

Comparison of Operational Parameters for Conventional and Single-Point Diamond Interchanges

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Five thousand headway measurements were taken at seven interchanges in the metropolitan Phoenix area. Saturation flow rates and lost times are given for each turning movement under different phasing conditions. A comparison is performed between saturation flow rates of conventional diamond interchanges with high design and with low design. There is a significant difference in saturation flow rates for left-turn movements from the arterial street, which indicates that high-design interchanges have higher saturation flow rates. Conventional diamond interchanges are compared with single-point diamond interchanges with respect to saturation flow rates and lost times. Single-point diamond interchanges have significantly higher saturation flow rates for the exclusive left turn from the ramp, but not for the left turn from the arterial or the arterial through movement. There is no significant difference in start-up lost times between the two interchange forms. Single-point diamond interchanges have significantly higher clearance lost times for the ramp left turn and the arterial through movement.

Although conventional diamond interchanges have been a common design, single-point diamond interchanges are being built with increasing frequency. The conventional diamond interchange, Figure 1, is characterized by two closely spaced intersections where the ramp movements terminate into the cross street. The ramps generally enter the intersection perpendicular to the cross street and the turning radii are typically less than 100 ft.

The single-point diamond interchange (Figure 1), also known as urban interchange and single-signal interchange, has only one intersection through which all ramp movements pass. The intersections created by this interchange type are usually quite large, and turning radii are typically greater than 200 ft. Because these interchanges also have much wider intersections, they are typically timed with much longer clearance intervals. Single-point diamond interchanges are typically more expensive because of the large bridge span that is needed to create the intersection.

Even though single-point diamond interchanges are being selected more often, few studies have been performed that compare the operational differences between the two interchange forms. A 1990 study by Poppe (1) measured saturation flow rate, starting lost time, and ending lost time at single-point diamond interchanges.

Poppe's research found that the saturation flow rates of the single-point diamond interchange are much higher than the

1,800 passenger cars per hour of green per lane (pcphgpl) presented in the 1985 *Highway Capacity Manual*. Although these measurements provide important factual data on the operation of single-point diamond interchanges, unless the same parameters are measured at conventional diamond interchanges, a comparison between the two cannot be made.

Therefore, this research collected similar information at conventional diamond interchanges. A comparison of the flow

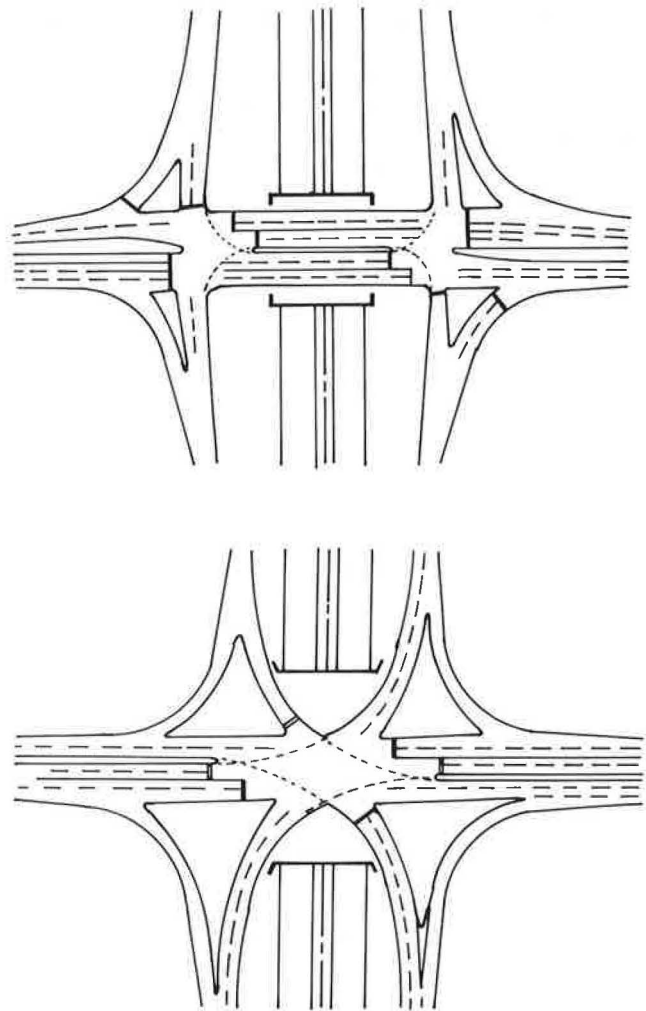


FIGURE 1 Conventional diamond interchange, top, and single-point diamond interchange, bottom (6,p.22).

characteristics of the two interchange types was conducted. The research results should aid transportation planning officials in determining if the extra construction costs for single-point diamond interchanges are outweighed by the added operational benefits that might occur.

BACKGROUND

Conventional Diamond Interchange Operation

A conventional diamond interchange is no more than two separate intersections that happen to be closely spaced, but the two intersections do not operate as independent entities. The operation at one intersection greatly affects the operation of the other. Three types of phasing are commonly used for conventional diamond interchanges in the metropolitan Phoenix area: three-phase, four-phase, and four-phase with overlap. All three phasing patterns were represented among the intersections at which data were collected in this study.

Saturation Flow Rate and Lost Time Studies

Perhaps the most widely known study on the operation of vehicles as they start progressing through an intersection is the work done by Greenshields (2) in the 1940s.

Greenshields found, that on the average, the first car entered the intersection 3.8 sec after receiving the green indication; successive vehicles entered with headways of 3.1, 2.7, 2.4, and 2.2 sec. After the fifth vehicle entered the intersection, all remaining vehicles entered with a 2.1-sec headway.

The starting lost time, then, was the difference between the sum of the number of seconds it took for the vehicles to enter the intersection minus the number of seconds it would have taken if they all entered with the 2.1-sec headway (2,p.27).

$$(3.8 + 3.1 + 2.7 + 2.4 + 2.2) - (5 * 2.1) = 3.7 \text{ sec starting lost time}$$

The operation of the vehicles through the intersection is depicted in Figure 2, which was popularized by Webster in 1958 (3,p.28). Time is shown along the abscissa, and rate of discharge of queue is shown along the ordinate. According to the Webster model, once a queue is released, departure rates increase until they reach a maximum departure rate called the saturation flow, or *s*. Vehicles enter the intersection at this rate until the signal indication turns yellow, at which point the departure rate rapidly returns to zero. The term *L*₁ refers to the starting lost time as previously discussed. The term *L*₂ refers to the ending lost time—the time lost at the end of the queue when the signal turns yellow—analogue to the starting lost time. If *g* is the green time for that phase and *a* is the yellow time, then

$$g + a = L_1 + L_2 + g_e \tag{1}$$

The term *g_e* is the “effective green time,” and the capacity (*c*) of an phase is calculated from the following equation:

$$c = s(g_e/C) \tag{2}$$

where *C* is the cycle length (4, p.16).

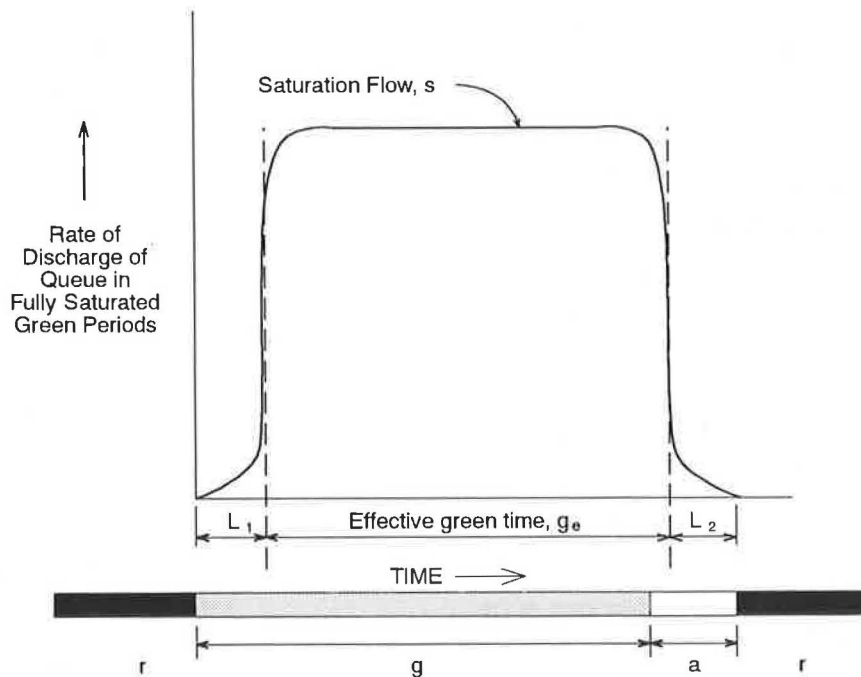


FIGURE 2 Variation of discharge rate of queue with time in a fully saturated green period.

The effective green time (g_e) can be written from Equation 1 above as (4,p.16)

$$g_e = g + a - L_1 - L_2 \quad (3)$$

Substituting Equation 3 into Equation 2 yields the following:

$$c = \frac{s(g + a - L_1 - L_2)}{C} \quad (4)$$

Inspection of Equation 4 shows that to calculate the capacity of an intersection, six parameters must be obtained. Three parameters— g , a , and C —can be referred to as “timing parameters,” and s , L_1 , and L_2 can be referred to as “operational parameters.”

Because it is an integral part of the operation of a traffic stream, several studies of saturation flow have occurred over the years. A summary of the results appears in work by Stokes (4,p.19). The range of observed saturation flow rates is 1,250 to 2,000 pcphgpl. Obviously, saturation flow rate is not a constant from time to time and place to place. The *Highway Capacity Manual* (5) recommends that capacity calculations consider the local saturation flow rate.

A study of saturation flow rates and lost times at single-point diamond interchanges in Phoenix was conducted by Poppe (1) in 1990. He found that saturation flow rates for left and through movements at these interchanges are very high, much higher than the 1,800 pcphgpl recommended by the *1985 Highway Capacity Manual*. There are three possible explanations for this: (a) the design of single-point diamond interchanges may be such that traffic flows through them more efficiently than other interchanges; (b) saturation flow rates in Phoenix, overall, are higher than national averages; (c) some combination of the previous two explanations. Indeed, the large turning radius at urban diamond interchanges makes for smoother left turns. However, there is no apparent reason that the through movement should be any greater than comparable through movements at similar locations.

STUDY METHODOLOGY

Interchange Selection

Seven interchanges were chosen where headway and lost time were measured. These interchanges included several types of signal phasing and geometric design and thus represent the operations at interchanges in the metropolitan Phoenix area. The task of selecting interchanges was challenging because several characteristics were desired and no one interchange could satisfy more than one or two requirements.

To determine the saturation flow rate for left turns from the arterial street it was necessary to find a location in which the left-turn movement was either protected-permissive or protected only. Although it is possible to collect flow rates at locations that are permissive-protected, vehicles begin to make the left turn as soon as there are gaps in the opposing traffic stream. As a result, when the protected phase begins, vehicles are already entering through the intersection and they experience reduced starting lost time. In the metropolitan Phoenix area, interchanges operate almost exclusively in a

permitted-protected manner. No intersections could be found that use a protected-permissive operation. However, some locations which have dual left turns at the interchange operate in a protected-only manner.

Another desire was to obtain saturation flow rates for right-turning traffic. To collect these data, a location that operates without right turn on red was required.

To collect flow rates for interior through movements, it was necessary to find a location that had heavy turning traffic and operated with three phases so that the ramp movement stored internally to the interchange.

Interchange A was chosen because it has dual left turns from the arterial street. It had a saturated ramp movement. The deck of the intersection provides two 11-ft lanes in each direction and one 10-ft left-turn lane northbound and two 10-ft left-turn lanes southbound. Because there are dual left turns from the interior approach, this movement is protected only.

Interchange B, constructed in 1964, was chosen because it typifies some of the older interchanges in the Phoenix area. The external through movements have 10- to 11-ft lanes. One operational deficiency of the intersection is that the arterial left-turning traffic must reach the deck during the arterial green phase in order to make the turn.

Interchanges C and F were chosen because they do not allow right turn on red for the ramp movement.

Interchange D was chosen because it has dual left turns from the arterial street. This interchange was reconstructed in 1989 and generally has a good geometric design. The lane widths are all 12 ft or larger. The dual left turns operate in a saturated condition in the afternoon peak hour, which allowed for several samples of ending lost time.

Interchange E, which operates in three phases and stores traffic on the deck, has a good geometric design with 12-ft lanes. The three-phase operation allowed for the collection of saturation flow rates and lost times for the internal through movements.

Interchange G was chosen because it uses four-phase-with-overlap operation. This timing is necessary to collect the saturation flow rates of left-turn movements that are stored externally to the diamond intersection.

Interchange Timing and Geometric Configuration

Interchange geometric configuration is shown in Table 1, and signal timing is shown in Table 2.

None of the study locations had a shared-lane condition. The term “dedicated” in this paper indicates those lanes that only allow one movement: either left, through, or right from the lane.

Data Collection

A TURBO BASIC computer program, COLSAT, was written to collect saturation flow rates and lost times at the intersections. This program was loaded onto a laptop computer that was taken into the field. TURBO BASIC has a command, *mtimer*, that operates much like a stopwatch. After the user

TABLE 1 Geometric Configuration of Study Lanes

Interchange	Direction	Lane ¹	Lane Width (feet)	Approach Grade (percent)	Turning Radius (feet)	Distance Stop Bar from Intersection (feet) ²
Interior Left Turn Movement						
A	SB	3	10	-1.6	82	50
A	SB	4	10	-1.6	72	50
D	NB	3	12	-0.2	66	35
D	NB	4	12	-0.2	54	35
Interior Through Movement						
E	NB	1	14	0.2	na ³	42
E	NB	2	12	0.2	na	42
Ramp Dedicated Left Turn Movement						
B	NB	3	12	1.0	55	15
B	SB	3	12	-0.5	55	21
A	WB	2	14	0.7	80	16
C	WB	3	13	0.2	90	30
F	WB	3	12	0.2	85	24
Ramp Dedicated Right Turn Movement						
C	WB	1	14	0.2	80	30
F	WB	1	12	0.2	84	24
External Dedicated Through Movement						
B	EB	2	11	1.0	na	27
B	EB	3	11	1.0	na	27
B	WB	2	10	1.7	na	21
B	WB	3	10	1.7	na	21
A	SB	2	11	1.4	na	40
C	SB	2	11	3.2	na	36
C	SB	3	11	3.2	na	36
External Left Turn Movement (Four-phase with overlap timing)						
G	NB	2	12	3.5	na	43
G	NB	3	12	3.5	na	43

1. Curb lane = 1

2. Measured from extension of face of curb from roadway to right

3. na = Not applicable

TABLE 2 Interchange Timing by Movement

Interchange	Movement	Minimum Green Time (sec)	Yellow Time (sec)	All Red Time (sec)	Type of Phasing
A	SB Left	38	4.0	2.0	Four Phase
A	SB Through	57	4.0	2.0	Four Phase
A	WB Left	15	4.5	1.5	Four Phase
B	NB Ramp	30	3.9	1.8	Four Phase
B	SB Ramp	30	3.9	1.7	Four Phase
B	E&W Through	63	3.9	1.0	Four Phase
C	WB Ramp	30	4.5	1.5	Three Phase
C	SB Through	47	4.0	2.0	Three Phase
D	NB Left	20	3.9	2.0	Four Phase ¹
E	NB Through	60	3.9	2.0	Three Phase
F	WB Ramp	30	4.5	1.5	Four Phase
G	NB Left	56	3.9	2.0	Four Phase with Overlap

1. All Extra WB ramp time is given to NB left

inputs the location, direction, lane, and yellow and all-red time, headways and actual phase and interval lengths can then be collected by actuating various keys. For this study, the top bar line was used as the roadway reference point and the rear wheel of the vehicle was used as the vehicle reference point. Data were collected for all vehicles, but the analysis in the paper is for passenger cars only. At these locations, there were no cycles used in the analysis in which pedestrians interfered with the right-turning traffic.

Once data are collected, they are written to an ASCII text file for further processing.

Data Analysis

A program was written in Fortran IV that reads the data file from the collection program and calculates start-up time, clearance time, and saturation headways. The program, ANALSAT, reads the data file collected with COLSAT. The first portion of ANALSAT prints out a report of the time, date, location, signal times, and headways of each study. The second part of the program analyzed the headways.

TABLE 3 Variation of Standard Error of Mean

Data Set	Size	Standard Deviation	Standard Error of the Mean
ALL VEH	180	.76	.056
N-1	160	.71	.056
N-2*	140	.61	.052
N-3	120	.52	.047
N-4	100	.49	.049
N-5	81	.51	.057

* For example, all vehicles except the first two in the queue

RESULTS—CONVENTIONAL DIAMOND INTERCHANGES

Saturation Headways

The first step in determining the saturation headways was to determine which vehicles experience starting headway lost time and which vehicles do not. One of the ANALSAT calculated parameters is the standard error of the mean. This value is the standard deviation divided by the square root of the sample size. The parameter is an estimation of how much

error is associated with each data set. This parameter was useful in determining which vehicles experienced starting headway lost times. An example of how the standard error of the mean varied with different data sets is given in Table 3.

As shown in Table 3, the first few data sets have a large standard deviation, but after about N-3 vehicles, this value levels off to about .43 to .44. However, as vehicles are subtracted from the analysis, the standard error begins to get larger. Several outputs from ANALSAT were studied for different movements, and it was thought that a good approximation was that the first three vehicles experienced starting headway lost times and the fourth through the last vehicles were used in calculating average headways. This is consistent with results found by Poppe (1), who also determined that the first three vehicles experience starting headway lost time at single-point diamond interchanges.

The results for headway measurements for each of the lane movements are shown in Table 4. Saturation flow rates are calculated by dividing 3,600 sec/hr by the mean headway.

The results of the headway measurements are widely scattered. Of the 22 study lanes, only 4 locations have saturation flow rates less than or equal to the 1,800 set forth in the 1985 *Highway Capacity Manual*.

Another interesting observation is that the range of saturation flow rates is very similar for the different movements.

TABLE 4 Study Lane Mean Saturation Headways and Mean Saturation Flow Rates

Location	Sample Lane ¹	Mean Size (N)	Mean Headway (sec)	Sample Standard Deviation (sec)	95% Confidence Interval (sec)	Mean Saturation Flow Rate (vphgpl) ²
Interior Left Turn Movement						
1	3	131	1.89	0.45	1.81 - 1.97	1905
1	4	111	2.02	0.56	1.91 - 2.12	1782
2	3	210	1.71	0.41	1.66 - 1.77	2105
2	4	305	1.81	0.43	1.76 - 1.86	1988
Interior Through Movement						
3	1	88	1.87	0.48	1.77 - 1.98	1925
3	2	111	1.75	0.46	1.66 - 1.83	2057
Ramp Dedicated Left Turn Movement						
4	3	125	1.99	0.56	1.89 - 2.09	1809
5	3	111	1.84	0.53	1.74 - 1.94	2068
6	2	203	1.78	0.44	1.72 - 1.84	2022
7	3	67	1.89	0.49	1.77 - 2.01	1905
8	3	83	1.89	0.43	1.80 - 1.99	1905
Ramp Dedicated Right Turn Movement						
7	1	109	2.00	0.58	1.89 - 2.11	1800
8	1	63	1.84	0.48	1.72 - 1.96	1956
External Dedicated Through Movement						
9	2	94	1.85	0.52	1.75 - 1.96	1945
9	3	122	1.73	0.37	1.66 - 1.79	2080
10	2	148	2.01	0.61	1.91 - 2.11	1791
10	3	119	2.01	0.57	1.90 - 2.11	1791
11	2	117	1.74	0.46	1.66 - 1.82	2068
11	2	99	1.83	0.57	1.71 - 1.94	1967
11	3	170	1.85	0.71	1.74 - 1.96	1946
External Left Turn Movement (Four Phase with overlap timing)						
12	2	124	1.86	0.45	1.78 - 1.94	1935
12	3	108	1.89	0.45	1.80 - 1.97	1904

1. Curb lane = 1

2. Vehicles per hour of green per lane

TABLE 5 Range of Saturation Flow Rates for Various Movements

Movement	Number of Locations	Range of Saturation Flow Rates (vphgpl)	Mean Saturation Flow Rate (vphgpl)
Arterial Left Turn	4	1782 - 2105	1975
Ramp Left Turn	5	1809 - 2068	1956
Ramp Right Turn	2	1800 - 1956	1857
Arterial Through	9	1791 - 2080	1948

Table 5 shows the range and mean of saturation flow rates for the various movements. The mean in Table 5 is the weighted average of the individual flow rates shown in Table 4.

Table 5 indicates that saturation flow rates are in the 1,800- to 2,100 range for all movement types. The mean saturation flow rate is 1,948 to 1,975 vehicles per hour for all left and through movements. The right-turning movement has a much lower saturation flow rate than the other movements. This flow rate may be due to the short-radius right turn that the vehicles must make during a right turn.

The saturation flow rates measured in this study are higher than those recommended in the 1985 Highway Capacity Manual. For "ideal" conditions, the manual recommends using 1,800 (units) as the maximum possible saturation flow rate. These results in Phoenix are much higher, even though some locations did not have ideal conditions. The left-turn movements have a mean saturation flow rate that is higher than the through movements. The manual recommends using 95 percent of the through-movement value to find the left-turning value. It appears that at conventional diamond interchanges, the left-turn value should be equal to or greater than the through-movement value.

The data also show an interesting result with respect to left turns. In both cases in which there were dual left turns from the arterial street, the lane with the larger turning radius had a shorter headway. The different turning radii probably affect driver perception and behavior. To determine if there is a strong correlation between turning radius and headway, these parameters were plotted against the data from this research and the Poppe study (1). The 21 data points of radius versus the saturation flow rate are plotted in Figure 3.

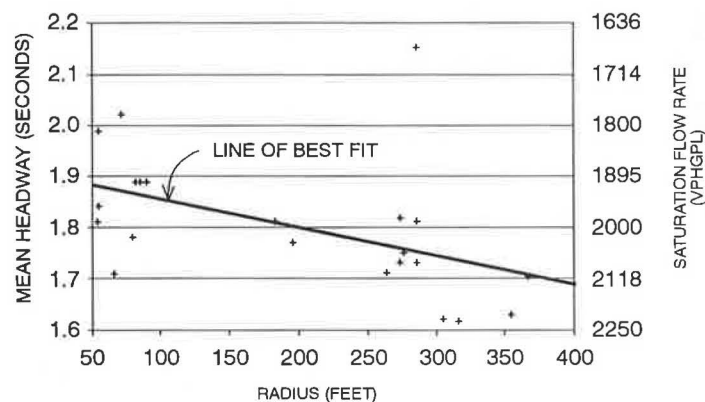


FIGURE 3 Plot of radius versus headway for left-turn movements.

There are two clusters of data points in the figure: the cluster from 50 to 100 ft is data points from conventional diamond interchanges, and the cluster from 250 to 350 ft is data points from the single-point diamond interchange. The data points for radii between the two clusters fall on the regression line. There is a tendency for headways to decrease as turning radius increases, even though the r^2 -value is low (.23).

Start-Up Lost Time

Because the first three vehicles were determined to be experiencing start-up lost time, the start-up lost time for a phase was calculated as the sum of headways for the first three vehicles in the queue minus three times the mean headway. It is interesting to note that when the saturation flow rate is higher, the start-up lost time is also higher. This is explained by the fact that three times the mean headway is subtracted from the sum of the first three vehicles. If the headways are smaller, then the result is a larger number. The results of the start-up lost time by study lane are shown in Table 6.

Start-up lost times for similar movements are combined to show start-up headways by movement type:

Movement	Range (sec)	Mean (sec)
Arterial left turn	1.27-1.68	1.41
Ramp left turn	1.19-2.03	1.62
Ramp right turn	1.39-1.52	1.43
Arterial through	0.80-2.40	1.56

The mean start-up lost time for all movements is in the range of 1.41 to 1.62 sec. This value is lower than the 2.0 sec recommended in the 1985 Highway Capacity Manual.

Clearance Lost Time

The clearance lost time is the time from when the last vehicle passes the stop bar until the start of green for the next phase. Because not every approach was saturated, clearance lost times were not collected for some movements. The results of clearance lost times are shown in Table 6.

The range of clearance lost times is from 2.8 to 4.7 sec. There doesn't appear to be any correlation with saturation flow rate and lost time. There also doesn't appear to be any correlation with movement type and clearance lost time.

TABLE 6 Study Lane Mean Start-Up Lost Time and Mean Clearance Lost Time

Location	Lane ¹	Mean Start-up Lost Time				Mean Clearance Lost Time				
		Sample Size (N)	Mean Start-up Lost Time (sec)	Sample Standard Deviation (sec)	95% Confidence Interval (sec)	Sample Size (N)	Mean Clearance Lost Time (sec)	Sample Standard Deviation (sec)	95% Confidence Interval (sec)	
Interior Left Turn Movement										
1	3	13	1.35	1.11	0.69 - 2.02	0	na			
1	4	11	1.40	0.80	0.86 - 1.93	0	na			
2	3	33	1.68	1.81	1.03 - 2.32	20	3.51	0.97	3.06 - 3.96	
2	4	56	1.27	1.38	0.90 - 1.64	33	4.22	1.26	3.78 - 4.67	
Interior Through Movement										
3	1	15	1.20	0.67	0.83 - 1.57	0	na			
3	2	16	1.63	0.83	1.18 - 2.07	0	na			
Ramp Dedicated Left Turn Movement										
4	3	18	1.20	0.91	0.74 - 1.65	0	na			
5	3	14	1.89	1.14	1.23 - 2.54	3	3.08	0.27	2.41 - 3.75	
6	2	36	2.03	1.52	1.52 - 2.55	28	2.78	1.32	2.27 - 3.29	
7	3	19	1.19	1.09	0.67 - 1.72	0	na			
8	3	14	1.43	1.09	0.80 - 2.06	2	4.71	2.03		
Ramp Dedicated Right Turn Movement										
7	1	30	1.39	0.89	1.06 - 1.72	0	na			
8	1	17	1.52	1.44	0.78 - 2.26	0	na			
External Dedicated Through Movement										
9	2	10	1.75	1.40	0.75 - 2.76	0	na			
9	3	9	1.94	1.21	1.01 - 2.87	0	na			
10	2	13	0.80	1.01	0.19 - 1.41	0	na			
10	3	7	1.79	0.99	0.88 - 2.70	0	na			
11	2	20	2.40	1.32	1.78 - 3.02	0	na			
11	2	11	1.27	0.85	0.70 - 1.84	0	na			
11	3	13	1.09	0.61	0.72 - 1.46	2	3.88	0.07	3.25 - 4.52	
External Left Turn Movement (Four Phase with overlap timing)										
12	2	25	1.16	2.24	0.23 - 2.08	2	3.23	0.33	0.24 - 6.21	
12	3	13	0.64	0.97	0.06 - 1.23	3	3.54	1.05	0.94 - 6.14	

1. Curb lane = 1

Comparison of Low- and High-Design Interchanges

A comparison was made between the left-turning movements at Interchange A with Interchange D. Interchange A is typical of a low-design standard. The interchange has 10-ft left-turn lanes, and the radius of the left turn is 54 to 66 ft. Interchange D is typical of a high-design standard. This interchange has 12-ft lanes and turning radii of 72 to 82 ft. The approach grades for both interchanges are nearly level. A statistical *t*-test was performed, and the saturation flow rate of Interchange D (1,988 to 2,105) is statistically significantly higher than that of Interchange A (1,782 to 1,905) at the 95 percent confidence level.

for external through movements, there were seven samples for conventional diamond interchanges and seven samples for single-point diamond interchanges. Data were collected for dedicated right-turn movements at conventional diamond interchanges, but these data were not collected at single-point diamond interchanges in the Poppe study. A one-tailed *t*-test was performed for each movement type, and the results of the comparisons are shown in Table 7.

As shown in Table 7, the dedicated through movement is very similar for the two interchange types. The single-point diamond interchange has a slightly higher saturation flow rate, but it is not statistically significant. The arterial dedicated left-

COMPARISON OF CONVENTIONAL AND SINGLE-POINT DIAMOND INTERCHANGES

Saturation Headways

Data for saturation headways for single-point diamond interchanges were obtained from Poppe (1). Both this study and the Poppe study used the rear axle as the vehicle reference point and the stop bar as the intersection reference point. Both studies assumed that the first three vehicles in queue experience start-up lost time. To compare the two interchange types, it was assumed that each study lane for each movement type was a sample for that interchange type. Therefore,

TABLE 7 Comparison of Mean Headways Between Conventional and Single-Point Diamond Interchanges

Interchange Type	N	Mean Saturation Flow Rate (vphgpl)	Statistically Significant?
Ramp Dedicated Left			
SPDI	8	2069	
CDI	4	1915	Yes
Arterial Dedicated Left			
SPDI	6	2045	
CDI	4	1935	No
Arterial Dedicated Through			
SPDI	7	1935	
CDI	7	1895	No

turn movement is much higher for the single-point diamond interchange, but the results are not significantly different. The ramp exclusive-left movement, however, shows a much higher saturation flow rate for the single-point diamond interchange than for the conventional diamond interchange and is statistically significant at the 95 percent level.

It is interesting to note that the left turn from the arterial street does not have a statistically significant difference in saturation flow rates between the two interchange forms but the left turns from the ramp do. One possible explanation is that of geometric configuration. At both interchange types, the left turn from the arterial street must turn through 90 degrees to reach the ramp. However, at the ramp locations the situation is different. The ramp at the conventional diamond interchange is nearly perpendicular to the arterial street; the ramp at the single-point diamond interchange is at a large skew to the intersection. The angle that ramp-turning traffic must turn through is much less at the single-point diamond.

Start-Up Lost Time

A comparison of start-up lost times was performed using the same sample sizes used in the headway comparison. A one-tailed *t*-test was performed to compare the two interchange types. The comparison and statistical significance of the results are shown in Table 8.

The start-up lost times for each interchange type are very similar. The start-up lost times are almost identical for the two interchange forms, except for the ramp left movement. Again, because start-up lost time is a function of the saturation flow rate, it is not surprising that the conventional diamond interchange has a lower start-up lost time.

Clearance Lost Time

A comparison of clearance lost times was performed for the two interchange forms. The results (Table 9) show that clearance lost times are much higher for single-point diamond interchanges than for conventional diamond interchanges. This is not surprising, because the intersection widths at single-point diamond interchanges can be quite large. A statistical significance was not expected for the ending lost times because of the small sample size. However, it is quite clear that the ending lost times are quite different.

TABLE 8 Comparison of Start-Up Lost Time Between Conventional and Single-Point Diamond Interchanges

Interchange Type	N	Start-up Lost Time (sec)	Statistically Significant?
Ramp Dedicated Left			
SPDI	8	1.84	No
CDI	4	1.49	
Arterial Dedicated Left			
SPDI	6	1.43	No
CDI	4	1.39	
Arterial Dedicated Through			
SPDI	7	1.55	No
CDI	7	1.58	

TABLE 9 Comparison of Clearance Lost Time Between Conventional and Single-Point Diamond Interchanges

Interchange Type	N	Clearance Lost Time (sec)	Statistically Significant?
Ramp Dedicated Left			
SPDI	3	6.67	Yes
CDI	3	3.52	
Arterial Dedicated Left			
SPDI	2	3.97	No
CDI	2	3.87	
Arterial Dedicated Through			
SPDI	3	6.41	Yes
CDI	3	3.80	

The arterial dedicated left turn is not statistically different. In fact, the two means are almost identical. This is because the clearance time in the controller at one of the Poppe single-point diamond interchanges was timed with very short yellow and all-red times.

CONCLUSIONS

The results of this research show that there is no significant difference for saturation flow rates for through movements between the two interchange forms. The saturation flow rates are almost the same. This seems reasonable because there is very little operational difference for through movements between the two interchange forms.

This research found that there is no significant difference for saturation flow rates for left turns from the arterial street. For left turns from the ramps, there is a significant difference in saturation flow rates. One possible explanation for the difference in saturation flow rates for left-turning ramp traffic is that this movement must turn through a smaller arc than the left-turning ramp traffic from a conventional diamond interchange.

There is no significant difference in start-up lost time between the two interchange types. The results show that the start-up lost times for the arterial movements are almost the same value for each interchange type.

There is a statistically significant difference in clearance lost times between the single-point diamond interchange and the conventional diamond interchange for the ramp left-turn movement and the arterial through movement. However, there is no significant difference between lost times for the arterial dedicated-left movement. Single-point diamond interchanges typically have much longer clearance intervals; therefore, it is expected that the clearance lost times would be greater.

It appears that saturation flow rates of the left-turn movement from the arterial street are higher for high-design conventional diamond interchanges than for low-design conventional diamond interchanges.

The study results will be useful in refining the capacity of a conventional diamond interchange. Typically, interchanges are designed to handle the projected traffic volumes. This research shows that flow rates through interchanges in the metropolitan Phoenix area are much higher than put forth in the 1985 *Highway Capacity Manual*. This could translate into

thousands of dollars if the larger actual capacity were to reduce the need for additional lanes at an interchange location.

The results of this research will also be of interest to those who recommend interchange forms. There are differences in saturation flow rate and lost time for the two interchange types for some movements. When analyses are performed to determine intersection capacity, these data will provide for a more accurate prediction of interchange capacity. The conventional diamond interchange has lower saturation flow rates for the left turns from the ramp, yet it has lower lost time for the ramp and through movements. Depending on the site-specific volumes and geometric designs, the capacity of both interchange forms can be determined. An analysis can be performed to determine the design for each interchange type that can handle the projected traffic and the associated construction costs. If it is determined that both can handle the projected traffic, perhaps the less costly of the two types can be constructed.

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