. A standard deviation for the clearance lost time was also calculated. Only those cycles that were saturated are included in the analysis. The results of the analysis are shown in Tables 6 and 7.

Large variations in clearance lost time may be observed for the various lane groups. This finding is largely a function of the length of the clearance interval ($Y + R$) provided for each movement. To evaluate the correlation between the clearance lost time and the length of the clearance interval, a plot was generated showing the amount of clearance lost time versus the length of the clearance interval. The plot, shown in Figure 8, indicates a near linear relationship between length of the clearance interval and the clearance lost time. Figure 8 indicates that a large portion of the clearance interval contributes to clearance lost time for long clearance intervals.

CONCLUSIONS AND RECOMMENDATIONS

The primary objectives of the research were to

1. Determine the saturation flow rates for the through and the left-turn movements at the SPDI, and
2. Determine the lost time per phase for the SPDI.

On the basis of the data collected it appears that 2,000 pcphgpl is a suitable base value for the saturation flow rate for both the through and the left-turn movements at the SPDI. The data do indicate that higher saturation flow rates may be in order for left-turn movements with radii greater than 300 ft. However, further study of the effects of side friction due to opposing movements should first be assessed.

Start-up lost time does not tend to vary a great deal by type of movement. Most movements recorded start-up lost times between 1.5 and 2 sec per phase. Start-up lost time is calculated as a function of the mean saturation headway. As saturation headways decrease, the start-up lost time tends to increase. Therefore, in those cases where higher saturation flow rates are used, higher start-up lost time values should also be applied.

The clearance lost time per phase is closely tied to the length of the clearance interval. If long clearance intervals are required then higher clearance lost time values should be applied. The clearance lost time is generally 2.5 to 3 sec less per phase than the clearance interval time for the phase.

The values for saturation flow rate and lost time reported in this study may be used to provide a more accurate evaluation of the SPDI operation. These values may be used as input parameters to determine the capacity of the SPDI and when using computer models to simulate the operation of the SPDI for the conditions studied.

These values are based on data that were collected at a limited number of approaches, all in the same geographi-

TABLE 6 STUDY LANE GROUP MEAN CLEARANCE LOST TIME

Movement	Sample Size	Clearance Lost Time (sec)	Sample Standard Deviation	95% Confidence Interval
Interchange A:				
2	5	8.19	0.54	7.52-8.86
6	4	7.94	1.57	5.45-10.44
10	9	2.61	0.70	2.07-3.15
13	5	7.69	1.02	6.42-8.96
6 & 13	9	7.80	1.21	6.87-8.73
Interchange B:				
2	12	6.31	0.55	5.97-6.66
10	0	---a	---a	--------a
13	0	---a	---a	--------a
Interchange C:				
3	21	5.33	0.89	4.93-5.73
6	16	4.37	0.63	3.73-5.00
9	1	4.74	---b	--------b

a :No data collected due to traffic conditions
b :Insufficient data collected due to traffic conditions

TABLE 7 CLEARANCE LOST TIME AS A PERCENT OF CLEARANCE INTERVAL TIME

Movement	Clearance Interval (sec)	Mean Clearance Lost Time (sec)	Difference (sec)	Percent
Interchange A:				
2	10.5	8.19	2.31	78%
6	10.8	7.94	2.86	74%
10	5.0	2.61	2.39	52%
13	10.8	7.69	3.11	71%
6 & 13	10.8	7.80	3.00	72%
Interchange B:				
2	9.0	6.31	2.69	70%
Interchange C:				
3	7.7	5.33	2.37	69%
6	7.4	4.37	3.03	59%
9	7.8	4.74	3.06	61%

cal location. They do not represent a comprehensive evaluation of all the factors affecting saturation flow rates or total lost time at the SPDI. A more comprehensive study is required to evaluate all the factors that may influence saturation flow rates and total lost time at the SPDI, for all possible conditions.

The values presented in this research are based on 3,500 recorded headways on a total of ten approaches. More data would provide much tighter confidence intervals for start-up and clearance lost times.

Interchange C was half operational at the time data were collected. Further study of this interchange should be performed when it becomes fully operational. Further studies of this interchange could be used in conjunction with the data collected in this study to measure the effects of opposing traffic on the left-turn movements at the SPDI.

New striping and signal design plans are being considered to reduce the clearance interval time at Interchange A. Further study of this interchange could add insight into the relationship between clearance lost time and the length of the clearance interval.

Finally, it should be noted that traffic engineering is not solely concerned with capacity analysis and signal operations. Many safety issues regarding the operation of the SPDI should

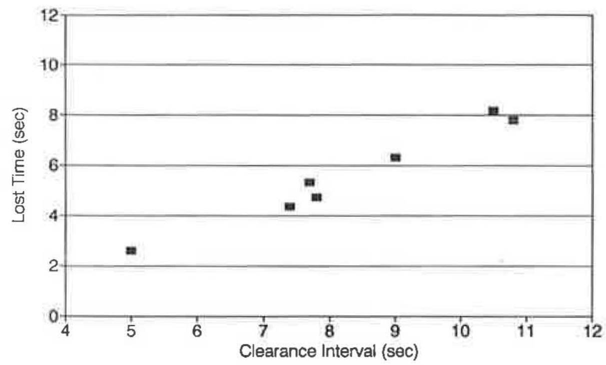


FIGURE 8 Clearance lost time versus clearance interval time.

be addressed. A number of vehicular and pedestrian traffic violations were observed during the course of data collection, most of which indicate road-user confusion. A comprehensive study of the potential safety problems associated with the operation of the SPDI should be conducted.

REFERENCES

1. S. J. Brown and G. Walters. The Single Signal Interchange. *Compendium of Technical Papers*, ITE, Washington D.C., 1988, pp. 180–184.
2. T. Darnell. Gridlock. *American City and Country*, Vol. 102, No. 8, Aug. 1987, pp. 20–28.
3. J. M. Witkowski. Benefit Analysis for Grade Separated Interchanges. *Journal of Transportation Engineering*, Vol. 114, No. 1, Jan. 1988, pp. 93–107.
4. J. P. Leisch, T. Urbanik, and J. P. Oxley. A Comparison of Two Interchange Forms in Urban Areas. *ITE Journal*, Vol. 59, No. 5, May 1989, pp. 21–26.
5. J. H. Warner. *Traffic Engineering Study of the Urban Interchange*. Arizona Department of Transportation, Traffic Engineering Section, Phoenix, Ariz., Aug. 1988, pp. 17–18.
6. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1988.
7. R. R. Johnsen and J. S. Matthias. Investigation to Determine the Capacity of Protected Left-turn Movements. In *Transportation Research Record 453*, TRB, National Research Council, Washington D.C., 1973, pp. 49–55.
8. D. G. Capelle and C. Pinnell. Capacity Study of Signalized Diamond Interchanges. *Highway Research Board Bulletin 291*, HRB, National Research Council, Washington D.C., 1961, pp. 1–25.
9. K. R. Agent and J. D. Crabtree. *Analysis of Lost Time at Signalized Intersections*. Report UKTRP-83-3. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1983.
10. D. S. Berry. Discussion of King and Wilkinson. In *Transportation Research Record 615*, TRB, National Research Council, Washington D.C., 1976, pp. 42–43.
11. K. R. Agent and J. D. Crabtree. *Analysis of Saturation Flow at Signalized Intersections*. Report UKTRP-82-2. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1982.
12. D. B. Fambro, N. A. Chaudhary, C. J. Messer, and R. U. Garza. *A Report on the User's Manual for the Microcomputer Version of PASSER III-88*. Texas Transportation Institute, Texas A&M University, College Station, 1988, p. 18.

Publication of this paper sponsored by Committee on Operational Effects of Geometrics.