EVALUATION OF OPERATIONAL EFFICIENCIES, COST AND ACCIDENT EXPERIENCE OF FOUR PHASE SINGLE POINT URBAN INTERCHANGES

Final Report 501

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16. Abstract

This research compares two interchanges types, the Tight Urban Diamond Interchange (TUDI) and the Four Phase Single Point Urban Interchange (4ØSPUI). The 4ØSPUI is referred to in this report as a Single Point Urban Interchange with Frontage Roads (SPUI/F).

The objectives of this research are to:

1. Evaluate the SPUI/F based on available accident data and conflict analysis techniques, right-of-way and construction costs, and operating efficiency.
2. Compare the performance of the SPUI/F and the TUDI.
3. Evaluate current SPUI/F design assumptions and operation; recommend design and/or operational changes to enhance performance.
4. Evaluate the interchange form selection (pre-design) process and to recommend changes where appropriate.

The research found no significant difference in the safety aspects of the two interchange types. The SPUI/F generally did not perform as well operationally as the TUDI, with increased SPUI/F delay as the distance between frontage roads increased.

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	2.54	centimeters	cm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	6.452	centimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.395	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	millimeters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0328	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

Note: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

These factors conform to the requirement of FHWA Order 5190.1A

*SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
yd	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
yd^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.53	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	millimeters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
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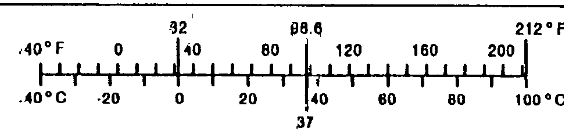
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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INTRODUCTION

One of the most important elements of our transportation infrastructure is the urban diamond interchange. There are several reasons for this, some of which are as follows:

- Operations. Historically, interchanges, especially diamond interchanges have been among our most congested facilities. Reasons for this may include rapid increase in traffic volumes, under-projected design volumes, imprecise analysis tools and less than optimum operational methods.
- Right-of-way and construction costs. Because of high right-of-way costs in urban areas, extra roadway capacity may not have been provided where it is otherwise desirable.
- Safety. With the high volumes of arterial street traffic interfacing with the large turning movements to and from a freeway, the diamond interchange is one with significant potential for traffic crashes.

In recent years, Arizona has seen considerable use of the single-point urban interchange (SPUI), which has become very popular with roadway designers and the motoring public. Some reasons for its popularity include:

- Increased left turn efficiency potential. The use of "inside left turns" to reduce the number of traffic signal phases for the traditional SPUI increases the left turn efficiency. Possibility of reduced arterial street right-of-way. This is primarily because of the use of "inside left turns" from the arterial street can be provided in the same right-of-way longitudinally. This is because they don't overlap across the structure.
- Simplified timing. Conventional diamond interchanges require special signal timing, the phasing for which varies depending on traffic volumes and ramp spacing. The SPUI can be effectively timed using a standard eight-phase signal controller. The only significant difference between timing it and a regular eight-phase intersection is the required change intervals.

There are also disadvantages associated with the SPUI, primarily the increased structure cost due to the large span and the lack of space for a center overpass bridge pier. Additionally, the ramp horizontal geometry necessary to accommodate the inside left turns will often require more right-of-way than the compact diamond.

The first SPUI constructed in Arizona which included frontage roads was at the Squaw Peak Parkway (now SR 51) and Thomas Road. This interchange type has become known as a Four-Phase Single Point Urban Interchange (4ØSPUI) or Single Point Urban Interchange with Frontage Roads (SPUI/F). The SPUI/F name is used in this research report. Other SPUI/F which have been constructed in the Phoenix metropolitan area in recent years include:

- I-17 / Northern Avenue
- I-17 / Bethany Home Road
- Loop 101 / Guadalupe Road
- I-17 / Dunlap Avenue
- I-17 / Camelback Road
- I-17 / Glendale Avenue
- Loop 101 / Frank Lloyd Wright Boulevard

It is particularly critical that the most efficient interchange type for the prevailing conditions be used in the Maricopa Association of Governments (MAG) region because we are now building our freeway system that will be with us for many years. The interchange constructed should operate efficiently for the next 20 years. Major modifications to interchanges are not only expensive, but they subject both the motoring public and construction workers to delays and increased hazard. It is likely that different interchange types provide the best results under differing conditions. The purpose of this research is to determine which of two interchanges types, Tight Urban Diamond Interchange (TUDI) or SPUI/F is preferable and under what conditions, to provide guidance on the selection of the appropriate type and to provide guidance on the design and operation of the SPUI/F.

SCOPE OF RESEARCH

The objectives of this research are to:

1. Evaluate the SPUI/F based on available accident data and conflict analysis techniques, right-of-way and construction costs, and operating efficiency.
2. Compare the performance of the SPUI/F and the TUDI.
3. Evaluate current SPUI/F design assumptions and operation; recommend design and/or operational changes to enhance performance.
4. Evaluate the interchange form selection (pre-design) process; recommend changes where appropriate.

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CHAPTER 1
LITERATURE REVIEW: FOUR-PHASE SINGLE POINT URBAN INTERCHANGE

INTRODUCTION

Overview

The single point urban interchange (SPUI) has been the subject of intensive research during the past 15 years. Research projects (*1, 2, 3, 4*) have been conducted for several state departments of transportation (DOTs), including the Arizona DOT (*1*), Texas DOT (*2*), Virginia DOT (*3*), and Michigan DOT (*4*). A major project (*5*) was also conducted for the National Cooperative Highway Research Program, on behalf of the American Association of State Highway and Transportation Officials. These projects evaluated the design and operation of the SPUI and also offered comparisons of it to other intersection and interchange forms (e.g., high-type at-grade intersection and diamond interchange). The projects have focused on SPUIs that are <u>not</u> associated with frontage road systems along the major roadway (i.e., SPUI/n). This latter focus was dictated primarily by the fact that few SPUIs included frontage roads at the time the research was conducted.

This chapter documents a critical review of the literature on the operation and safety of the "SPUI with frontage roads" (i.e., SPUI/F). This review identifies a range of issues related to SPUI/F operation including: signal phasing, signal coordination, clearance interval timing, saturation flow rate, efficiency, and pedestrian phase timing. In some instances, the review compares the SPUI/F with the tight urban diamond interchange (i.e., TUDI) to provide a context for the discussion. Such context is also provided through comparison of the SPUI/F with the more commonly found SPUI/n. References *1, 2, 3, 4,* and *5* describe the design and operation of the SPUI/n.

Several issues related to SPUI safety are addressed including: accident rates, prevalent accident types, conflict studies and pedestrian safety. Because of the limited number of SPUI/Fs, no safety studies were found for the SPUI/F.

Definitions

<u>Tight Urban Diamond Interchange</u>

The TUDI form is commonly used in densely developed urban areas. This interchange has two ramp (or frontage road) terminals that are separated by about 60 to 120 m (200 to 390 ft) on the cross street. Each terminal is signalized and serves traffic with a three-phase sequence. In most instances, one signal controller is used to control the signals at both terminals. Because the SPUI is also well-suited to constrained urban environments, the TUDI and the SPUI are competitors at many urban locations.

The conventional diamond interchange and the compressed diamond interchange have the same basic shape as the TUDI, however, they are not often used in constrained urban environments because of their large ramp separation distances. Specifically, the conventional diamond interchange has ramp terminals separated by 245 m (800 ft) or more. The compressed diamond interchange has ramp terminals separated by 120 to 245 m (390 to 800 ft).

Overpass vs. Underpass SPUI

One means of describing a SPUI’s design is based on the manner in which the two intersecting alignments are vertically separated. One type of SPUI has the major-street through movements passing above the ramp/cross-street intersection. This interchange is termed an “overpass” SPUI, as shown in Figure 1. All eight of the SPUI/Fs observed by Messer *et al.* (*5*) have the overpass design.

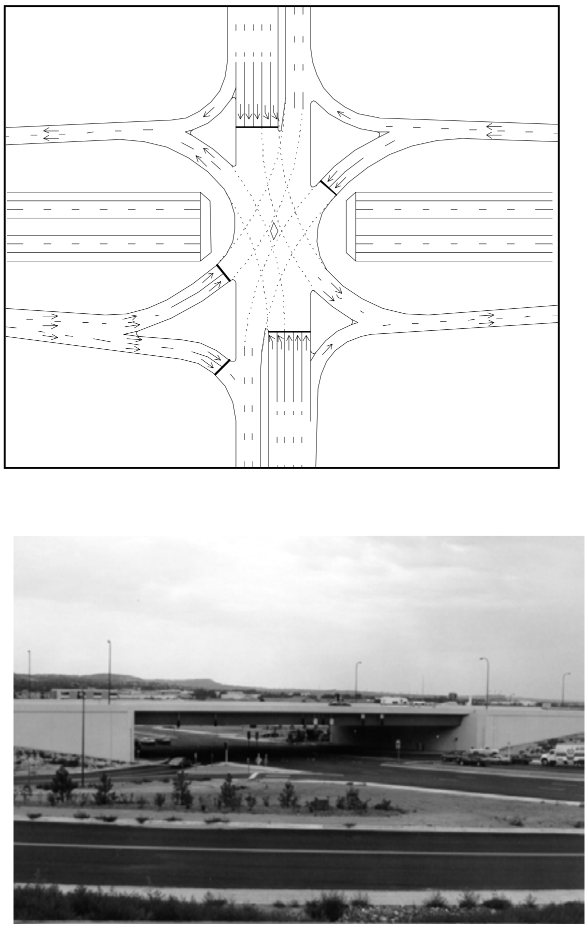


Figure 1. Overpass SPUI design.

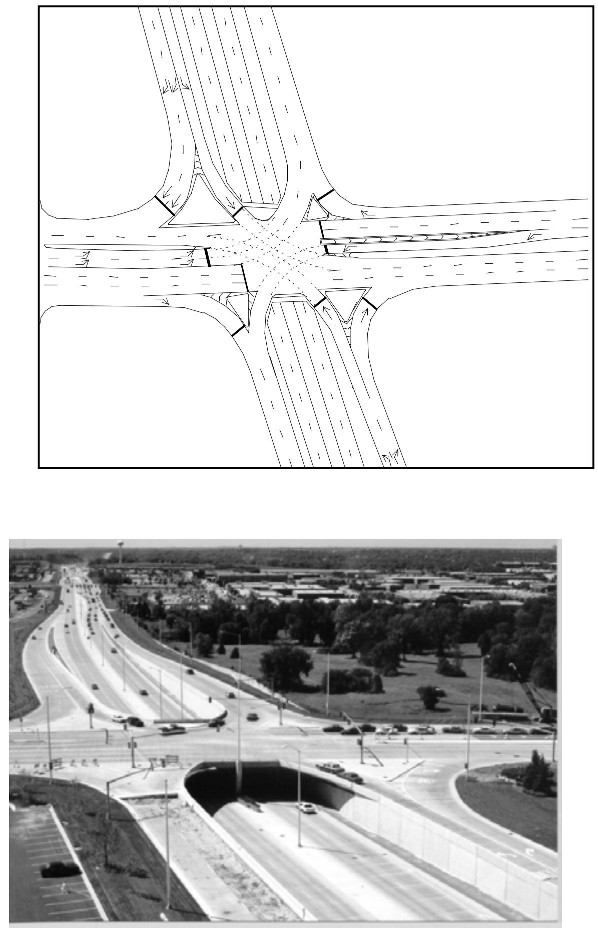


Figure 2. Underpass SPUI design.

SPUIs that have the major-street through movement passing under the ramp/cross-street intersection are called “underpass” SPUIs. This type of SPUI is shown in Figure 2. The advantage of the underpass SPUI (relative to the overpass SPUI) is that the intersection conflict area is relatively open, well lit, and resembles that of other high-type at-grade intersections. Dorothy *et al.* (*4*) noted these advantages in a recent observational study of SPUIs in six states. The disadvantages of the underpass design include: it requires a more complicated and expensive bridge structure, the parapet walls associated with the bridge railing tend to block sight lines for ramp traffic, and the open conflict area appears unnecessarily expansive (and possibly more intimidating) to drivers.

Single Point Urban Interchanges with Frontage Roads

In a 1991 report, Messer *et al.* (*5*) noted that there were 36 operational SPUIs of which only 11 (31%) had frontage roads. The 11 SPUI/Fs found to have frontage roads were noted to be one of two types: (1) frontage roads combined with on/off-ramp terminals, and (2) frontage roads offset from the on/off-ramp terminals. The majority (i.e., eight) of these SPUI/Fs had the frontage roads combined with the on/off-ramp terminals. Two examples of this latter type of SPUI/F are provided in Figure 3.

Interchanges in frontage road systems tend to have significant U-turn traffic volumes. The SPUI/F design is able to serve U-turn traffic efficiently within the signal phase. However, even greater efficiency is achieved by the provision of exclusive U-turn lanes. The disadvantage of these lanes is that they require a longer bridge structure.

The eight SPUI/Fs observed by Messer *et al.* (*5*) were found to vary in physical size. One measure of this size is the distance between the opposing cross-street stop lines. In 1991, this distance was found to range from 64 to 82 m (210 to 270 ft). The largest SPUI/F found by Messer in 1991 is shown in Figure 3a. Larger SPUI/Fs have been constructed since then, one of which is shown in Figure 3b.

OPERATIONAL ELEMENTS

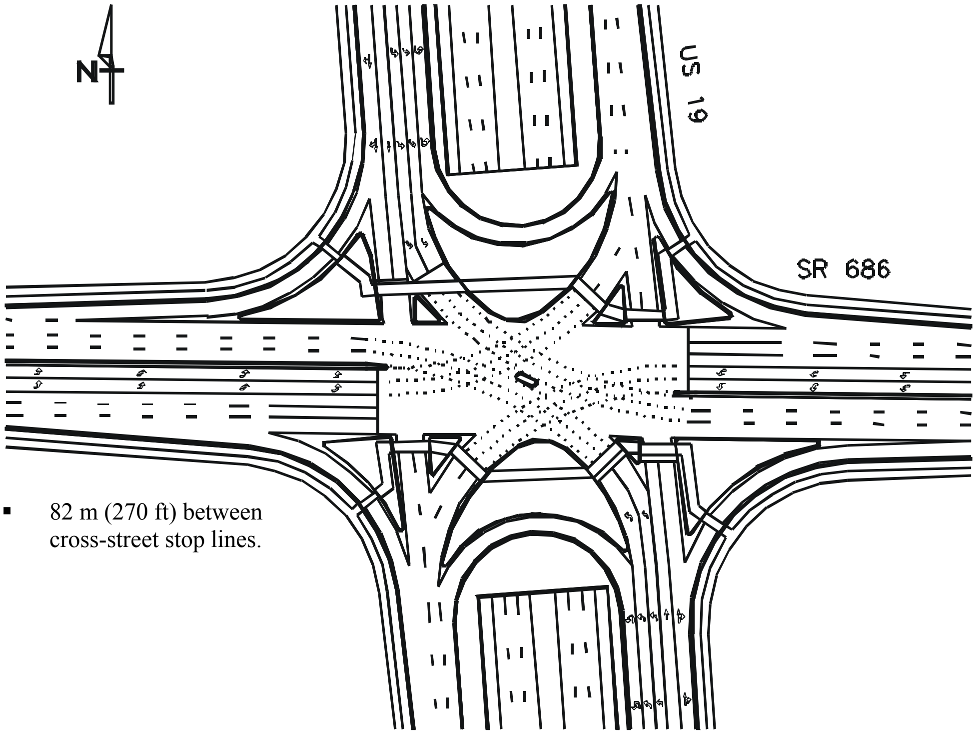
Signal Phase Sequence

Messer *et al.* (*5*) reported that the SPUI/F typically uses one actuated controller and operates with four basic phases, one typical phasing arrangement is shown in Figure 4.

The phasing shown in Figure 4 is referred to as "lead-lead" phasing because the left-turn phases precede the through phases. In this figure, Phases 3 and 4 serve the frontage road movements. Initially, Phase 3 serves the left-turn movements together in a "leading left" arrangement. Next, an "overlap" phase is available when left-turn demands are unbalanced (i.e., unequal in volume). Finally, Phase 4 provides for the simultaneous service of the frontage road through movements.

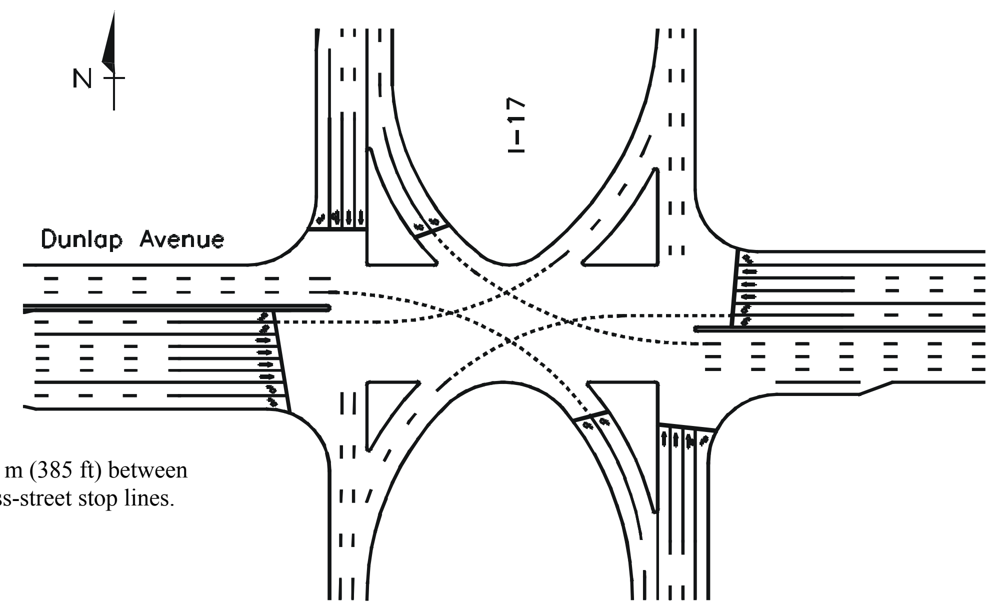
Other phase sequences can be used at the SPUI/F. Specifically, "lead-lag," "lag-lag" and "split" phasing are also possible. Lead-lag phasing has the cross-street left-turn phase preceding the cross-street through phase and the frontage-road left-turn following the frontage-road through phase. Lag-lag phasing has both left-turn phases following the through phases. Finally, split phasing serves the frontage-road through and left-turn movements on a common approach at the same time. Two phases are used in this scheme, one for each frontage-road approach.

Messer *et al.* (*5*) report that lagging left-turn phases have been used to reduce the clearance interval duration for some SPUI phases (relative to that used for leading left-turn phases). Such a reduction is attractive because it can increase interchange capacity by minimizing phase lost time. The reduced clearance interval duration stems from consideration of the clearance path length associated with each signal phase, relative to the travel path of the movement that enters on the *subsequent* phase. Because of this dependence on one phase always following another phase, this approach is only applicable to pretimed control. It can be used for actuated control when all phases are recalled to display the green interval each cycle; however, the recall defeats some of the benefits of actuated control.



- 82 m (270 ft) between cross-street stop lines.

a. Overpass SPUI/F with U-turn lanes.



- 117 m (385 ft) between cross-street stop lines.

b. Underpass SPUI/F with no U-turn lanes.

Figure 3. Alternative frontage road arrangements at the SPUI/F.

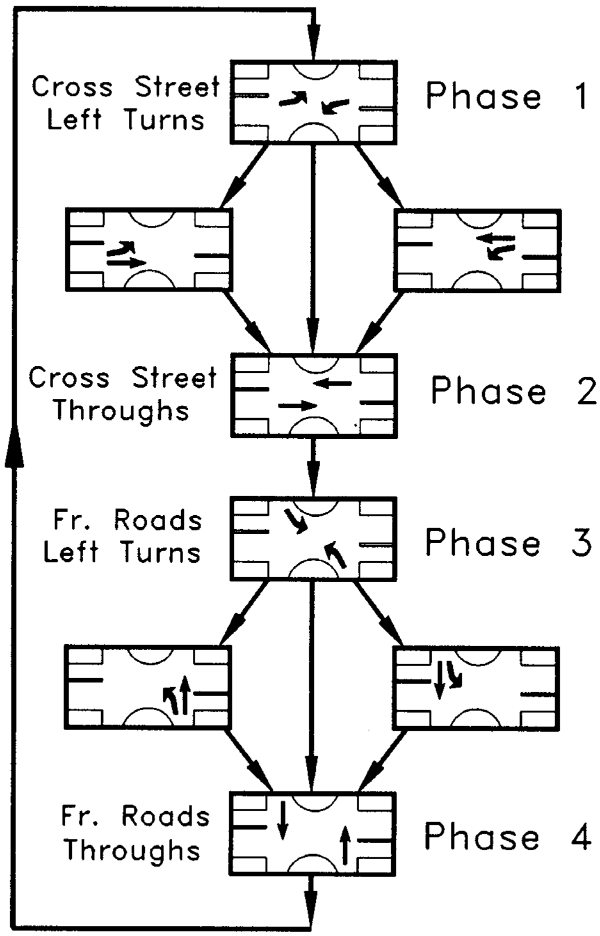


Figure 4. Typical SPUI/F phase sequence.

Messer *et al.* (*5*) report that split (or directional) phasing is used when the lateral separation between the opposing frontage-road left-turn travel paths is inadequate. This problem can be caused by one of two factors: (1) there is extreme skew in the intersecting alignments, or (2) the left-turn paths were "pushed" together in the design in an effort to minimize the size of the conflict area. This latter problem often surfaces when extreme measures are taken to minimize the size of the bridge structure. It should be noted that lateral separation distances are believed to be "inadequate" when they are less than about 2.0 m (6.5 ft).

Finally, Messer *et al.* (*5*) report that left-turn phases at the 36 operational SPUIs included in their survey used a "protected-only" mode. Protected-permitted left-turn operation is not used because of the large expanse of the conflict area, the length of the left-turn travel path, and the difficulty of viewing the left-turn signal head when the permitted left-turn driver creeps ahead of the stop line (in preparation for the permitted maneuver).

Signal Coordination

A recent survey by Garber and Smith (*3*) indicated that a large percentage of engineers believe that the SPUI is much easier to coordinate with adjacent signalized intersections on the cross road (relative to the TUDI). The justification for this claim is that the SPUI has only one signalized junction whereas the TUDI has two, closely-spaced junctions. On the other hand, Dorothy *et al.* (*4*) report that the long cycle length associated with the SPUI (relative to the cycle used at adjacent intersections) tends to make coordination difficult. This problem may be particularly acute for the SPUI/F as its lengthy clearance intervals and fourth phase tend to make cycle lengths extremely long (even when compared to the SPUI/n).

Clearance Interval and End Lost Time

The timing of the all-red clearance interval at a SPUI requires special consideration because of the SPUI's large size and lengthy travel paths. This requirement was noted by Leisch *et al.* (*6*) who found that SPUIs often need clearance intervals in excess of 8 to 9 s. They pointed out that long clearance intervals create considerable lost (or unused) time at the end of the phase and, thereby, have an adverse effect on a SPUI's phase capacity.

End Lost Time

The lost time at the end of a phase is equal to the change interval duration (i.e., yellow warning interval plus all-red clearance interval) less the initial portion of the yellow warning interval that is typically used by clearing drivers. Bonneson (*7*) and Poppe *et al.* (*8*) report that this "end-use" of the yellow interval varies between 2.5 and 3.0 s. Both of these authors found that end lost times ranged from 3 s for the frontage road-through phase to 8 s for the cross-street through and left-turn phases. For four-phase operation, this translates into about 25 s during each signal cycle that vehicles are not being served by the SPUI/F.

Phase Change Interval Duration

One methodology for calculating the duration of the phase change interval is that proposed by Technical Committee 4A-16 working under the direction of the Institute of Transportation Engineers (ITE) (*9*). The formula recommended by this committee for determining the length of the yellow interval is:

$$Y(v) = T_{pr} + \frac{V_a}{2\ d_r + 2\ g\ G_r} \tag{1}$$

where:
$Y(v)$ = yellow interval evaluated at speed $V_a = v$, s;
d_r = deceleration rate, use 3.05 m/s^2 (10 fpss);
g = gravitational acceleration, use 9.81 m/s^2 (32.2 fpss);
G_r = approach grade, m/m;
T_{pr} = driver perception-reaction time, use 1.0 sec; and
V_a = speed of vehicle approaching the intersection, m/s.

The all-red clearance interval follows the yellow indication and is intended to provide time for those vehicles entering during the yellow to safely clear the intersection conflict area. If there is sufficient pedestrian activity, the extent of the conflict area is extended to include the crosswalk. The all-red interval is calculated as:

$$AR(v) = \frac{L + d}{V_c} \tag{2}$$

where:
$AR(v)$ = all-red interval evaluated at speed $V_c = v$, s;
d = length of the average vehicle, use 6.10 m (20 ft);
L = the length of the clearance path, m; and
V_c = speed of clearing vehicle, m/s.

The value of L in Equation 2 is based on the amount of pedestrian activity. It is computed as:

- If there is no pedestrian activity then $L = L_w$.
- If there is some activity then L = the larger of $L_p - d$ or L_w.
- If there is significant activity then $L = L_p$.

where:
L_w = length of the clearance path measured from the near-side stop line to the far edge of the farthest conflicting traffic lane along the actual vehicle path, m; and
L_p = length of the clearance path measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path, m.

Phase Change Interval Calculation for Through Movements. Application of Equations1 and 2 is specific to the maneuver made by the approaching vehicle. For a through movement, the ITE technical committee recommends that the phase change interval should be calculated twice: once for the 15th percentile approach speed V_{15} and once for the 85th percentile approach speed V_{85} (the difference between V_c and V_a is considered to be negligible). The longer of these two change intervals is then used. The yellow interval is always based on the 85th percentile speed. The all-red clearance interval is based on the difference between the change interval and the yellow interval. This procedure is summarized in the following calculation steps:

Step 1. $CI_{85} = Y(V_{85}) + AR(V_{85})$
Step 2. $CI_{15} = Y(V_{15}) + AR(V_{15})$
Step 3. CI = Larger of CI_{85} or CI_{15}
Step 4. $Y = Y(V_{85})$
Step 5. $AR = CI - Y(V_{85})$

In these equations, CI_{85} = phase change interval based on V_{85}, s; CI_{15} = phase change interval based on V_{15}, s; CI = phase change interval retained for use, s; Y = yellow interval retained for use, s; and AR = all-red clearance interval retained for use, s.

The purpose of the two phase change interval calculations in Steps 1 and 2 is to insure that the change interval duration is adequate for both the slow and fast driver. This sensitivity to slow speeds is particularly important when the clearance path is long, as is found at the SPUI, because the slow driver typically requires a longer clearance time than the fast driver.

If speed data are not available, the ITE technical committee suggests that the 85th percentile approach speed can be assumed to equal the posted speed limit. This committee also suggests that the 15th percentile approach speed can be assumed to be 16.1 km/h (10 mph) slower than the posted speed limit.

Change Interval Calculation for Left-Turn Movements. The ITE technical committee's methodology is not as precisely defined for left-turn movements. In particular, the committee did not specifically state whether both the 85th and the 15th percentile turn speeds need to be considered; however, it does state that the difference between these speeds is likely to be small. Therefore, to simplify the calculation, the change interval is computed for one speed only.

The approach speed used to compute the yellow warning interval represents a compromise speed. It is recognized that approaching left-turn drivers could be in a free flow situation and approaching at a high speed or, they could be in a moving queue approaching the intersection at a crawl speed. As a compromise solution, ITE committee recommends that the average of the 85th percentile approach speed V_{85} and the average left-turn execution speed V_e be used to determine the length of the yellow interval. The following calculation steps describe the procedure for calculating the change interval components for left-turn movements:

Step 1. $V_m = (V_{85} + V_e) / 2$
Step 2. $Y = Y(V_m)$
Step 3. $AR = AR(V_e)$

where, V_m = compromise approach speed, m/s.

<u>Clearance Interval Lengths for SPUIs</u>

The procedure described in the preceding section was used to determine the clearance interval requirements of through and left-turn movements typically found at SPUIs. The left-turn execution speed was computed using an equation developed by Bonneson (*7*) that relates left-turn speed to left-turn path radius. The form of this equation is:

$$V_e = 3.0 \quad x \quad R^{0.326} \tag{1}$$

where:
V_e = average left-turn execution speed, m/s; and
R = centerline radius of curvature of left-turn travel path, m.

The results of the analysis are shown in Figures 5 and 6. Figure 5 illustrates the clearance interval duration for through movements as a function of approach speed and clearance path length. The three thick trend lines correspond to 85th percentile approach speeds of 56, 64, and 72 km/h (35, 40, and 45 mph) used with the "ITE method" previously described. Clearance intervals increase significantly with clearance path length. In fact, a clearance interval of almost 11 s is needed when path length is 120 m (390 ft) and the approach speed is 56 km/h (35 mph).

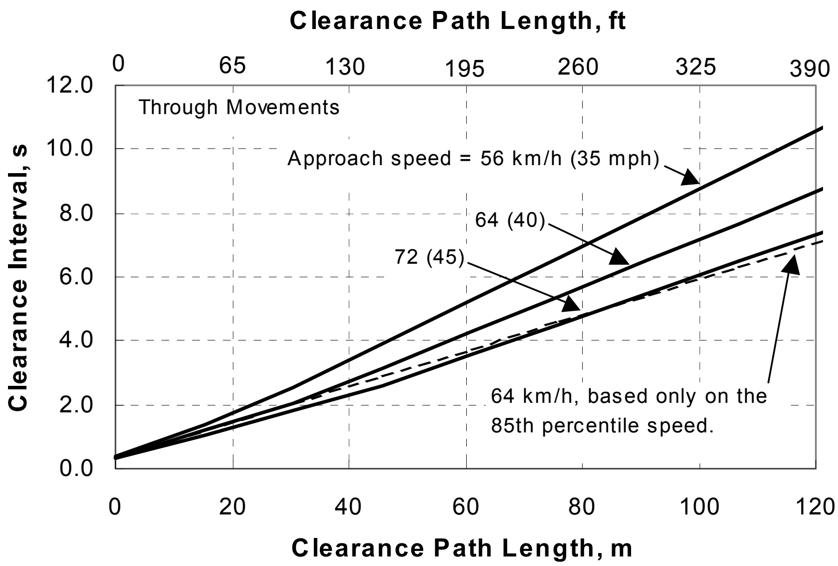


Figure 5. Clearance interval duration for through movements.

The dashed line shown in Figure 5 illustrates the clearance interval duration based on the direct application of Equations 1 and 2 with an 85th percentile speed of 64 km/h (40 mph). This trend line can be compared with the thick line labeled "64 km/h," which is based on the "ITE method." For path lengths in excess of about 30 m (100 ft), the clearance interval duration from the "ITE method" is longer than that obtained from the direct use of Equations 1 and 2. This trend confirms that the slow driver (e.g., the 15th percentile driver) needs more clearance time at SPUIs and other large intersections, relative to faster drivers. The difference between the two trend lines suggests that the slow driver needs about 25 percent more time than the fast driver to clear the conflict area.

The clearance interval needs of the left-turn driver are shown in Figure 6 as a function of clearance path length and left-turn radius. Trend lines for radii of 45, 60, and 75 m (150, 200, and 250 ft) are shown. For a given path length, left-turn drivers require longer clearance intervals than through drivers because the turn speed tends to be slower than the typical approach speed. Equation 3 predicts speeds of 38, 41, and 44 km/h (24, 25, 27 mph) for radii of 45, 60, and 75 m (150, 200, and 250 ft).

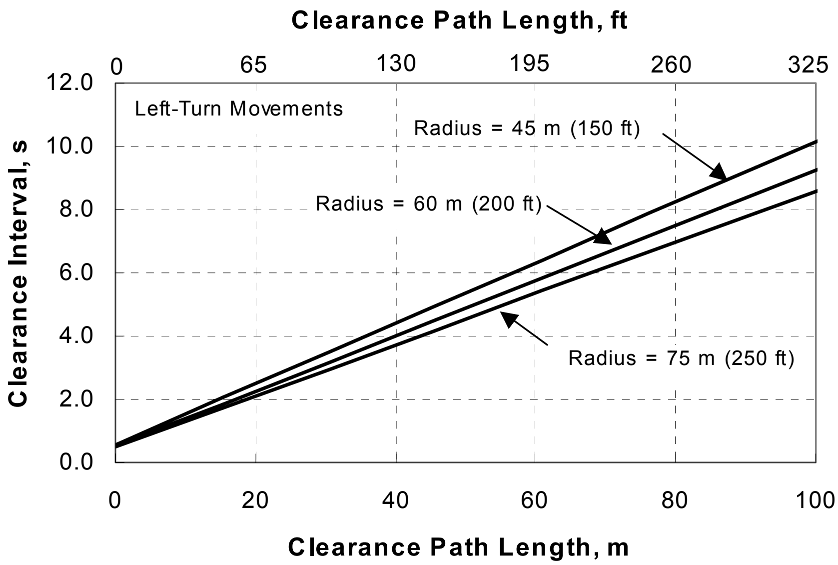


Figure 6. Clearance interval duration for left-turn movements.

Saturation Flow Rate and Start-Up Lost Time

The capacity of a SPUI or TUDI is highly dependent on the saturation flow rate and start-up lost time. In a review of the literature on the relative performance of both interchange types, Poppe *et al.* (*8*) found considerable disparity in the claims made by various researchers. Poppe *et al.* speculated that some of this disparity was due to uncertain knowledge about the operational characteristics of the SPUI, especially the saturation flow rate and lost time.

The most obvious difference between the two interchange types is the left-turn path radius. Leisch *et al.* (*6*) noted that the TUDI left-turn radii range from 15 to 23 m (50 to 75 ft). *Messer et al.* (*5*) reported that the majority of SPUI left-turn radii range from 45 to 107 m (150 to 350 ft). Kimber *et al.* (*10*) found that saturation flow rate at intersections increased with increasing turn radius. A similar effect was found at SPUIs by Bonneson (*11*) and by Hook and Upchurch (12). These findings are compared in Figure 7. It should be noted that Bonneson found that saturation flow rate also increased with increasing traffic pressure (as measured by the number of vehicles served per cycle). The line attributed to Bonneson in Figure 7 reflects a traffic pressure of 10 veh/ln/cycle.

The trends in Figure 7 provide strong evidence that saturation flow rate increases with left-turn path radius. Based on the radii noted in the previous paragraph, the saturation flow rate for TUDI left-turn movements should range from 1,900 to 1,950 pc/h/ln; those for the SPUI movements should range from 1,950 to 2,100 pc/h/ln.

The average saturation flow rates reported by three researchers are compared in Table 1. The saturation flow rates reported by Bonneson (*11*) were measured in Florida. Those reported by Hook and Upchurch (*12*) and by Poppe *et al.* (8) were measured in Arizona. The radii at the Florida SPUIs ranged from 50 to 100 m (160 to 330 ft) as did the radii at the Arizona SPUI/F.

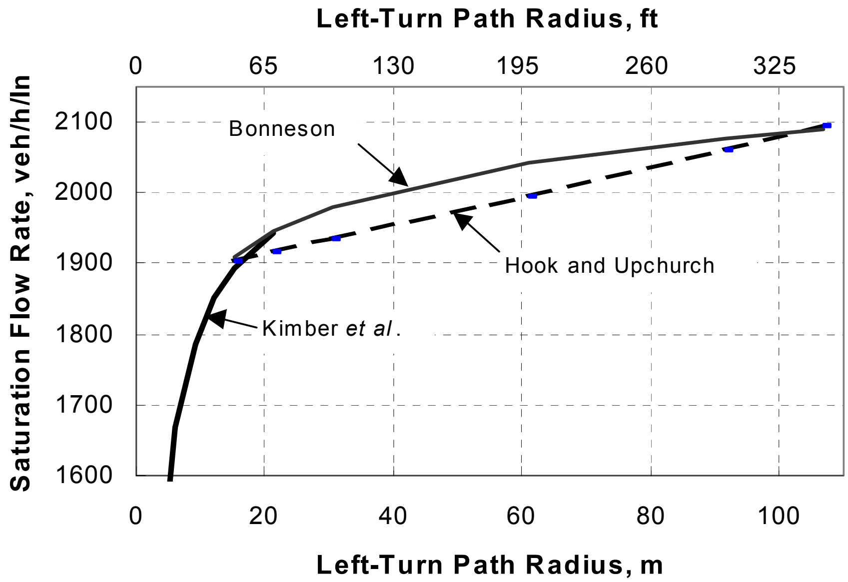


Figure 7. Saturation flow rate of left-turn movements as a function of radius.

Of note in Table 1 is the difference in SPUI saturation flow rates reported by Bonneson (*11*) and by Poppe *et al.* (8). This difference likely stems from the fact that Bonneson's data came from Florida and Poppe's data came from Arizona. It suggests that the Arizona drivers are more efficient in their use of the green indication. However, other factors such as measurement method or interchange size could also explain the differences noted.

Although the Florida and Arizona SPUI saturation flow rates differ, both sources indicate that the left-turn saturation flow rate at a SPUI is *larger* than the through movement rate by 5 to 10 percent. This trend is contrary to that found at most intersections. In fact, Chapter 16 of the *Highway Capacity Manual* (*13*) indicates that left-turn saturation flow rates are 5 to 8 percent *lower* than through movement rates. The data in Table 1 also suggest that saturation flow rates for through movements range from 1,750 to 2,000 pc/h/ln at both SPUIs and TUDIs, possibly dependent on location.

Finally, the saturation flow rates reported by Hook and Upchurch (*12*) and by Poppe *et al.* (*8*) are compared in the last column of Table 1. For each movement type, the saturation flow rate for the SPUI is larger than that for the TUDI. Specifically, the left-turn saturation flow rate for the SPUI is about 7 percent larger than that for the TUDI; the saturation flow rate for the through movement is about 2 percent larger.

The start-up lost times reported by Bonneson (*11*), Hook and Upchurch (*12*), and by Poppe *et al.* (8) are shown in Table 2. As noted for Table 1, the values reported by Bonneson are noticeably different that those reported by Hook and Upchurch and by Poppe. This difference is likely due to the number of queue positions considered in the computation of start-up lost time (although location effects may also be present). Bonneson based his estimate of start-up lost time on the first four queued vehicles. In contrast, the other two researchers based their estimate on the first three queue vehicles. The difference between the two methods implies that the fourth queued vehicle incurs about 0.6 s of lost time.

Table 1. Saturation flow rate comparison.

Movement	Interchange Type	Saturation Flow Rate, veh/h/ln [1]			
		Source			Comparison of Hook & Poppe
		Bonneson (*11*)	Hook *et al.* (*12*)	Poppe *et al.* (*8*) [2]	
Cross-Street Left	TUDI	—	1975	—	1975
	SPUI	1915	—	2110	2110
	% Difference	—	—	—	7%
Off-Ramp Left	TUDI	—	1956	—	1956
	SPUI	1935	—	2086	2086
	% Difference	—	—	—	7%
Cross-Street Through	TUDI	—	1948	—	1948
	SPUI	1739	—	1984	1984
	% Difference	—	—	—	2%
Number of interchanges studied:		3	7	3	
Queue positions used for estimate:		5th and higher	4th and higher	4th and higher	

Note:

1- Saturation flow rates were computed in accordance with procedures described in the *Highway Capacity Manual* (*13*) with the exception that the queue positions included in the average varied as noted in the table.

2- Saturation flow rates are based on a volume-weighted average of the reported mean headway.

The trends in Table 2 suggest that the off-ramp left-turn movement has the largest start-up lost time. This finding may be due to driver disorientation that takes place as the drivers transition from the freeway to the surface street environment. Driver disorientation appears to be the most extreme on the SPUI off-ramp. With the exception of the off-ramp left-turn movement, the differences in start-up lost time between the SPUI and TUDI are about 0.1 s or less which is negligible for practical purposes.

Table 2. Start-up lost time comparison.

Movement	Interchange Type	Start-Up Lost Time, s [1]			
		Source			Comparison of Hook & Poppe
		Bonneson (*11*)	Hook *et al.* (*12*)	Poppe *et al.* (*8*)	
Cross-Street Left	TUDI	—	1.41	—	1.41
	SPUI	2.50	—	1.30	1.30
	% Difference	—	—	—	-8%
Off-Ramp Left	TUDI	—	1.62	—	1.62
	SPUI	2.52	—	1.89	1.89
	% Difference	—	—	—	17%
Cross-Street Through	TUDI	—	1.56	—	1.56
	SPUI	2.09	—	1.65	1.65
	% Difference	—	—	—	6%
Number of interchanges studied:		3	7	3	
Queue positions used for estimate:		1st through 4th	1st through 3rd	1st through 3rd	

Note:

1 - Start-up lost time computed as summed headways for queue positions noted in table less an equivalent number of saturation headways.

Performance Comparisons

Single Point Urban Interchanges Without Frontage Roads

There is some uncertainty about the operational merits of the SPUI/n and TUDI. Some of the first publications on the topic suggested that the SPUI was more efficient than the TUDI under all traffic conditions, given similar physical sizes. Hawkes and Falini (*14*) claimed that the SPUI/n had twice the capacity of the TUDI because it was able to serve the left-turn movements concurrently and with one less signal phase. A similar claim was repeated by Brown and Walters (*15*). They reported that the SPUI had between 10 and 50 percent more capacity than the TUDI, with the larger amount being realized when the off-ramp volumes are balanced and high relative to the cross street volumes.

In response to the aforementioned claims, Leisch *et al.* (*6*) conducted some comparative analyses of SPUI/n and TUDI delays for a range of volume scenarios. They found that the TUDI was more efficient than the SPUI/n for most volume patterns.

More recently, Fowler (*16*) conducted an analysis of SPUI/n and TUDI operation using the same approach as used by Leisch *et al.* (*6*). Fowler was more methodical in developing the volume patterns such that a wide range of conditions were evaluated. Based on his analysis, he concluded that the SPUI/n had more capacity than the TUDI under most volume conditions. He suggested that the possibility that the TUDI would have more capacity would increase as volume pairs (e.g., both off-ramp left-turns, both cross street throughs, both cross-street left-turns) became more unbalanced.

Most recently, Garber and Smith (*3*) analyzed SPUI/n and TUDI operations for a range of volume patterns and levels. They found that the SPUI operated with lower delay than the TUDI, regardless of volume pattern. When traffic demands were relatively low, the two interchanges produced similar delays; however, when demands were high, delays at the TUDI were much higher than those as the SPUI/n. The relationship between delay and total interchange volume found by Garber and Smith is shown in Figure 8.

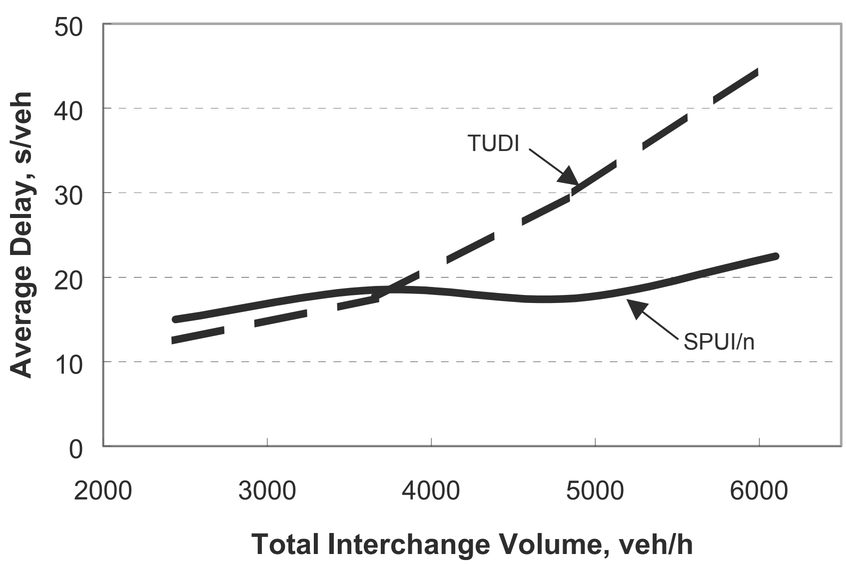


Figure 8. Delays associated with the *SPUI/n* and *the TUDI as a function of volume.*

Single Point Urban Interchanges with Frontage Roads

An article by Leisch *et al.* (*6*) represents the only published information describing the operational performance of the SPUI/F relative to the TUDI. Leisch *et al.* conducted two analyses, in one analysis both interchanges had the same number of lanes and in the other both interchanges had the same cross section width. The second analysis was conducted because it was recognized that the TUDI requires a wider cross section than the SPUI/F when both have the same number of approach lanes. For the second analysis, one through lane was eliminated from each TUDI approach.

When both interchanges had the same number of lanes, Leisch *et al.* (*6*) found that the TUDI had 30 percent lower overall delay. When both interchanges had the same cross section width, they found that the TUDI had 17 percent lower overall delay. From this analysis, they concluded that the TUDI was the "obvious choice" and that the SPUI/F is "an impractical design" for frontage road situations.

Bonneson (*17*) examined the SPUI with and without frontage roads. He found that the SPUI/F increased delays from 20 to 80 percent, relative to a similarly sized SPUI without frontage roads. The percentage increase was larger for larger volume-to-capacity ratios. This increase was attributed to: (1) an additional 10 s of total end lost time that resulted from the SPUI/F's larger conflict area and (2) the SPUI/F's need for an additional signal phase.

In a recent study, Garber and Smith (*3*) conducted a survey of 49 traffic engineers throughout the U.S. The information they obtained indicated that there is strong belief that motorist delay is significantly higher at the SPUI/F, relative to the TUDI. Based on this finding, they concluded that the TUDI is the "preferred" choice when a frontage road system exists.

Pedestrian Considerations

At the SPUI/F, pedestrians crossing the cross road can easily be accommodated by serving them concurrently with the adjacent frontage road phase. However, the excessive width of the cross road in the vicinity of the SPUI/F generally results in an excessively long pedestrian crossing phases. Although the pedestrian phase may be actuated and not called each cycle, when it is called it can be disruptive to traffic progression for several cycles thereafter. In addition, the long pedestrian phase tends to greatly exceed that needed for the frontage road phase. As a result, pedestrian service introduces significant motorist delay at SPUI/Fs. In fact, Messer *et al.* (*5*) report that only one of seven SPUI/Fs that they observed had pedestrian service across the cross road.

SAFETY ELEMENTS

Overview

The objective of this section is to document a critical review of the literature on the safety of the SPUI/F. There is limited safety-related data for the SPUI/F, primarily because of the small number of them across the country. For that reason safety-related literature for the more commonly found three-phase SPUI (i.e., SPUI/n) is presented. References *1, 2, 3, 4,* and *5* provide a thorough discussion of the design and operation of the SPUI/n.

Safety of SPUI and TUDI

There is no generally accepted consensus among traffic engineers about the relative safety of the two interchange types. There have been articles and publications with varying degrees of analysis and/or presentation of methodology. Leisch, *et al.* (6) opined that the potential for increased accidents is present at SPUIs because of the large, uncontrolled open pavement area and the opposing left turns. In a nationwide survey of engineers, five of twenty-nine respondents identified a belief that the SPUI is a safer design than the TUDI (*3*).

Accident Data

Cheng (*18*) reported a lower accident rate at SPUI/ns than at TUDIs in a study comparing crash data for three of each interchange type. The objectives of the research were to compare the safety experience of the SPUI/n as compared to the TUDI and to determine if there were any predominant factors involved in SPUI/n accidents. The lower SPUI/n accident rate was attributed to the fewer number of conflict points with the SPUI/n as compared to the TUDI. The predominant accident type for the SPUI/n was rear end collisions at the off ramp, which accounted for 46% of total collisions. There were no left turn collisions reported in the total of 39 SPUI/n crashes, leading to the conclusion that the protected left-turn phases perform well from a safety standpoint. The accidents at the SPUI/n appear to be less severe than those at the TUDI, with 91% of them being property damage only crashes as compared to 67% for the TUDI. Most of the SPUI/n crashes occurred in daytime (87%), on a clear or cloudy day (87%) and under dry road surface conditions (74%), leading to the conclusion that weather does not present any unusual problem for the SPUI/n design. Cheng also concluded that lack of familiarity with the SPUI/n design is not a major factor in accident occurrence. Other factors such as lighting, older driver population, pedestrian, and trucks also did not present any major problems according to Cheng's research.

Cheng's findings that the rear-end collision is the predominant crash type is consistent with Bonneson's findings (*2*) that the prevalent crash type at a SPUI/n in Clearwater, Florida was the rear end collision.

Messer, *et al.* (*5*) presented accident rates at five SPUI/ns, which ranged from 0.64 to 2.70 accidents per million entering vehicles. One of these locations had a considerably higher accident rate than the other four, with the predominant accident type being a rear end collision on the off ramps. At this location the left and right turn lanes were not exclusive at the two-lane off ramp, resulting in a high degree of interaction between the left and right turns. Additionally, a relatively close downstream intersection caused frequent spillback, which may have been unexpected to the off-ramp drivers. Messer also noted that a frequent conflict was between the clearing and entering vehicles in successive phases. This was attributed to the extended use of the yellow interval by clearing vehicles, suggesting the need for an additional all red change interval. In spite of these observed problems, Messer concluded that a modern SPUI/n is "as safe as a signalized at-grade intersection or TUDI interchange operating at the same volume levels". The accident rates at the SPUI/ns were compared to the accident rate at a typical signalized at-grade intersection (with an accident rate of about 1.2 accidents per million entering vehicles) with the conclusion that the SPUI/n design does not lead to a higher number of accidents.

In an accident study of eight SPUI/ns and five TUDIs, Garber *(3)* concluded that there is no significant difference between SPUI/n and TUDI accident rates (including total, injury and property damage only). He further concluded that a higher proportion of SPUI/n crashes occurred on the off ramps and on ramps as compared to those of the TUDI. Furthermore, he concluded that the proportion of accidents in the center of the intersection is greater at TUDIs.

Abbey, et al (*19*) concluded that accident rates and severity for the SPUI/n can be expected to be less than for the TUDI based on review of accident records of both the SPUI/n and TUDI. He concludes that the likely reason for this safety advantage is the fewer conflict points resulting from the single intersection.

Conflict Studies

Although traffic accident records provide the most direct measure of safety for a roadway location, adequate data may not be available for analysis. At times a more rapid approach may be desired. For this reason the traffic conflicts technique (TCT) has been developed. According to Glauz, *et al.* (*20*), "a traffic conflict is a traffic event involving the interaction of two vehicles where one or both drivers may have to take evasive action to avoid a collision."

Messer *et al.* (*5*) noted the frequent conflict between clearing and entering vehicles of successive phases, resulting from the extended use of the yellow interval by clearing vehicles. The most often observed such conflict was between the clearing off ramp and entering cross-road left turning vehicles due to the location of the conflict point and the duration of the clearance interval. He noted that this results in a long clearance interval coupled with a short entering distance, thereby creating the longest SPUI time separation requirement in the SPUI signal phase sequence.

Messer also noted that the conflict between off-ramp right-turn and cross-road through traffic required "the right turning driver to monitor (over the left-shoulder) two different sources of conflicting traffic, to negotiate a curve to the right, and to monitor traffic conditions downstream on the cross road (directly ahead)." These complications likely contributed to the conflicts observed for the movements involved.

Messer observed a frequent left turn erratic maneuver when drivers were observed turning left from the cross road through lane rather than from the left turn lane. He reported that these left turning drivers would stay in the inside through lane while traveling through the interchange and then make the turn across the opposing through traffic. It was hypothesized that these drivers may be confused by the SPUI design and were behaving as they would in a more typical diamond configuration. He noted that when frontage roads are present (e.g. a SPUI/F) "even the off-ramp left turns can and sometimes are made two ways by motorists".

Another unusual maneuver observed by Messer (*5*) was the occurrence of drivers stopping well beyond the stop line. It was presumed that the drivers may be unable to identify conflicting traffic streams because of the SPUI's unusual design and large conflict area. Additionally, the presence of signal heads at most overpass SPUIs mounted on the near side bridge face may lead to the drivers' incorrect judgments about the location of the conflict area and the appropriate stopping point.

Pedestrian Considerations

The excessive width of the cross road in the vicinity of the SPUI/n generally results in an excessively long pedestrian crossing phases. This is complicated by the fact that there is no vehicle phase which runs concurrently with the pedestrian phase crossing the cross road. As a result, pedestrian service introduces significant motorist delay at SPUIs to the extent that most agencies do not provide pedestrian crosswalks nor pedestrian signal heads in an attempt to discourage pedestrians from crossing the cross road at the SPUI/n (5).

At the SPUI/F, pedestrians crossing the cross road can easily be accommodated by serving them concurrently with the adjacent frontage road phase. However, the excessive width of the cross road in the vicinity of the SPUI/F generally results in an excessively long pedestrian crossing phases. Although the pedestrian phase may be actuated and not called each cycle, when it is called it can be disruptive to traffic progression for several cycles thereafter. In addition, the long pedestrian phase tends to greatly exceed that needed for the frontage road phase. As a result, pedestrian service introduces significant motorist delay at SPUI/Fs. In fact, Messer *et al.* (*5*) report that only one of seven SPUI/Fs that they observed had pedestrian service across the cross road.

Although pedestrians crossing the frontage roads in the SPUI/n have an associated vehicular phase and a more narrow crossing distance, there still may be pedestrian safety issues. Messer *et al.* (*5*) report two potential problems for pedestrians crossing the on-ramp and off-ramp pair. The first noted is the fact that the right turning movements to and from the cross road are not signalized, because to do so would reduce the capacity of the SPUI. Secondly, the width of the SPUI is such that it cannot be crossed in one signal cycle. This is further complicated with the SPUI/F because there are two signalized roadways on each side (left turns and through movements) when crossing the ramps for a total of four signalized roadways to cross.

SURVEY OF PRACTITIONERS

This section describes the findings from a survey of engineers at selected state departments of transportation. The objective of this survey was to learn of these agency's methods and procedures for selecting, designing, and operating the SPUI/F. The information sought through the survey pertained to the following topics:

- Availability of formal guidelines for interchange selection, design, or operations.
- Availability of safety study reports for existing SPUI/Fs.
- Remedial measures taken to alleviate operational or safety problems.
- Information on cost and right-of-way issues and concerns, as related to interchange selection.
- Location of other SPUI/Fs that exist in surrounding states.
- Control equipment and special control features used (e.g., variable change interval).
- Clearance interval timing procedures.
- Location and placement of signal heads and pedestrian crosswalks.

Information on the topics listed above was solicited through the use of a questionnaire. Due to the broad range of topics, two separate questionnaires were developed. One questionnaire addressed the traffic operations and control topics; a second questionnaire addressed the interchange planning and design topics. The number questions on each questionnaire was kept brief to insure a high response rate. A copy of both questionnaires is provided in Appendix A.

Distribution of Survey

Eight questionnaires were mailed to engineers at four state DOTs. One Traffic Operations questionnaire was mailed to the State Traffic Engineer at each state DOT. Similarly, one Planning & Design questionnaire was mailed to the State Design Engineer at each DOT.

Each questionnaire was tailored for the person to whom it was sent. Specifically, each questionnaire listed one or two SPUI/F′s located in the state to which it was mailed. This refinement was intended to eliminate any confusion over the type of interchange being addressed in this research as well as to provide the respondent with a specific interchange on which to focus his or her answers. Seven SPUI/F′s were referenced in these questionnaires, they are listed in Table 3.

Table 3. Location of several single point urban interchanges with frontage roads.

City, State	Intersecting Streets	Data[1]	Plan[2]
Huntsville, Alabama	1. US 231 & US 72 (University Ave.)	a	
	2. US 231 & Governors Drive		
Largo, Florida	3. U.S. 19 and SR 686 (East Bay Drive)	a	a
	4. U.S. 19 and SR 688		
Atlanta, Georgia	5. Peachtree Industrial Blvd. & Winters Chapel Road	a	a
	6. Peachtree Industrial Blvd. & Jimmy Carter Blvd.	a	a
Wichita, Kansas	7. U.S. 54 (Kellog) & West Street	a	a

Notes:
1 - a indicates data describing the physical attributes of the SPUI/F are provided in Appendix B.
2 - a indicates a plan view of the horizontal geometry of the SPUI/F is provided in Appendix B.

A total of five completed questionnaires were returned. Of these five, four were Traffic Operations questionnaires and one was a Planning & Design questionnaire. This represents a 100 percent response rate for the former questionnaire category and a 25 percent response rate for the latter category. Overall, 62 percent of the questionnaires were completed and returned. The responses recorded on the completed questionnaires are summarized in the remainder of this chapter.

As noted in the last two columns of Table 3, information describing several of the SPUI/Fs is provided in Appendix B. Based on a review of this information, the following generalizations are offered: (1) all of the existing SPUI/Fs have the major-street traffic lanes elevated such that they are above the cross street-ramp intersection, (2) exclusive u-turn lanes are very commonly used, (3) there is a tendency to provide two lanes (each direction) for the through movements and two lanes for the left-turn movements on both the cross street and the ramps, and (4) the distance between the opposing stop lines on the cross street varies from 64 to 85 m (210 to 280 ft).

Few of the aforementioned generalized attributes are shared by the SPUI/Fs in Phoenix. In general, the Phoenix SPUI/Fs have (1) the cross street-ramp intersection elevated above the major route, (2) no exclusive u-turn lanes, (3) three lanes (each direction) for the through movements on both the cross street and the ramps, and (4) a distance between opposing stop lines that varies from 98 to 128 m (320 to 420 ft).

Summary of Responses to Traffic Operations Questionnaire

The Traffic Operations questionnaire had questions that addressed: (1) the overall performance of the SPUI/F, relative to other interchange forms; (2) the all-red clearance interval; and (3) typical locations of the traffic control devices. The responses to each of these areas of inquiry are summarized in the next three sections.

Traffic and Design Element Ratings

The respondents were asked to rate several key traffic and design elements of the SPUI/F's in their jurisdiction. This rating was based on a subjective scale of 1 to 5, with 5 representing "excellent" conditions. The basis for comparison was the tight urban diamond interchange (TUDI). The results of this comparison are shown in columns 4 through 8 of Table 4.

Table 4. Traffic control features and comparative ratings of the SPUI/F.

Traffic Control Features			**Traffic and Design Element Rating [1] (1 = poor, 5 = excellent)**				
No.[2]	**Actuated or Pretimed?**	**NEMA or 170/2070?**	**Traffic Capacity**	**Arterial Coordination**	**Traffic Safety**	**R.O.W. Requirements**	**Construction Cost**
1	Actuated	170	2	2	3	4	3
2	Actuated	170	3	3	3	4	3
3	Actuated	NEMA	4	5	4	5	3
4	Actuated	NEMA	3	5	4	5	3
5	Actuated	170	4	4	3	5	4
6	Actuated	NEMA	4	4	3	5	4
7	Actuated	170	3	3	4	3	?
		Average Rating:	**3.3**	**3.7**	**3.4**	**4.4**	**3.3**
		Reported by Garber & Smith (*3*):	4.0	3.9	3.4	—	2.9

Note:
1- Rating is based on how well the SPUI/F met the expectations for each traffic or design element, as compared with a similar TUDI.
2- SPUI/F numbers correspond to the numbers assigned to each SPUI/F in Table 3.

The average ratings shown in the second to last row of Table 4 provide some indication of the overall opinion of the respondents. For each element considered, the average is above 3.0 which indicates that the respondents believe the SPUI/F is believed to be a better choice than a TUDI at the subject location. The SPUI/F gets its highest rating in the "ROW Requirements" category which indicates that the SPUI/F's best attribute is its ability to minimize right-of-way requirements (presumably along the major street).

The last row of Table 4 contains the findings from a survey of state DOT engineers conducted by Garber and Smith (*3*). Their questionnaire included a request to rate the Traffic Capacity, Arterial Coordination, Traffic Safety, and Construction Cost elements of the SPUI/n relative to the TUDI. The wording and rating scale used by Garber and Smith was identical to that used on the questionnaire shown in Appendix A. A comparison of the averages found in this research with those reported by Garber and Smith lead to the following conclusions:

- When comparing the SPUI/F with the TUDI and the SPUI/n with the TUDI, both SPUI variations are believed to have slightly more capacity than the TUDI. Of the two SPUI forms, the SPUI/F is believed to offer less of a capacity increase than the SPUI/n.

- When comparing the SPUI/F with the TUDI and the SPUI/n with the TUDI, both SPUI variations are believed to be somewhat more conducive to coordination than the TUDI.

- When comparing the SPUI/F with the TUDI and the SPUI/n with the TUDI, both SPUI variations are believed to be slightly safer than the TUDI.

- When comparing the SPUI/F with the TUDI and the SPUI/n with the TUDI, the SPUI/F is believed to have a lower construction cost than the TUDI while the SPUI/n is believed to have a slightly higher cost than the TUDI.

All-Red Clearance Interval

A series of questions were asked about the methods used to time the all-red clearance interval for each of the SPUI/F signal phases. The responses indicate that all four agencies base the all-red interval duration on the clearing vehicle's travel time through the interchange conflict area. As such, the all-red interval is dependent on the length of the clearance path and the speed of the clearing vehicle.

Three of the four agencies measure the clearance path length from the near-side stop line to the far edge of the farthest conflicting traffic lane, as measured along the vehicle path and based on all possible conflicting movements. One agency measures the clearance path length slightly different than the other agencies. This agency measures from the stop line to the farthest conflicting traffic lane associated with the *next* signal phase. By using this technique, this agency can reduce the all-red interval duration; however, they must set the controller so it never skips a phase (even if there is no demand for this phase).

All respondents indicated that no special control features were used to control the interchange. Specifically, they indicated that features that would vary the duration of the all-red clearance interval (e.g., based on knowledge of the next phase or the presence of a late-clearing vehicle) were not used.

Traffic Control Device Locations

The questionnaire included an idealized plan view of a SPUI/F and asked the respondents to identify the location of crosswalks, signal heads, and pedestrian push buttons. The findings from this request are graphically recreated in Figure 9.

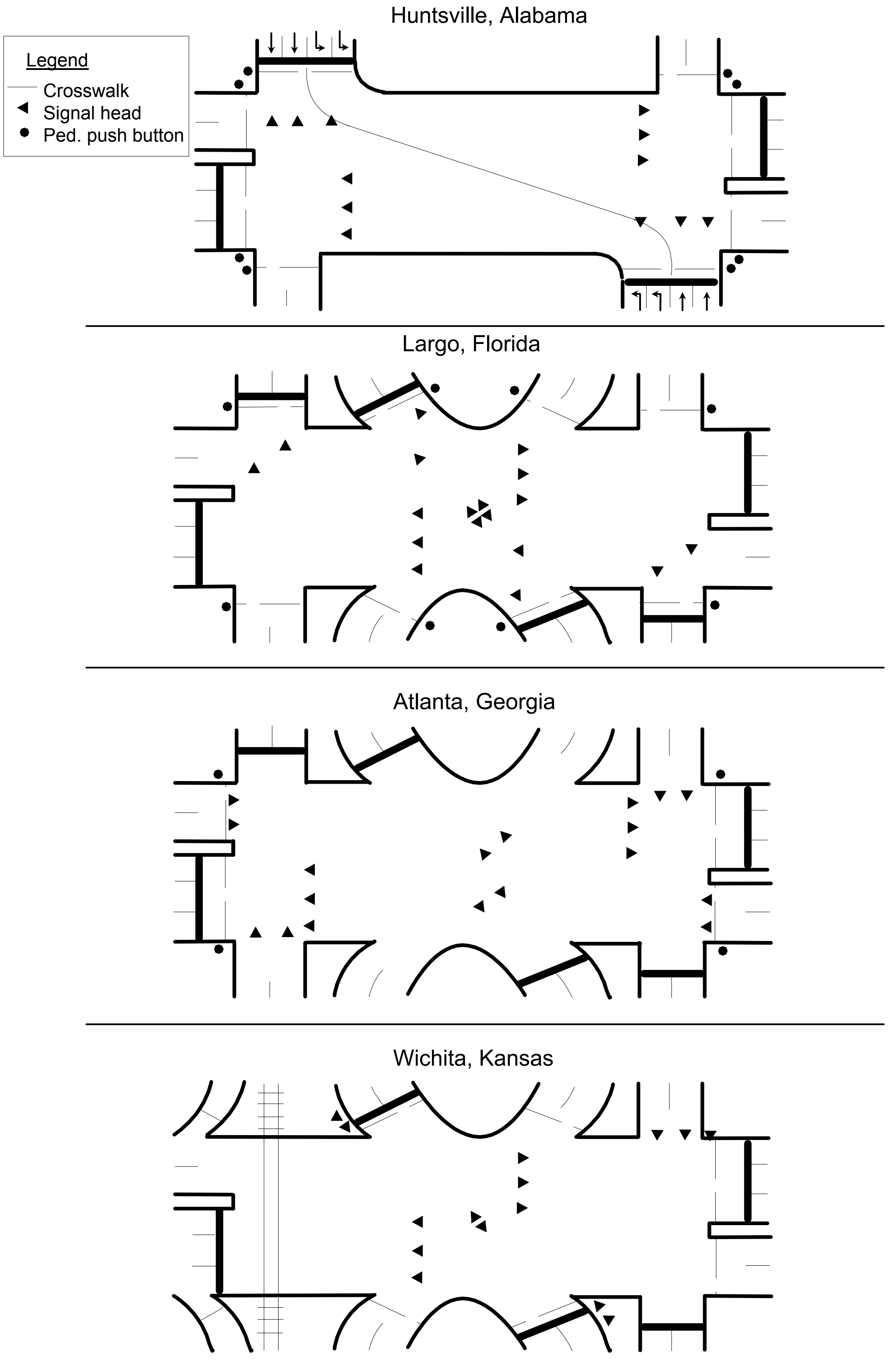


Figure 9. Traffic control device locations at four SPUI/F.

An examination of the information in Figure 9 did not reveal evidence of consistent trends in terms of control device location. Some trends that were noted regarding pedestrian crossings include: (1) pedestrian crossings across the ramps were provided by three of the four responding agencies, and (2) pedestrian crossings across the cross street were provided by three of four agencies. Information sent by one agency indicated that the pedestrian call buttons were set to call the signal phase corresponding to the adjacent through movement. One agency indicated that providing control for "pedestrian traffic through the SPUI has been a problem."

Finally, the questionnaire inquired about the availability of agency guidelines for SPUI/F operation or existing safety study reports. All respondents indicated that no formal guidelines or reports were available.

Summary of Responses to Planning & Design Questionnaire

Only one of the four State Design Engineers contacted completed and returned the Planning & Design questionnaire. This respondent indicated that he did not know of any other SPUI/Fs. He also indicated that right-of-way cost, capacity benefits , and lower construction costs of the SPUI/F (relative to the TUDI) were the primary reasons for its selection at the subject location. The respondent also indicated that his agency did not have formal guidelines for interchange selection or design.

CHAPTER 2
INTERCHANGE SITE SELECTION

INTRODUCTION

The scope of work contained in the ADOT Request for Proposal specified that six diamond interchanges and four SPUI/F interchanges be selected for a safety comparison. Presumably, there were only four SPUI/F interchanges operational when this scope was prepared. Additionally, it called for a side-by-side comparison of the SPUI/F at I-17 & Dunlap with the tight diamond interchange at I-17 & Indian School. The project team recommended that five of each type be selected in order to realize a similar sample size of data for comparison. It was also recommended that the operational comparison be extended to all 10 interchanges, rather than only to the two previously mentioned, in order to compare a wider range of operating characteristics.

INTERCHANGE SELECTION CRITERIA

It was desired that the sites selected for this study be reasonably similar in terms of their physical size, number of traffic lanes, traffic demands, age and location relative to other signalized intersections. They should have “typical” (or representative) geometrics and traffic demand patterns. Finally, they should have available geometric, crash and cost data for appropriate comparisons. In order to compare similar conditions, the TUDIs were only considered as a study site when they had through traffic on the frontage road. Although it was desired that the newest interchanges of each type be studied, older interchanges offered better crash history. Also, it was desired that both underpass and overpass interchanges be studied. The selected TUDIs should include both 3-phase and 4-phase operation.

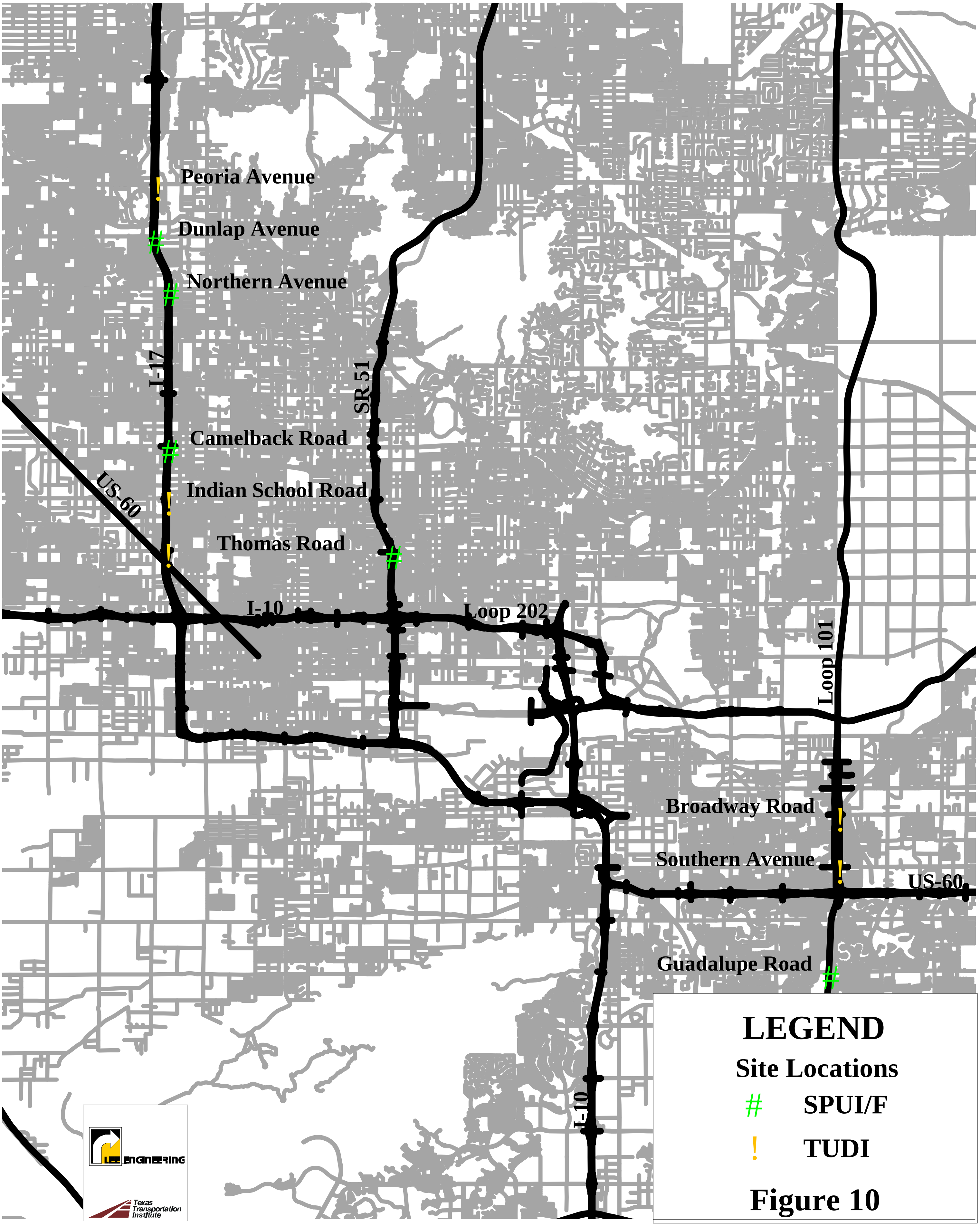
Another possible variable is the sun in the morning and evening affecting the east – west approaches. Since the interchanges predetermined for the study have east – west cross streets, it was desirable that all study interchanges be oriented in that manner to remove that possible variable from the study.

STUDY SITES

The following describes the five interchanges of each type with a brief description of each. The interchange locations are shown in Figure 10.

1. I-17 & Dunlap (SPUI/F). This interchange was one of two identified by the sponsor in the Request for Proposal (RFP) for study. It represents one of the newer SPUI/F designs. This interchange is an underpass, i.e. the freeway goes under the cross street.

2. I-17 & Indian School (TUDI). This is the other interchange identified by the sponsor for study and side-by-side comparison with the SPUI/F at I-17 & Dunlap. It represents one of the newer tight diamond interchange designs. It is an underpass operating in a four-phase manner.



Peoria Avenue
Dunlap Avenue
Northern Avenue
I-17
SR 51
Camelback Road
Indian School Road
US-60
Thomas Road
I-10
Loop 202
Loop 101
Broadway Road
Southern Avenue
US-60
Guadalupe Road
I-10
LEE ENGINEERING
Texas Transportation Institute
LEGEND
Site Locations
SPUI/F
! TUDI

Figure 10

3. SR 51 & Thomas (SPUI/F). This is the first SPUI/F constructed in Arizona which gives it the longest period of crash data. It is an overpass, i.e. the freeway goes over the cross street.

4. I-17 & Peoria (TUDI). This is one of the older interchanges to be studied. It is an overpass design, which permits a comparison with the Thomas Road / SR 51 SPUI/F overpass. A member of the research team had collected a considerable amount of data at this interchange in a previous research project. It has a single left-turn lane from the cross street to the frontage road.

5. I-17 & Thomas (TUDI). This underpass TUDI is operating in a four-phase manner. It also is an older interchange than the other three TUDIs being studied. It also has a single left-turn lane from the cross street to the frontage road.

6. I-17 & Northern (SPUI/F). This underpass interchange is one of the newer SPUI/F designs.

7. Loop 101 & Southern (TUDI). This underpass interchange is operating in a three-phase manner and includes dual left turns from the cross street.

8. Loop 101 & Broadway (TUDI). This underpass interchange is also a three-phase with dual left turns from the cross street.

9. I-17 & Camelback (SPUI/F). This is one of the newer underpass interchanges. It also has some decorative pedestrian / bridge railings which could affect driver behavior due to visibility issues

10. Loop 101 & Guadalupe (SPUI/F). Although this interchange was still under construction during the data collection phase of this research, the TAC recommended its inclusion in the study due to its width (stop line to stop line) and different traffic control methods. This includes programmed visibility signal heads in an attempt to reduce driver confusion. It also contains a solid signal bridge as opposed to the tubular signal structures found on I-17 SPUI/F interchanges.

This selection of interchanges provided a variety of SPUI/F and TUDI operations. Both types are represented by both overpass and underpass designs. There are TUDIs included in the study operating both as three-phase and four-phase. Inclusion of older interchanges provided better crash history. Inclusion of newer interchanges reflected any recently improved geometry. All study interchanges have continuous frontage roads. Pertinent information about the ten interchanges is shown in Table 5. At all of the locations the freeway is listed first and has a north/south orientation.

Table 5. Information for the Ten Interchanges

Interchange	Type	Geometry	Width[1]	Remarks
I-17/Dunlap Ave.	SPUI/F	Underpass	90 m (300’)	One of newer SPUI/F designs
I-17/Indian School Rd.	TUDI	Underpass	80 m (260’)	4 phase, one of newer tight diamond interchange designs
SR 51/Thomas Rd.	SPUI/F	Overpass	70 m (225’)	Driveway to shopping center located near NB frontage road merge area
I-17/Peoria Ave.	TUDI	Overpass	110 m (360’)	Single left-turn lane from cross street to frontage road
I-17/Thomas Rd.	TUDI	Underpass	95 m (310’)	Single left-turn lane from cross street to frontage road
I-17/Northern Ave.	SPUI/F	Underpass	88 m (290’)	One of newer SPUI/F designs
Loop 101/Southern Ave.	TUDI	Underpass	140 m (465’)	3 phase, dual left turns from the cross street
Loop 101/Broadway Rd.	TUDI	Underpass	150 m (490’)	3 phase, dual left turns from the cross street
I-17/Camelback Rd.	SPUI/F	Underpass	90 m (300’)	This is one of the newer underpass interchanges
Loop 101/Guadalupe Rd.	SPUI/F	Underpass	107 m (350’)	SB on-ramp closed construction not completed at time of study

[1]distance measured between center of frontage roads

CHAPTER 3
TRAFFIC SAFETY COMPARISON OF
THE SINGLE POINT URBAN INTERCHANGE WITH FRONTAGE ROADS
AND THE TIGHT URBAN DIAMOND INTERCHANGE

INTRODUCTION

Among traffic engineers, there tends to be some uncertainty about the relative safety merits of the SPUI/n and TUDI. Some of the first publications on the topic suggested that the SPUI/n had a lower accident rate than the TUDI. Others opined that the TUDI was safer. More recent studies tend to support that while there is no significant difference in severity distribution or in accident rates between the two types, there is a difference in the prevalent type of crash. The findings from these recent studies suggest that the SPUI/n has a greater proportion of on-ramp and off-ramp accidents and that the TUDI has a greater proportion of accidents occurring in the center of the signalized intersection.

Because of the limited number of SPUI/Fs, no studies were found comparing the TUDI accident experience with the SPUI/F. This chapter describes the findings from a crash analysis of the study interchanges, both SPUI/F and TUDI. It includes both an overall crash rate analysis and a comparison of the crash types that are more prevalent at each type. Additionally, the results of a conflict analysis are described and presented.

DATA COLLECTION

Locations

Field studies were conducted on Mondays through Thursdays, between July 24 and September 7, 2000, at ten study interchanges (5 TUDIs and 5 SPUI/Fs). The locations are shown in Figure 10 with geometry information noted in Table 5.

Video Recording Stations

Video recording stations were assembled at each of the ten study interchanges. The videotapes were used to extract traffic count data, and allow for a review of any traffic occurrences during the study periods. Portable construction scaffolding was erected, and a tripod-mounted camcorder was placed on top of the scaffolding. The camcorder recorded traffic conditions from an elevation of about 3 meters (10 ft), which allowed for viewing of traffic for a long distance and reduced the likelihood of camera views being interfered by large trucks, buildings, or other obstructions. Because of the hot temperatures during the data collection periods, it was necessary that both the cameras and data collector be shaded (see Figure 11).

Study Periods

Traffic conflict data was collected for a duration of two hours for each of three different periods to collect various traffic data:

Figure 11. Typical video recording station.

PERIOD 1 (Mid-morning study): During the mid-morning traffic studies, the video cameras were deployed on the four external study approaches (A, B, C, D). Four team members conducted conflict studies (one member per approach). Period 1 data collection generally started at 7 a.m.

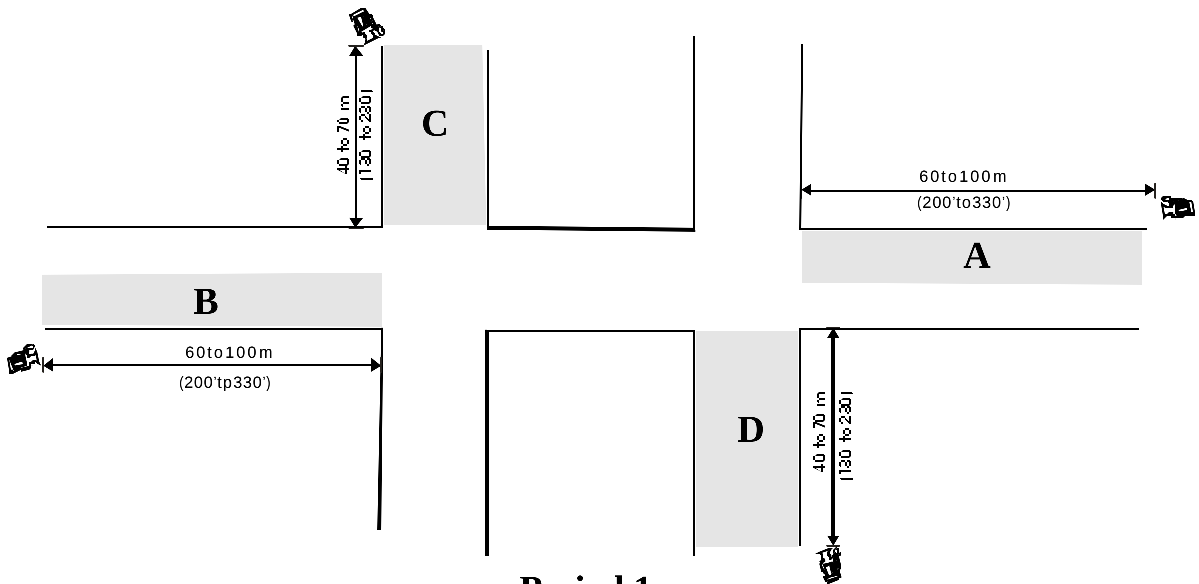
PERIOD 2 (Late-morning study): During the late-morning conflict studies, two team members and four video cameras were used to conduct conflict studies. At both interchange types, both directions of travel in the internal study zone and both on-ramp departure legs were studied. Period 2 data collection generally started at 10:30 a.m. or 11 a.m.

PERIOD 3 (Late-afternoon study): The late-afternoon studies were conducted in the same manner as that of the late-morning studies (see PERIOD 2). Period 3 data collection generally started at 4 p.m.

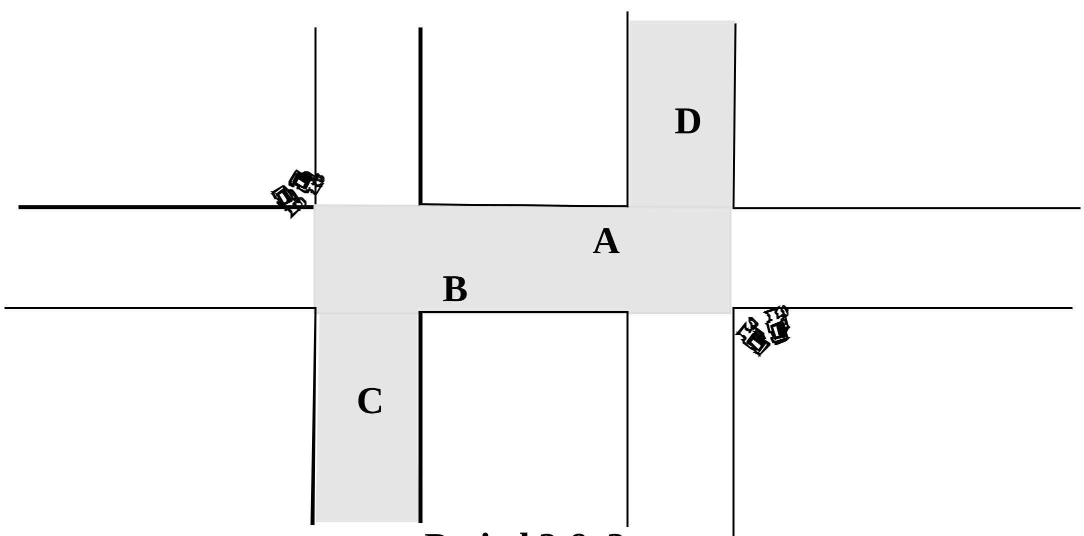
The study areas for each of the three periods are shown in Figure 12 (for TUDIs) and Figure 13 (for SPUI/F). At the conclusion of the study, turning movement counts were extracted from the videotapes for each of the three study periods (a total of 6 hours per interchange). These counts were used in computing the average daily traffic (ADTs), conflict rates, and accident rates at each interchange.

Average Daily Traffic Volumes

Average daily traffic (ADT) volumes were obtained in the following manner:



Period 1



Period 2 & 3

Legend

VideoCamcorder

StudyArea
(Designated A, B, C, D)

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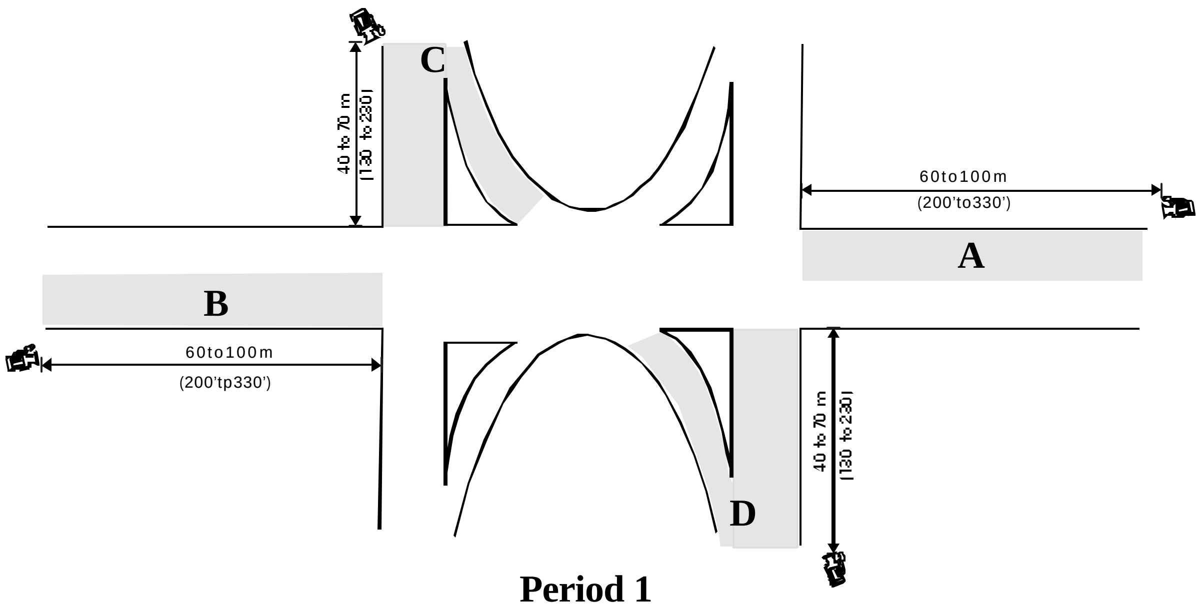
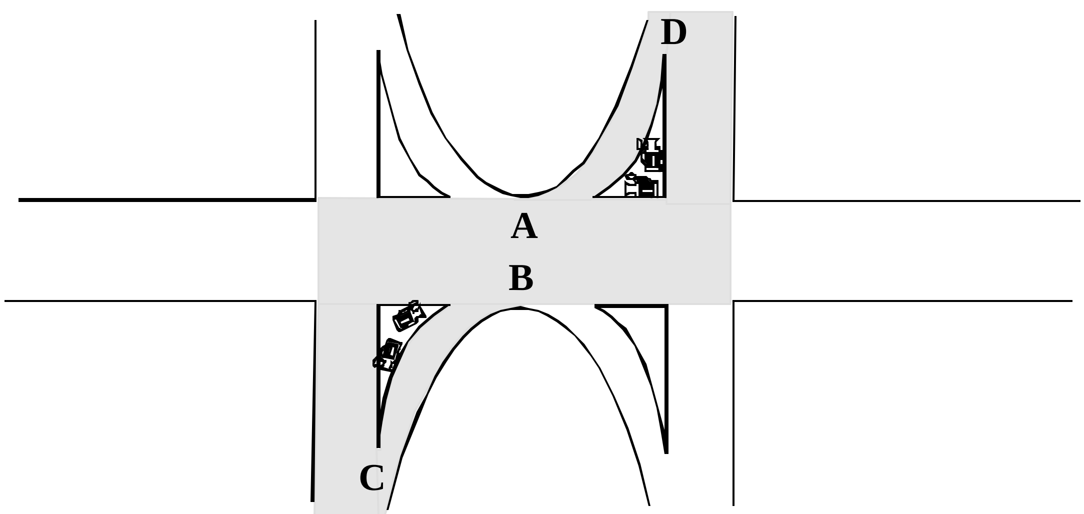
Video Recording Station Locations
TUDI Configuration

Figure 12



Period 1



Period 2 & 3

Legend

VideoCamcorder

StudyArea
(Designated A, B, C, D)

Nottoscale

LEE ENGINEERING

Video Recording Station Locations
SPUI/F Configuration

Figure 13

1. The most recent ADTs for the interchange cross streets were obtained from City of Phoenix, City of Tempe, and City of Mesa. This data consisted of counts from 1998, 1999, and 2000. Where applicable, a 3% yearly growth factor was used to obtain year 2000 ADTs.

2. The ratio of 6-hour volumes (collected by video from turning movement counts) to 2000 cross street ADTs was calculated.

3. The ratio presented in (2) was inversely applied to the 6-hour volumes on the frontage roads to obtain 2000 ADTs for the frontage roads. These frontage road ADTs were then used as a basis for estimating 1999, 1998, and 1997 ADTs by using a 3% annual reduction.

4. A 3% annual reduction was also applied to the cross street ADTs that were obtained from various agencies in order to estimate 1999,1998, and 1997 ADTs where applicable.

An exception to the procedures noted above was at SR 51/Thomas Road, where 24-hour counts were collected in October 2000 for all approaches. Additionally, due to recent construction of Loop 101, traffic growth on Broadway Road and Southern Avenue has been inconsistent compared to that of the interchanges located on I-17 and SR 51. Therefore, more specific ADTs were obtained for those interchanges with respect to each year. Cross street ADT data was available for 1997 and 1999, from which 1998 ADTs were interpolated. The 3% annual growth was still used to obtain 2000 cross street ADTs in order to conform with the procedure used with the other interchanges.

Frontage road ADTs were also available from the City of Tempe for the Broadway Road and Southern Avenue interchanges for 1998 only. These values were used to calculate 1999 ADTs for the frontage roads by interpolating between 1998 and 2000, which were obtained from video observations and subsequent calculations explained above. This was the main reason for this specific attention to these interchanges since the frontage roads at these locations were actually directional roadways for Price Road. Since Price Road served as the only means for Loop 101 traffic to proceed to or from US-60 and southward, it was important to accurately estimate the ADTs on these frontage roads.

In March of 1999, the interchange between Loop 101 and US-60 became fully operational. As was done for the other interchanges, a 3% annual reduction was applied to the 1998 frontage road ADTs in order to obtain 1997 ADTs.

This study used the ADTs to calculate total entering vehicles for each interchange. To do this, the ADTs on the east and west legs (*Note: the selected study interchanges were all configured so that the cross-street was east-west and the frontage roads were north-south*) were divided by two to obtain the volume entering the interchange from each cross-street leg. The frontage road ADTs were calculated from video observations so no adjustments were necessary to determine entering volume. The exception was in the case of Broadway Road and Southern Avenue interchanges where specific frontage road (Price Road) ADTs were obtained from the City of Tempe and adjusted to represent entering volumes based on directional splits provided by the City.

The average daily total entering vehicles for each interchange are shown in the Table 6.

Table 6. Average Daily Total Entering Vehicles (TEV) by Interchange and Year

Interchange		1997 TEV	1998 TEV	1999 TEV	2000 TEV
T U D I	I-17 / Thomas Road	64,302	66,231	68,218	70,264
	I-17 / Indian School Road	80,378	82,790	85,273	87,831
	I-17 / Peoria Avenue	78,674	81,034	83,465	85,969
	Loop 101 / Broadway Road	51,437	51,850	57,200	64,132
	Loop 101 / Southern Avenue	69,387	66,850	62,150	61,788
S P U I / F	I-17 / Camelback Road	69,315	71,395	73,537	75,743
	I-17 / Northern Avenue	52,665	54,245	55,873	57,549
	I-17 / Dunlap Avenue	66,401	68,393	70,445	72,558
	SR 51 / Thomas Road	65,572	67,540	69,566	67,814
	Loop 101 / Guadalupe Road	49,266	50,744	52,266	53,834

CRASH STUDIES

Crash Report Retrieval

A list of crashes occurring for the most recent 3-year period at the time of investigation was obtained from ADOT and their Accident Location Identification Surveillance System (ALISS) database. The crashes analyzed in this study were from July 1, 1997 through June 30, 2000. The search criteria was specified to only tabulate a list of crashes that occurred on the cross street or frontage roads up to a distance of 90 m (300 ft) upstream and downstream of the interchange. The search produced a list of 1,121 crashes occurring in the 3-year span for 9 interchanges (the interchange at Guadalupe Road and Loop 101 was omitted from the crash analysis since at the time of study, Loop 101 did not exist south of the inter-change). This search result was also supplemented by crash data from the City of Phoenix for the SR 51/Thomas Road interchange and from the City of Tempe for the Loop 101 interchanges at Broadway Road and Southern Avenue. These supplemental crashes added about 200 crashes to bring the overall total of crashes analyzed to about 1,300. The supplemental crashes were confirmed to be different crashes than were included in the ALISS database.

These lists of crashes were then used to locate copies of the actual crash reports on file at ADOT Traffic Records. The crash reports were evaluated and printed out for later use. Some of the crashes were excluded due to insufficient data reported (e.g., unable to determine crash location or reason), the crash occurred due to construction, or the crash occurred on the on- or off-ramps for the freeway lanes. Additional crash reports were excluded on an interchange by interchange basis concerning whether the interchange was in the midst of being reconstructed. The crash reports were reviewed to determine an approximate date of when construction was completed and the interchange began operations as the type of interchange it operates at present (in all cases where the 3-year desired study period was reduced was due to a reconstruction of the interchange from TUDI operation to SPUI/F operation). Any crashes prior to this date were excluded from the crash totals for that particular interchange. Thus, the period of time for which crashes were analyzed varies between interchanges with a maximum analysis time of 36 months and a minimum of seven and a half months at the I-17/Camelback interchange.

Crash Data Reduction

In order to aid analysis, collision diagrams were created for each interchange based on the actual crash reports. This activity allowed for interpretation of certain crash tendencies at the interchanges as well as what location within the interchange tends to have more crashes (i.e., frontage road versus cross street). These crashes were also logged in a database (essentially a portion of the ALISS database for the associated crashes being analyzed) which showed key information about the crash. At this time, the crash types were tabulated for each interchange.

The following tables are a summarization of the crash results obtained from the analysis:

Table 7. Crash Data Summarization

			Specific Crash Type Breakdown (% of total)						
	Interchange	Total Crashes	Single Vehicle	Side-swipe	Angle	Left-Turn	Rear-End	All Others	Period (mos.)
T U D I	I-17 / Thomas Road	153	3 (2.0)	9 (5.9)	73 (47.7)	31 (20.3)	36 (23.4)	1 (0.7)	36
T U D I	I-17 / Indian School Road	114	3 (2.6)	12 (10.5)	34 (29.8)	18 (15.8)	44 (38.7)	3 (2.6)	36
T U D I	I-17 / Peoria Avenue	151	8 (5.3)	51 (33.8)	23 (15.2)	12 (8.0)	54 (35.7)	3 (2.0)	36
T U D I	Loop 101 / Broadway Road	133	4 (3.0)	23 (17.3)	27 (20.3)	12 (9.0)	62 (46.7)	5 (3.7)	36
T U D I	Loop 101 / Southern Avenue	125	6 (4.8)	16 (12.8)	20 (16.0)	8 (6.4)	72 (57.6)	3 (2.4)	36
S P U I / F	I-17 / Camelback Road	18	1 (5.6)	1 (5.6)	9 (50.0)	3 (16.7)	4 (22.1)	0 (0)	7.5
S P U I / F	I-17 / Northern Avenue	44	2 (4.5)	6 (13.6)	16 (36.4)	2 (4.5)	15 (34.2)	3 (6.8)	16
S P U I / F	I-17 / Dunlap Avenue	103	4 (3.9)	20 (19.4)	21 (20.4)	8 (7.8)	45 (43.7)	5 (4.8)	26
S P U I / F	SR 51 / Thomas Road	133	2 (1.5)	8 (6.0)	28 (21.0)	18 (13.5)	77 (58.0)	0 (0)	36
S P U I / F	Loop 101 / Guadalupe Road	n/a							

Maximum analysis period studied was from July 1, 1997 to June 30, 2000 (36 months)

Table 8. Crash Data Summarization

Type	Interchange	Crashes	Single Vehicle	Side-swipe	Angle	Left-Turn	Rear-End	All Others	Period (mos.)
			Specific Crash Type Breakdown						
TUDI	I-17 / Thomas Road								
	Total	153	3	9	73	31	36	1	36
	’97-‘98	58	1	3	29	11	13	1	12
	’98-’99	48	2	3	23	10	10	0	12
	’99-‘00	47	0	3	21	10	13	0	12
	I-17 / Indian School Road								
	Total	114	3	12	34	18	44	3	36
	’97-‘98	33	1	3	14	5	9	1	12
	’98-‘99	35	2	3	4	9	17	0	12
	’99-‘00	46	0	6	16	4	18	2	12
	I-17 / Peoria Avenue								
	Total	151	8	51	23	12	54	3	36
	’97-‘98	37	2	12	4	3	15	1	12
	’98-‘99	57	3	24	10	4	15	1	12
	’99-‘00	57	3	15	9	5	24	1	12
	Loop 101 / Broadway Road								
	Total	133	4	23	27	12	62	5	36
	’97-‘98	42	1	7	8	3	22	1	12
	’98-‘99	52	3	7	13	6	21	2	12
	’99-‘00	39	0	9	6	3	19	2	12
	Loop 101 / Southern Avenue								
	Total	125	6	16	20	8	72	3	36
	’97-‘98	36	0	4	5	3	24	0	12
	’98-‘99	49	4	8	6	3	27	1	12
	’99-‘00	40	2	4	9	2	21	2	12
SPUI/F	I-17 / Camelback Road								
	Total	18	1	1	9	3	4	0	7.5
	’97-‘98	0	0	0	0	0	0	0	0
	’98-‘99	0	0	0	0	0	0	0	0
	’99-‘00	18	1	1	9	3	4	0	7.5
	I-17 / Northern Avenue								
	Total	44	2	6	16	2	15	3	16
	’97-‘98	0	0	0	0	0	0	0	0
	’98-‘99	9	0	0	4	1	4	0	4
	’99-‘00	35	2	6	12	1	11	3	12
	I-17 / Dunlap Avenue								
	Total	103	4	20	21	8	45	5	26
	’97-‘98	13	0	1	3	0	8	1	2
	’98-‘99	51	2	12	13	6	15	3	12
	’99-‘00	39	2	7	5	2	22	1	12
	SR 51 / Thomas Road								
	Total	133	2	8	28	18	77	0	36
	’97-‘98	46	1	2	6	4	33	0	12
	’98-‘99	50	0	2	16	8	24	0	12
	’99-‘00	37	1	4	6	6	20	0	12
	Loop 101 / Guadalupe Road	n/a							

Crash Rate Comparisons

Crash rates are used to compare crash histories between sites, whether they are roadways, intersections, or in this case interchanges. The crash rate is based on a ratio of the number of crashes occurring at the location over a specified period of time and the amount of traffic using the roadway during this same period of time. In this study, the ratio was between crashes occurring in the period appropriate for the particular interchange and the total entering vehicles for that interchange over the same period. The calculation is shown below (the ratio is multiplied by one million in order to give results in terms which are easier to compare):

$$\text{RMEV} = (\text{Crashes} \;/\; \text{TEV}) \times 1{,}000{,}000 \qquad \textbf{(1)}$$

where:
RMEV = crash rate per million entering vehicles
Crashes = total crashes in analysis period
TEV = total entering vehicles for analysis period (using data from Table 6)

The crash rates were calculated for total crashes occurring at a particular interchange for the specific period of time pertaining to each interchange. The results are shown in Table 9:

Table 9. Crash Rates (RMEV)

	Interchange	RMEV (All Crash Types)
TUDI	I-17 / Thomas Road	2.08
TUDI	I-17 / Indian School Road	1.24
TUDI	I-17 / Peoria Avenue	1.68
TUDI	Loop 101 / Broadway Road	2.18
TUDI	Loop 101 / Southern Avenue	1.76
	All TUDIs (5 locations)	1.79 (mean)
SPUI/F	I-17 / Camelback Road	1.05
SPUI/F	I-17 / Northern Avenue	1.60
SPUI/F	I-17 / Dunlap Avenue	1.85
SPUI/F	SR 51 / Thomas Road	1.79
SPUI/F	Loop 101 / Guadalupe Road	n/a
	All SPUI/Fs (4 locations)	1.57 (mean)

Crash Data Analysis

Total Crashes

Statistical analysis was performed with these crash rates to determine whether this is a significant difference between the crash rates at TUDIs versus the crash rates at SPUI/Fs for the nine interchanges. A statistical test called the t-test was used, which is based on the premise of testing a hypothesis, in this case whether the crash rates at TUDIs and SPUI/Fs can be considered equal. If the calculated “t” value (absolute value) is less than the “t” value listed in statistical tables, then the result is that there is no significant difference. The "t" value listed in the statistical tables is based on the type of test being performed (in this case, a one-tailed test), number of samples/degrees of freedom, and the significance level, which is 5% for this study. The significance level means that there is a .05 probability or less that the difference observed is due to chance if the null hypothesis is rejected. For the comparison of crash rates between TUDI and SPUI/F the null hypothesis, H_0 and the alternate hypothesis, H_A, are as follows:

H_0: $R_T = R_S$, and
H_A: $R_T > R_S$ or $R_T < R_S$,

Where R_T is the mean crash rate for all TUDIs and R_S is the mean crash rate for all SPUI/Fs.

The calculated "t" value for comparing crash rates between the two interchange types was –0.352. The "t" value for 0.05 significance (one-tailed test) is 2.365. Since the absolute value of the calculated "t" is less than 2.365, the null hypothesis that there is not a significant difference between the crash rates at TUDIs and SPUI/Fs must be accepted.

The crash rates shown in Table 9 were calculated based on the length of analysis period. Since the analysis period at some locations were less than others, an additional t-test was performed using the crash rates calculated on a one-year analysis period basis. For instance, a TUDI location that has a 36-month analysis period would result in three separate crash rates for that site. These three rates would be summed with the yearly crash rates from the other TUDI sites in order to calculate an overall mean. The same calculations were performed for the SPUI/F locations although the total sample size was smaller due to SPUI/F analysis periods being shorter (in the case of the Camelback location, only 7 ½ months of data was applicable and thus was not used since it was less than a year's worth of data). In the end, the mean crash rates for TUDIs (1.79) was based on 15 separate crash rates, while the mean crash rate for SPUI/Fs (1.76) was based on six separate crash rates. Table 10 shows the data used in this approach.

Table 10. Yearly Crash Rates

	Interchange	Crash Rate ('97-'98)	Crash Rate ('98-'99)	Crash Rate ('99-'00)
TUDI	I-17 / Thomas Road	2.43	1.96	1.86
TUDI	I-17 / Indian School Road	1.11	1.14	1.46
TUDI	I-17 / Peoria Avenue	1.27	1.90	1.84
TUDI	Loop 101 / Broadway Road	2.23	2.61	1.76
TUDI	Loop 101 / Southern Avenue	1.45	2.08	1.77
Mean Yearly Crash Rate (All TUDIs, All years)		**1.79**		
SPUI/F	I-17 / Camelback Road	n/a	n/a	n/a (only 7 ½ mo.)
SPUI/F	I-17 / Northern Avenue	n/a	n/a (only 4 mo.)	1.69
SPUI/F	I-17 / Dunlap Avenue	n/a (only 2 mo.)	2.01	1.49
SPUI/F	SR 51 / Thomas Road	1.89	2.00	1.48
SPUI/F	Loop 101 / Guadalupe Road	n/a	n/a	n/a
Mean Yearly Crash Rate (All SPUIs, All years)		**1.76**		

The t-test revealed that there is not a statistical difference between the crash rates associated with TUDIs as compared to the crash rates associated with SPUI/Fs. The calculated "t" value was 0.155 which was compared against a "t" value for 0.05 significance (one-tailed test) of 1.729 (smaller than the value used in the prior t-test of the same nature due to increased sample size/degrees of freedom).

Crashes By Type

Another type of statistical test was performed in order to test whether certain types of crashes were significantly different when comparing the two types of interchanges. The test needed to determine this is called the Proportion Test. It is based on the proportion of the type of crash being investigated versus the total number of crashes occurring. Therefore, the proportions of single vehicle, sideswipe, angle, left-turn, and rear-end crashes were analyzed for significant differences between the two types of interchanges. The table below shows the percent totals of crash types for each type of interchange.

Table 11. Percentage of Crash Types by Interchange Type

	Total Crashes	Single Vehicle	Sideswipe	Angle	Left Turn	Rear-End	All Others
All TUDIs	676	3.6%	16.4%	26.2%	12.0%	39.6%	2.2%
All SPUI/Fs	298	3.0%	11.7%	24.8%	10.4%	47.3%	2.7%

For the comparison of crash types between TUDI and SPUI/F the null hypothesis, H_0 and the alternate hypothesis, H_A, are as follows:

H_0: $P_T = P_S$, and
H_A: $P_T > P_S$ or $P_T < P_S$,

Where P_T is the proportion of a certain crash type for all TUDIs and P_S is the proportion of a certain crash type for all SPUI/Fs. The result of the Proportion Test calculations is a "z" value that equates to the following:

$$z = \frac{(P_T - P_S)}{\sqrt{P(1-P)\left[\left(\frac{1}{n_T}\right) + \left(\frac{1}{n_S}\right)\right]}} \qquad \textbf{(2)}$$

where:

$$P = \frac{(x_T + x_S)}{n_T + n_S}$$

$$P_T = \frac{x_T}{n_T}$$

$$P_S = \frac{x_S}{n_S}$$

x = number of crashes of specific type
n = total number of crashes

There was found to be two significant differences ("z" calculated is greater than "z" from table) when analyzing rear-end and sideswipe crashes at the two types of interchanges. The table below shows the calculated "z" values and the "z" values from the statistical tables. If the absolute "z" value calculated is less than the "z" value from the table, then the hypothesis is accepted which states that the proportion of crash type for TUDIs is equal to the proportion of crash types for SPUI/Fs.

Table 12. "z" Values from Proportion Test

Crash Type	"z" calculated (absolute)	"z" from table at .05 significance
Single Vehicle	0.421	1.645
Sideswipe	1.883	1.645
Angle	0.444	1.645
Left-Turn	0.712	1.645
Rear-End	2.235	1.645

The tests show that there is a significant difference between the rear-end and sideswipe crash types at TUDIs and SPUI/Fs. The evidence suggests that the hypothesis should be rejected and that the proportion of rear-end crashes are more prevalent at SPUI/Fs than TUDIs since the SPUI/F proportion is 47% versus 40% for TUDIs. Also, the evidence suggests that the proportion of sideswipe crashes are greater at TUDIs than SPUI/Fs since the TUDI proportion is 16% versus 12% for SPUI/Fs. However, an additional Proportion Test was conducted to determine whether there is a significantly greater proportion of sideswipe crashes occurring at the I-17/Peoria interchange than at the other TUDI interchanges. The results of the test did confirm this hypothesis, so the broader conclusion that sideswipe crashes are more prevalent at TUDIs than SPUI/Fs may have been influenced by the sideswipe crashes occurring at the I-17/Peoria interchange.

The Proportion Test was also used to determine whether the location within the interchange (frontage road versus arterial/cross-street) was significantly different for rear-end crashes and total crashes between the two interchange types. For the TUDI interchanges, 38.9% occurred on the frontage roads and for the SPUI/F interchanges, 49.0% occurred on the frontage roads. In the equation above, the "x" value would represent the number of rear-end crashes at a particular location in this case. The test for whether there was a significant difference in the proportion of rear-end crashes occurring on the frontage road between TUDIs and SPUI/Fs resulted in there being a significant difference ("z" calculated of 1.978 versus "z" from the table of 1.645), which suggests that SPUI/Fs have a greater proportion of rear-end crashes occurring on the frontage roads than at TUDIs. The test also found that the proportion of rear-end crashes occurring on the arterial road is greater at TUDIs when compared to SPUI/F.

The same test was performed for total crashes occurring on the frontage road or arterial road for TUDIs and SPUI/F. The result was the same as for the test performed for rear-end crashes. Therefore, the evidence suggests that a greater proportion of crashes occur on the frontage roads at SPUI/F than on the frontage roads at TUDIs. It was also found that a greater proportion of crashes occur on the arterial roads at TUDIs than at SPUI/F.

There is at least one possible reasoning for why there are more rear-end crashes on the frontage roads at SPUI/F rather than TUDIs. Drivers could be distracted by the "splitting" of traffic to the left-turn lanes (in all cases) and to the right-turn lanes (in SR 51/Thomas case). One would suspect that the rear-end crash frequency in the through lanes of frontage roads at TUDIs and SPUI/F would be close since these areas of the frontage roads operate in a similar manner.

However, the variable that sets SPUI/F apart from TUDIs is the “splitting” of traffic due to turn lane locations. This could increase the frequency of rear-end crashes on the frontage roads at SPUI/Fs due to increased merging of traffic and potential driver confusion due to the complexity of interchange layout as compared to a TUDI. This argument could also be applied to the reason rear-end crashes in general at SPUI/F are significantly greater than TUDIs.

Comparison of SPUI/F and SPUI/n Crash Analysis

Cheng (*18*) reported a lower accident rate at SPUI/n than at TUDIs in a study comparing crash data for three of each interchange type. In an accident study of eight SPUI/n and five TUDIs, Garber *(3)* concluded that there is no significant difference between SPUI/n and TUDI accident rates (including total, injury and property damage only). Messer, *et al.* (*5*) presented crash rates at five SPUI/n, which ranged from 0.64 to 2.70 accidents per million entering vehicles. The current research found crash rates for SPUI/F ranging from 1.48 to 2.01 crashes per million entering vehicles with a mean crash rate of 1.76 crashes per million entering vehicles. The current research also found no significant difference between crash rates of a SPUI/F when compared to a TUDI.

Garber concluded that a higher proportion of SPUI/n crashes occurred on the off ramps and on ramps as compared to those of the TUDI. Furthermore, he concluded that the proportion of accidents in the center of the intersection is greater at TUDIs. In the current research the evidence suggests similar findings that a greater proportion of crashes occur on the frontage roads at SPUI/Fs than on the frontage roads at TUDIs and that a greater proportion of crashes occur on the arterial roads at TUDIs than at SPUI/Fs. It also concludes that there are a greater proportion of sideswipe crashes at TUDIs when compared to SPUI/F and a greater proportion of rear-end crashes occur at SPUI/F when compared to TUDI.

CONFLICT STUDIES

Definition

Although traffic accident records provide the most direct measure of safety for a roadway location, adequate data may not be available for analysis. Accidents are sometimes not reported, or records may be only available for a time period which may not represent current conditions at the study area. For this reason, *traffic conflicts* are analyzed.

According to Glauz, *et al.* (20):

> A traffic conflict is a traffic event involving the interaction of two vehicles where one or both drivers may have to take evasive action to avoid a collision.

Glauz, *et al.* (20) also states:

> A traffic conflict is a traffic event involving two or more road users, in which one user performs some atypical or unusual action, such as a change in direction or speed, that places another in jeopardy of a collision unless an evasive maneuver is undertaken.

Traffic conflicts most often involve a braking or weaving of a vehicle as a result of the interaction with another vehicle.

There is often a fine line between what should and what should not be considered a traffic conflict. A driver braking due to the presence of a red signal or traffic queuing is not considered a traffic conflict. However, a driver who brakes to avoid a collision with a slow-moving vehicle who is unobstructed by any queuing or a red signal is considered a traffic conflict. Therefore, in heavily queued traffic conditions, a traffic conflict analysis will be less effective.

There are 14 basic types of conflicts at intersections:

1. Left-turn same-directions conflict
2. Right-turn same-direction conflict
3. Slow vehicle same direction conflict
4. Lane-change conflict
5. Opposing left-turn conflict
6. Right-turn cross-traffic-from-right conflict
7. Left-turn cross-traffic-from-right conflict
8. Through cross-traffic-from-right conflict
9. Right-turn cross-traffic-from-right conflict
10. Left-turn cross-traffic-from-left conflict
11. Through cross-traffic-from-left conflict
12. Opposing right-turn-on-red conflict
13. Pedestrian far-side conflict
14. Pedestrian near-side conflict

The following guidelines were used in identifying traffic conflicts:

- In addition to the initial (or "primary") conflict, a conflict as a result of the primary conflict (a "secondary" conflict) was often encountered. A maximum of one secondary conflict was recorded for each primary conflict, even if more than one secondary conflict occurred.

- Unusual occurrences due to the presence of ambulances, fire trucks or police vehicles, were identified but not included in the calculation of conflict rates.

- Actions taken by vehicles in response to traffic control devices, highway geometrics, or adverse weather, were not considered traffic conflicts.

- Conflicts due to faulty or stalled vehicles were considered. However, a maximum of two conflicts (one primary and one secondary) was recorded for each of these occurrences.

Conflict Rate Comparisons

The conflict rate is based on a ratio of the number of conflicts occurring at the location over a specified period of time and the amount of traffic using the roadway during this same period of time. In this study, a conflict rate was calculated for each of the three Periods, and an overall comparison was made between TUDI and SPUI/F conflict rates. The calculation is shown below (the ratio is multiplied by one thousand in order to give results in terms which are easier to compare):

$$\underbrace{\text{RTEV}}_{\substack{\textit{Rate per Thousand}\\ \textit{Entering Vehicles}}} = (\underbrace{\text{Conflicts}}_{\substack{\textit{Conflicts per}\\ \textit{Time Period}}} \ / \ \underbrace{\text{TEV}}_{\substack{\textit{Total}\\ \textit{Vehicles per}\\ \textit{Time Period}}}) \times 1{,}000 \qquad \textbf{(3)}$$

Traffic conflicts were recorded at the ten study interchanges in Period 1 (mid-morning), Period 2 (late-morning), and Period 3 (late afternoon), for a duration of two hours each. Conflict rates were calculated using the conflict data and traffic volumes for the two-hour periods, which are shown in Table 13.

Table 13. Conflict Rates (RTEV)

	Interchange	Period 1	Period 2	Period 3
TUDI	I-17 / Thomas Road	2.26	0.36	0.67
	(total number of conflicts)	16	3	7
	(2-hr volume collected)	7,084	8,221	10,426
	I-17 / Indian School Road	0.87	0.51	0.65
	(total number of conflicts)	8	6	9
	(2-hr volume collected)	9,202	11,856	13,924
	I-17 / Peoria Avenue	1.39	1.26	0.69
	(total number of conflicts)	12	14	9
	(2-hr volume collected)	8,639	11,146	13,117
	Loop 101 / Broadway Road	2.41	0.45	0.46
	(total number of conflicts)	20	4	6
	(2-hr volume collected)	8,303	8,925	12,972
	Loop 101 / Southern Avenue	0.72	1.72	3.91
	(total number of conflicts)	7	14	50
	(2-hr volume collected)	9,695	8,158	12,788
	TUDI Mean Conflict Rate	1.53	0.86	1.28
SPUI/F	I-17 / Camelback Road	1.33	2.13	2.24
	(total number of conflicts)	10	15	22
	(2-hr volume collected)	7,523	7,045	9,820
	I-17 / Northern Avenue	0.81	3.60	1.43
	(total number of conflicts)	6	28	16
	(2-hr volume collected)	7,405	7,788	11,206
	I-17 / Dunlap Avenue	4.39	3.75	3.21
	(total number of conflicts)	37	34	43
	(2-hr volume collected)	8,419	9,060	13,395
	SR 51 / Thomas Road	1.06	5.81	9.05
	(total number of conflicts)	8	43	82
	(2-hr volume collected)	7,528	7,405	9,062
	Loop 101 / Guadalupe Road	2.80	0.76	1.48
	(total number of conflicts)	27	5	18
	(2-hr volume collected)	9,629	6,540	12,134
	SPUI/F Mean Conflict Rate	2.08	3.21	3.48

Comparison of Mean Conflict Rates for the Same Periods

A t-test was used to compare conflict rates between TUDIs and SPUI/Fs for Period 1, 2 and 3. If the calculated "t" value (absolute value) is less than the "t" value listed in statistical tables relative to the conditions of the test, there is not a significant difference between conflict rates. As shown in Table14, Periods 1 and 3 provide no significant statistical difference in data between TUDI vs. SPUI/F conflict rates, while Period 2 data does represent a significant difference in results.

Table 14. "t-test" Results for TUDI vs. SPUI/F Conflict Rates

Period	t (calculated)	t (tables)*	Significant Difference in TUDI/SPUI/F data?
1	0.723	< 1.860	No
2	2.643	> 1.860	Yes
3	1.402	< 1.860	No

** based on .05 significance level for a one-tailed test*

Comparison of Mean Conflict Rates between Periods

The study area for Period 1 differs from the study area for Periods 2 and 3, as shown previously in Figure 12 and Figure 13. While Period 1 focused on the external interchange approaches, Periods 2 and 3 analyzed the internal portion of the interchange and downstream frontage roads. Therefore, only a comparison of Period 2 and Period 3 data would be meaningful.

A comparison of mean conflict rates between Period 2 and Period 3 is shown below in Table 15 for all SPUI/Fs and TUDIs. As shown, SPUI/Fs have higher mean conflict rates than TUDIs during both Periods. A t-test of SPUI/F vs. TUDI data was performed by combining Period 2 and Period 3 conflict data (since these Periods contained identical study areas and conflict types), and comparing SPUI/F and TUDI conflict rates. The results showed that the calculated t-value of 1.832 fell just below the t-value (0.05 significance level) of 1.860 listed in statistical tables. Therefore, there was no significant difference at the 0.05 significance level between SPUI/F and TUDI conflict for Periods 2 and 3 combined. However, one can conclude that SPUI/Fs had a greater conflict rate than TUDIs in Periods 2 and 3 combined at the 0.10 significance level.

Table 15. Period 2 and Period 3 Mean Conflict Rates

Interchange Type	Period 2 Mean Conflict Rate	Period 3 Mean Conflict Rate
SPUI/F	3.21	3.48
TUDI	0.86	1.28

Field Observations

The following occurrences were observed as it relates to traffic conflicts:

- Conflicts at Driveways: The presence of high-volume driveways within the TUDI and SPUI/F study area tended to increase the number of conflicts that occurred at an interchange. These conflicts often occurred as a result of vehicles braking to turn into driveways, which caused the following vehicle(s) to brake or weave. At the NB on-ramp at SR 51/Thomas Road, an access point to a busy shopping center was located a short distance from the frontage road merge area. The high volume of right turns in and out of driveways resulted in numerous right-turn same-direction conflicts. In addition, erratic maneuvers were frequently noted. Drivers were often seen entering the frontage road in the far left lane, then attempting to move to the far right lane to access the driveway. Driveways at Loop 101/Southern and I-17/Dunlap also heavily influenced interchange conflict rates at these locations.

- On-ramp conflicts on SPUI/Fs: A common scenario for conflicts on SPUI/Fs occurred between right turns from the crossroad and opposing left turns from the cross road, as shown in Figure 14. Right-turn-on-red vehicles from the cross street frequently turned wide into the center lane (and sometimes the left-most lane) of the ramp, where opposing left-turning traffic was traveling in the green phase. This created a dangerous weave condition on the ramp where the two movements joined. An exception to this case occurred at SR 51/Thomas, where an island granted right-turn channelization for this movement. The island provided guidance for right-turning traffic into the right-most lane of the frontage road. As discussed previously, SR 51/Thomas conflict rates were still relatively high, due to heavy driveway traffic on the frontage roads.

  The conflict in Figure 14 (at the on-ramp merge) was observed at a higher frequency than for the right turn from the off-ramp (conflicting with the left turn from the opposing ramp). Driver expectancy could provide a reason for this difference in results. Under normal conditions, a right-turning driver would expect to look left before proceeding on red. This is the case for off-ramp right turns, where conflicting traffic enters the driver's view from the left. However, right turns from the crossroad must also be aware of left-turns from the crossroad, and u-turns from the far side off-ramp.

- Erratic Turns on SPUI/Fs: Erratic turns on SPUI/Fs were noted when drivers on the off-ramp turning left or right from the thru lane. Similarly, drivers were observed taking left turns into the through lanes of the on-ramp, instead of into the left-turn lanes. (see Figure 15)

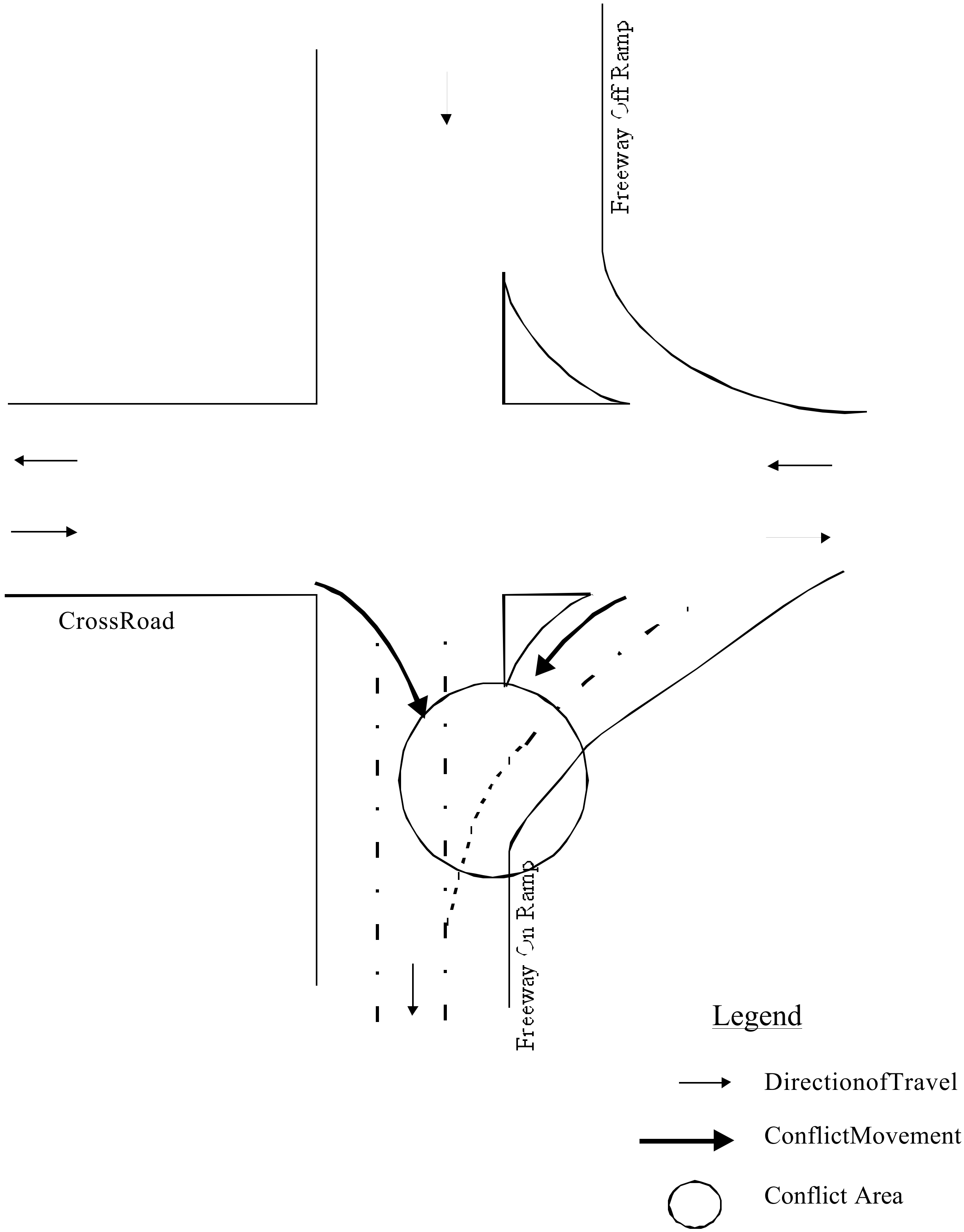
- Red-Light Violators: Garber, *et al.* (3) noted a problem observed at the SPUI/F involving the off-ramp left-turning vehicles. Inadequate sight distance from the driver eye to the left-turn signal head resulted in occasional "runs" of red lights. Although this is worth noting, the data collection did not provide any conclusive evidence regarding TUDI and SPUI/F red light "runs".

- Intersection Spacing: At the Loop 101/Guadalupe Road interchange, a signalized intersection is located within a few hundred feet of the NB on-ramp. Heavy traffic volumes for WB traffic approaching the ramp during the AM peak hour queued traffic beyond the adjacent intersection. This condition created new conflicts in the area.

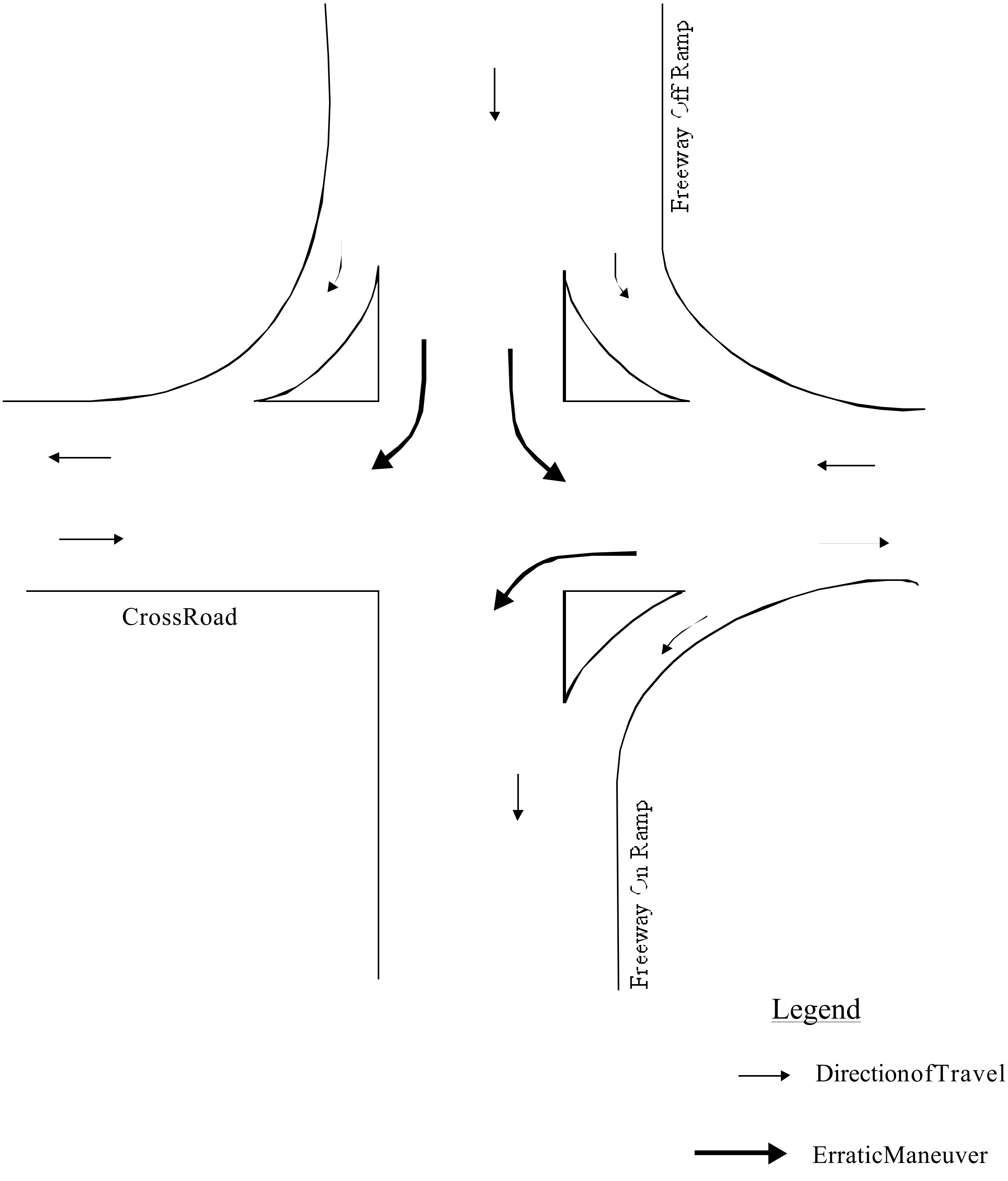
- Congested Traffic: Conflict rates at the Loop 101/Guadalupe interchange were relatively low when compared to other SPUI/F locations, and congestion at the interchange provides an explanation for this. Congested traffic in some instances reduces the number of *recorded* conflicts at an interchange, since a braking due to the presence of a red signal or traffic queuing is not considered a conflict.

<u>Additional Statistical Analyses on Period 3 Conflict Rates.</u>

A statistical analysis was performed using Period 3 data, dropping out data sets at interchanges where conflict rates were heavily influenced by the presence of driveways on the frontage roads. The SR 51/Thomas, Loop 101/Southern, and I-17/Dunlap interchanges were dropped from the data set to examine their impacts to conflict rates. Four separate "cases" were identified, as shown in Table 16. The use of Case A and Case B results in no significant difference in TUDI/SPUI/F conflict data. The Case C and D analyses resulted in a significant statistical difference between TUDI and SPUI/F data, with SPUI/Fs tending to have more conflicts than TUDIs.



Freeway Off Ramp
CrossRoad
Freeway On Ramp
Legend
DirectionofTravel
ConflictMovement
Conflict Area



Frequently Observed Erratic Maneuvers on SPUI/Fs

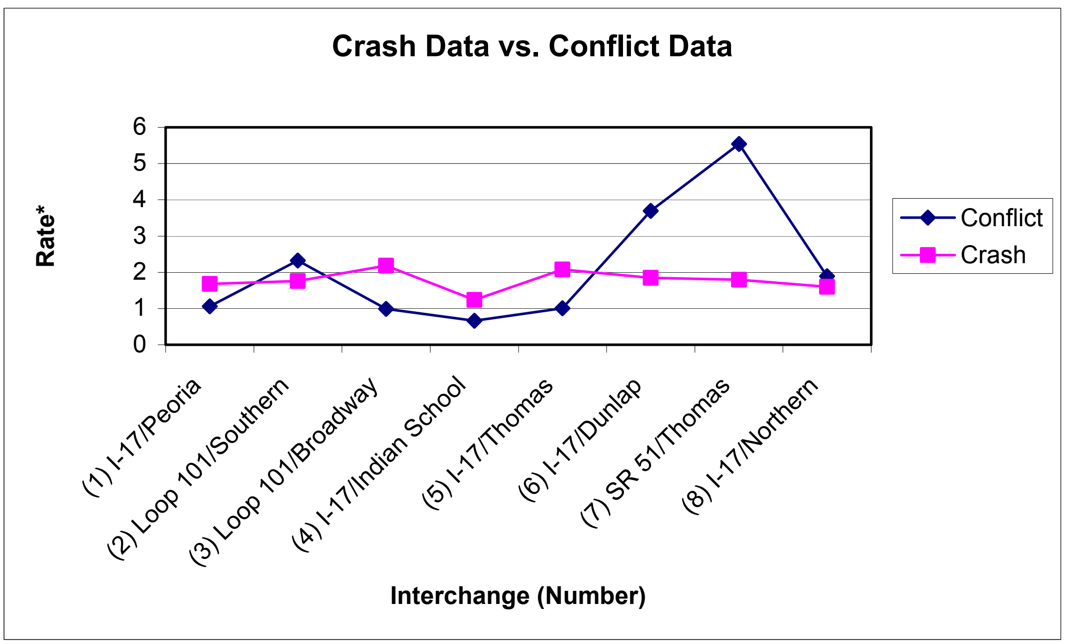
Figure 15

Table 16. "t-test" Results for Additional Analyses for Period 3

Case	Removed Data Set	t (calculated)	t (tables)*	Significant Difference in TUDI/SPUI/F Conflict Data?
A	SR 51/Thomas	.977	< 1.894	No
B	Loop 101/Southern	1.767	< 1.894	No
C	SR 51/Thomas & Loop 101/Southern	3.745	> 1.943	Yes – SPUI/Fs tend to have more conflicts
D	SR 51/Thomas, Loop 101/Southern, I-17/Dunlap	2.656	> 2.015	Yes – SPUI/Fs tend to have more conflicts

Crash and Conflict Data Comparison

The correlation between crash and conflict data was tested. A comparison was made of crash data vs. conflict data by plotting the rates and interpreting the data pairs. For this comparison, data from the I-17/Camelback interchange was not used since its crash data was based on a sample of less than a year. Also, the Loop 101/Guadalupe interchange was not included since crash rates were not calculated at this location. Figure 16 shows the relationship between the crash rate (per million vehicles) and conflict rate (per thousand vehicles) data. The conflict rates were obtained by summing the conflicts observed in Periods 1, 2 and 3 for each interchange and dividing by the total exposure volume for the three periods.



** Crashes per million vehicles, or conflicts per thousand vehicles*

Figure 16. Crash Data vs. Conflict Data

Inspection of the chart in Figure 16 reveals that interchanges 6 and 7, which were I-17 / Dunlap and SR 51 / Thomas respectively, appear to have a disproportionate conflict rate when compared to the crash rates. This is apparently the result of the nearby driveways. Interchange 2, (Loop 101 / Southern) also has a nearby driveway which raised the conflict rate at this location.

Other than those three interchanges, there appears to be a reasonable relationship between conflict rate and crash rate.

The chart was based on the most extensive data sample possible for crash data per interchange. In other words, if crash data was available for three years for a particular interchange, the crash rate for that interchange was based on the whole three years worth of data. The lowest data sample basis was for the Northern interchange (IC #8) which has 16 months worth of crash data.

In order to evaluate the correlation between the data sets, a Pearson-r correlation test was performed. The result when considering the eight interchanges involved, was a correlation factor of 0.22. This implies that there is not much correlation between the crash rates and the conflict rates. However, the chart shows that IC #6 (Dunlap IC) and IC #7 (SR 51/Thomas IC) have data pairs that vary from the other interchange data pairs. Therefore, a sensitivity analysis was conducted where the Pearson-r correlation was re-evaluated without the data from these two interchanges. The result was a correlation factor of 0.53, which implies that when the "outlying" data pairs are removed, there is some correlation between the crash rates and conflict rates at each interchange.

SAFETY-RELATED OBSERVATIONS

Observations were made at the various interchanges that are presented here related to safety issues.

Pedestrian Safety

The nature of the pedestrian crossings of the frontage road at the SPUI/F appears to result in decreased pedestrian compliance with the pedestrian signals. For a pedestrian to comply with the pedestrian signals crossing the frontage road requires four cycles. This excessive duration coupled with obvious periods (to the pedestrian) where there are no vehicle conflicts results in an almost total disregard of the pedestrian signals (see Figure 17).

Figure 17. Example of Disregard for Pedestrian Signal Indication.

Figure 17 shows a pedestrian crossing the left-turning lanes from the cross road to the frontage road against the DON'T WALK signal. Since the frontage road traffic has the green light, there is no conflict on the particular lanes being crossed.

Loop 101 / Guadalupe Interchange

The Technical Advisory Committee expressed concern about the interchange of Loop 101 and Guadalupe. The fact that it was still under construction at the time of data collection precluded it from being included in the crash analysis, however observations are presented here relating to safety issues:

- Nearby signalized intersection: The construction of the interchange resulted in a distance between the center of the northbound frontage road and the center of the existing signalized intersection at Carriage Lane of approximately 110 m (360 feet). This seriously compromised the operational efficiency of the interchange, which resulted in observed safety-related issues.

Figure 18. Westbound Traffic Queue on Guadalupe in the AM Peak.

A high percentage of these drivers shown in Figure 18 wish to make a right turn at Loop 101 to proceed north. The traffic signal at Carriage Lane not only reduced the efficiency of the interchange operation, but it also provided an opportunity for motorists who elect to avoid Guadalupe and pass through the neighborhoods on Carriage Lane or Noche De Paz to enter Guadalupe at the signal (see Figure 19). The high number of vehicles turning from Carriage Lane to go west on Guadalupe suggests that there is a time savings by doing so. This results in increased traffic through the neighborhood. The majority of these motorists wish to then make a right turn at the northbound frontage road 110 m (360 ft) away.

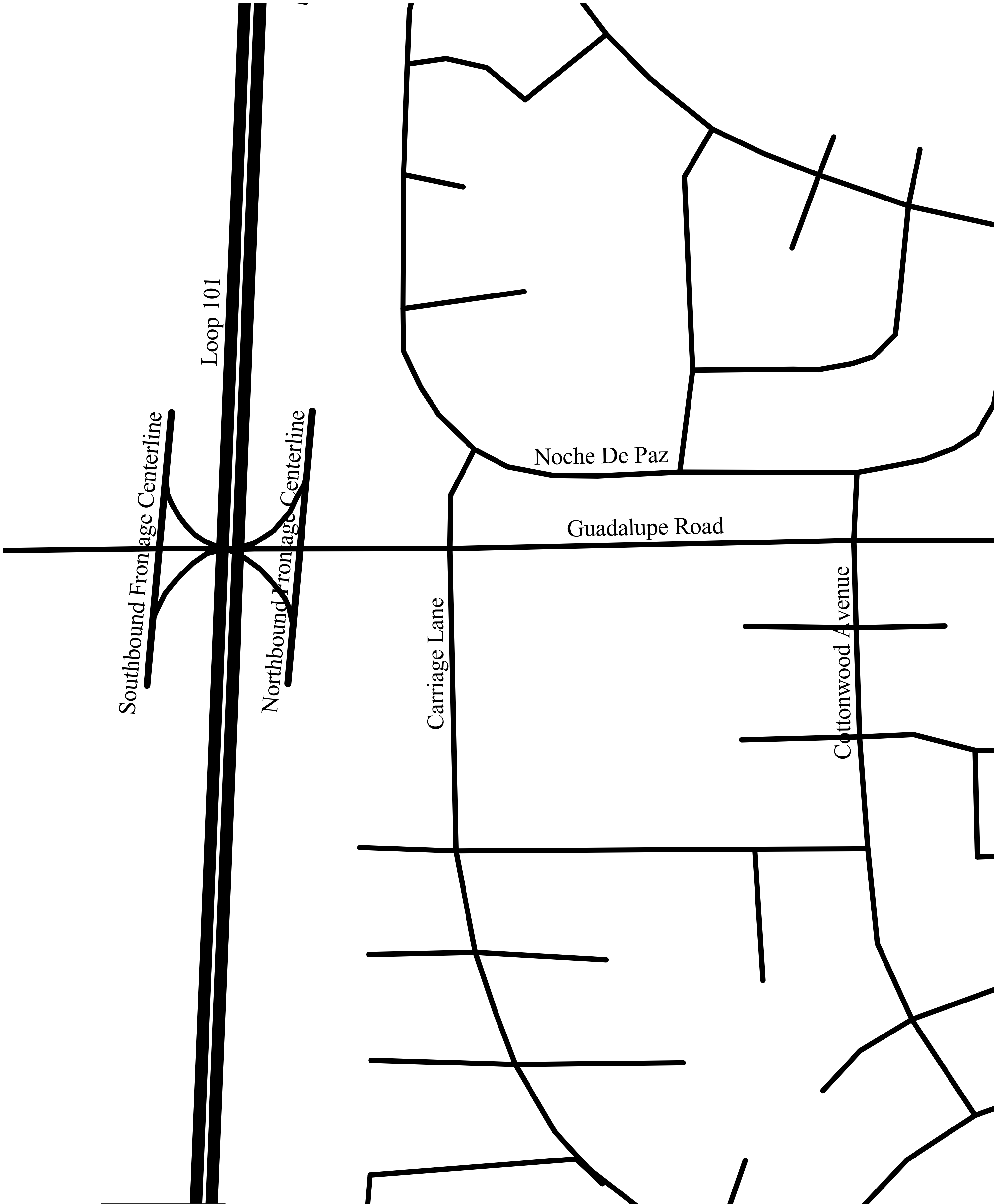


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Figure 19

Figure 20. Congestion on Westbound Guadalupe Road at Carriage Lane.

Figure 20 is looking west from Carriage Lane and shows the resulting congestion, especially in the right lane. One can see the small pickup which entered Guadalupe from northbound Carriage Lane when that movement had the green light blocking the center lane in an attempt to reach the right-turn lane. The sport utility vehicle which made a right turn did a similar thing. In the distance can be seen a vehicle in the bicycle lane attempting to merge into the right turn lane.

Figure 21. Looking East from Interchange at Westbound Guadalupe Traffic Congestion.

Figure 21 shows a motorist making a dual right turn from the through lane.

- Construction-related congestion: At the time of the data collection Loop 101 was still under construction. All southbound traffic approaching Guadalupe had to either exit at Guadalupe or merge into one southbound lane which passed under the interchange structure (see Figure 22).

Figure 22. Southbound Traffic Queues Associated with Loop 101 / Guadalupe Interchange

In the upper left of Figure 22 one can see the long queue of southbound vehicles approaching Guadalupe. All of these vehicles had to turn left or right at Guadalupe since the southbound frontage road south of the intersection was not open. The majority of the motorists desired to turn left, resulting in long delays.

- Red-light running: Perhaps because of the long delays, numerous motorists were observed running the red light at this interchange.

Figure 23. Southbound Vehicles Disregarding Red Signal Indication.

In Figure 23 the two southbound vehicles approaching the intersection from the left proceeded through the intersection although the left-turn signal is red.

SUMMARY

Crash Analysis

The interchange of Loop 101 and Guadalupe Road was not included since Loop 101 did not exist south of the interchange. Thirty-six months of crash data were analyzed at the nine other interchanges except I-17/Dunlap (26 months); I-17/Northern (16 months); and I-17/ Camelback (7.5 months). Crash rates per million vehicles entering the interchange were computed giving equal weight to each interchange without regard to the number of months of data available:

Mean rate for TUDIs	=	1.79
Mean rate for SPUI/Fs	=	1.57

A different approach, which weighted the data based on the number of months that data was available, indicated the rate for TUDIs continued to be 1.79, while the rate for SPUI/Fs was 1.76. This is believed to be a better statistic, because it doesn't give the same weight to a 7.5 month sample as to a 3-year sample.

Statistical tests indicated:

The difference in crash rates between TUDIs and SPUI/Fs is not significant.
The greater proportion of rear-end crashes occur on the frontage roads at SPUI/Fs.
The greater proportion of rear-end crashes occur on the arterial at TUDIs.

Conflict Studies

A traffic conflict is defined as a traffic event involving the interaction of two vehicles where one or both drivers may have to take evasive action to avoid a collision. Conflict studies were made at the ten interchanges during three periods – mid-morning, late morning and late afternoon.

At the 0.05 significance level, there was no significant difference between the SPUI/F and TUDI conflict rates, but at the 0.10 significance level, SPUI/Fs had a greater conflict rate than TUDIs.

Some correlation was found between the crash rates and conflict rates of each interchange.

Other Observations

The presence of high-volume driveways within the TUDI and SPUI/F study area tended to increase the number of conflicts that occurred at an interchange.

A common scenario for conflicts on SPUI/Fs occurred between right turns from the cross road and opposing left turns from the cross road.

The nature of the pedestrian crossings of the frontage road at the SPUI/F appears to result in decreased pedestrian compliance with the pedestrian signals. At wide SPUI/F locations (especially Loop 101 and Guadalupe) vehicles were observed stopping in the middle of the intersection area. These were apparently the result of driver confusion associated with the large expanse of pavement without clear definition of vehicle paths, stopping locations and applicable signal indications.

CHAPTER 4
EVALUATION OF INTERCHANGE OPERATIONS

INTRODUCTION

This chapter describes the findings from an evaluation of the operational performance of the tight urban diamond interchange (TUDI) and the single point urban interchange with frontage roads (SPUI/F). The evaluation consists of a delay-based comparison of the two alternative interchange forms and reflects the influence of turn movement volume patterns, number of traffic lanes, signal phase sequence, and change interval duration. The analysis does not explicitly address the influence of coordination with adjacent signalized intersections as experience indicates it is not used at most interchanges and, when used, it is difficult to define and sustain over time.

The objective of the evaluation was to define the conditions where one interchange form and phase sequence is operationally more efficient than another. This objective was achieved by developing, calibrating, and applying an analytic model of interchange capacity and delay. The remainder of this report describes the development of the model, the collection of model calibration data, and the use of the model to evaluate the TUDI and SPUI/F forms.

APPROACH

Background

Several researchers have attempted to characterize conditions where one interchange form is more appropriate than another, based on operational considerations. These efforts are summarized in Chapter 1. The conclusions reached by these researchers identify volume conditions and lane configurations for which one interchange tends to be more efficient than another. The following are examples of such conclusions:

1. Leisch *et. al.*(*6*):
   "Conditions where the single-point becomes a competitive alternative to the compressed [TUDI] are where the controlled access facility is four-lanes wide, certainly no more than six lanes, the cross street has no more than two through lanes in each direction, all turning movements are light-to-moderate (requiring single left-turn lanes), and there are no frontage roads."

2. Fowler (*16*):
   a. "As the directional split of the cross street through volumes increases, the performance of the TUDI improves."
   b. "As the volume of the cross-street left-turn opposing the heavy through movement increases, the performance of the TUDI increases."
   c. "As the off-ramp left-turns become more imbalanced, the performance of the TUDI improves."

3. Garber and Smith (*3*):
   "The SPUI is more efficient in situations where the proportion of traffic at the interchange to and from the major road (left-turn movements) is relatively higher than the other movements..."

The conclusions listed above, offered as guidance to engineers for interchange selection, are vague and subject to differing interpretations. Moreover, their emphasis on qualifying statements rather than quantitative information make them difficult to apply with any confidence. It is also interesting to note that there is no strong agreement or clear common-ground among the statements. As a result, there is little information that can be confidently gleaned from the collective work.

Model Description

A model for evaluating interchange operations is described in this section. This model was developed to facilitate the production of a quantitative procedure for evaluating alternative interchange forms. Initially, the analysis approach is described and evidence is offered to support its suitability for interchange analysis. Then, the interchange evaluation model analysis procedures are described.

Critical Movement Analysis Approach

The interchange evaluation model developed for this research is based on the "critical-movement analysis" (CMA) approach that forms the basis for signalized intersection analysis in the *Highway Capacity Manual 2000* (*HCM*) (*13*). The CMA approach is based on a mathematical representation of an intersection's (or interchange's) signal phase sequence as implemented in a single or dual-ring controller whose phase durations are dictated by lane volume. The CMA approach is described in Chapter 16 of the *HCM*.

Cycle Length. The thrust of the CMA approach is that the duration of a phase is dictated by the largest, or "critical," flow ratio (i.e., ratio of volume to saturation flow rate v/s) of all movements served during that phase. When single-ring control is used, the critical flow ratio for each phase can be totaled to obtain the sum-of-critical-flow-ratios. This sum dictates the cycle length, as defined by the following equation:

$$C = \frac{n_p \, l_t \, X_c}{X_c - \sum \left(\frac{v}{s} \right)_{ci}} \quad \textbf{(1)}$$

where,

C = cycle length, s;
n_p = number of phases;
l_t = lost time per phase, s;
X_c = critical volume-to-capacity ratio for the intersection;
v_{ci} = critical demand flow rate for phase i, veh/h;
s_{ci} = critical saturation flow rate for phase i, veh/h; and
$\Sigma(v/s)_{ci}$ = sum of critical flow ratios.

When dual-ring control is used, the operation is more complex; however, there is still an identifiable critical combination of phase flow ratios that can be added together and used in Equation 1 to predict the resulting cycle length.

The relationship between cycle length and the sum-of-critical-flow-ratios is shown in Figure 24 with a dashed line. The trend line shown indicates that cycle length increases with increasing sum-of-critical-flow-ratio.

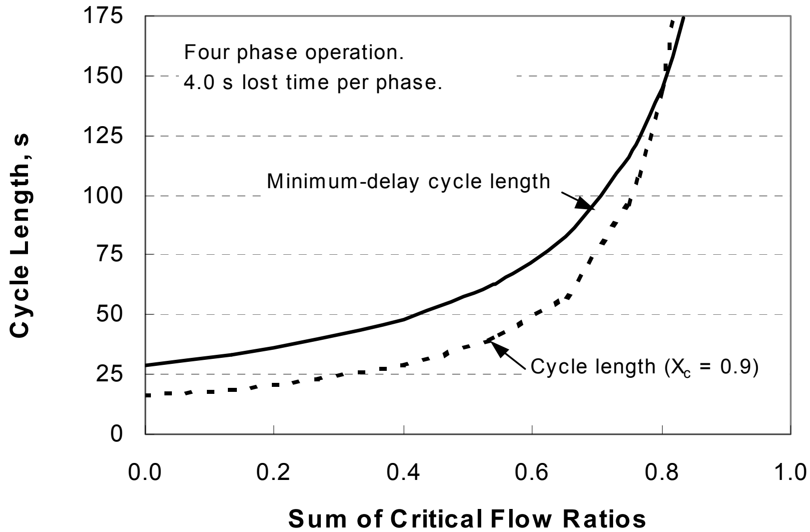


Figure 24. Effect of critical-flow ratio on cycle length.

Webster (*22*) derived an equation similar to Equation 1 that related the sum-of-critical-flow-ratios to a cycle length that, if distributed equitably among the phases, would minimize delay. Webster's equation is:

(5)

$$C_o = \frac{(1.5\ n_p\ l_t) + 5}{1 - \sum\left(\frac{v}{s}\right)_{ci}}$$

where,

C_o = minimum-delay cycle length, s.

The relationship between the minimum-delay cycle length and sum-of-critical-flow-ratios is shown in Figure 24 with a solid line. This trend line indicates that the minimum-delay cycle length C_o is larger than the cycle length obtained from Equation 1 for critical flow ratios less than about 0.8. The trend reverses for larger critical flow ratio sums. This point of intersection defines the critical volume-to-capacity ratio X_c associated with the minimum delay cycle length.

Average Delay. In addition to cycle length, the sum-of-critical-flow-ratios also has a direct influence on the average delay for a traffic movement. The delay equation provided in the *HCM* (*13*) was used to examine this relationship. This equation has the form:

(6)

$$d = d_1 + d_2$$

where:

d = average control delay, s/veh;

d_1 = delay due to uniform arrivals; s/veh; and

d_2 = delay due to random and oversaturation queues (i.e., incremental delay), s/veh.

The two delay terms are computed using the following two equations:

$$d_1 = \frac{0.5\ C\ (1 - \frac{g}{C})^2}{1 - \mathrm{Min}\,(1, X)\ \frac{g}{C}} \tag{7}$$

$$d_2 = 900\ T \left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8\ k\ I\ X}{c\ T}} \right] \tag{8}$$

where:

X = volume-to-capacity ratio (= v / c);

T = duration of analysis period, hours;

k = incremental delay factor;

I = upstream filtering/metering adjustment factor; and

c = capacity (= $s\ g / C$), veh/h.

The *HCM* (*13*) indicates that the incremental delay factor k ranges in value from 0.04 to 0.50 for actuated phases and is constant at 0.5 for pretimed phases. Values of k less than 0.5 reflect the tendency of actuated phases to avoid incremental delay by minimizing the frequency of phase termination by max-out (i.e., green extension to the maximum green interval setting). Theoretically, if the phase never terminates by max-out, then k (and d_2) equal 0.0. The *HCM* also indicates that the upstream filtering/metering factor I is equal to 1.0 for isolated intersections. It has a value less than 1.0 when there is a nearby signalized intersection.

The equations described previously were used to examine the relationship between critical flow ratio and delay. Three assumptions were made to facilitate the analysis. First, it was assumed that all actuated phases have a large maximum green interval setting such that k (and d_2) will equal 0.0. Second, the cycle length used for the analysis was assumed to equal to the minimum-delay cycle length C_o from Equation 2 and the value of X used is the critical X_c obtained from Equation 1 (when the cycle length C is set equal to C_o). Third, each phase was assumed to contribute equally to the sum-of-critical-flow-ratios such that all phases have the same phase duration, volume-to-capacity ratio, and delay.

The results of the delay analysis are shown in Figure 25. As with cycle length, average delay increases with increasing sum-of-critical-flow-ratios. This figure also illustrates that the number of phases can influence average delay. Specifically, a fourth phase increases the delay by about 40 percent relative to three-phase operation for a given sum-of-critical-flow-ratios. This increase is due the addition of the fourth phase. Other factors that influence lost time (e.g., the larger size of a SPUI/F relative to a TUDI) may also lead to equally distinct and separate relationships between flow ratio and delay.

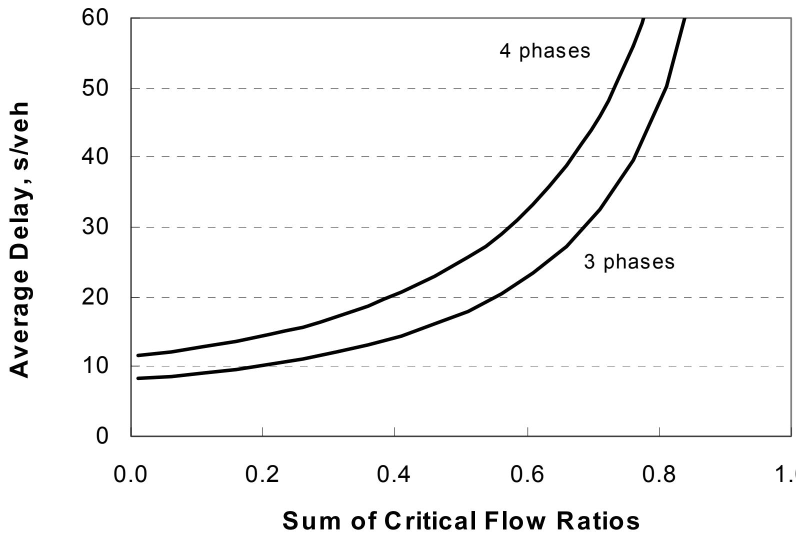


Figure 25. Effect of critical flow ratio on average delay.

Critical vs. Non-Critical Phases. The delay evaluation described in the previous section was focused on the critical phases for a typical intersection. This section examines the relationship between the sum-of-critical-flow-ratios and the overall intersection delay (i.e., a volume-weighted average delay that includes all intersection movements). As such, it includes the delay to both the critical and non-critical phases.

Table 17. Turn movement volume and lane configuration for delay evaluation.

Characteristic		Approach and Movement								Total
		Northbound		Southbound		Eastbound		Westbound		
		Left	Thru	Left	Thru	Left	Thru	Left	Thru	
Phase Number		3	8	7	4	5	2	1	6	—
Approach Lanes		2	2	2	2	2	3	2	3	18
Volume, veh/h	1	400	750	600	1000	800	1000	800	1000	6350
	2	300	950	700	700	800	1000	800	1000	6250
	3	350	575	450	750	350	825	550	850	4700
	4	250	700	550	525	350	825	550	850	4600
	5	350	575	450	750	350	850	550	825	4700
	6	250	700	550	525	350	850	550	825	4600
	7	350	575	450	750	475	625	475	1050	4750
	8	250	700	550	525	475	625	475	1050	4650
	9	300	400	300	500	300	600	150	600	3150
	10	200	450	400	350	300	600	150	600	3050

For this analysis, ten volume scenarios were developed that collectively offer a range in turn movement volume patterns and overall volume level. These volumes are listed in Table 17. Collectively, the volumes listed exhibit "balanced volumes," "heavy left-turn volumes," "heavy through volumes," and "heavy left plus heavy opposing through volumes."

Several assumptions were made for this examination. First, it is assumed that the phases are actuated and will have a large maximum green interval setting such that k (and d_2) will equal 0.0. Second, the cycle length used for the analysis is equal to the minimum-delay cycle length C_o from Equation 2 and the value of X used is the critical X_c obtained from Equation 1. Third, dual-ring controller operation was assumed as was leading, protected-only left-turn phases. The ring structure for this controller operation is shown in Figure 26.

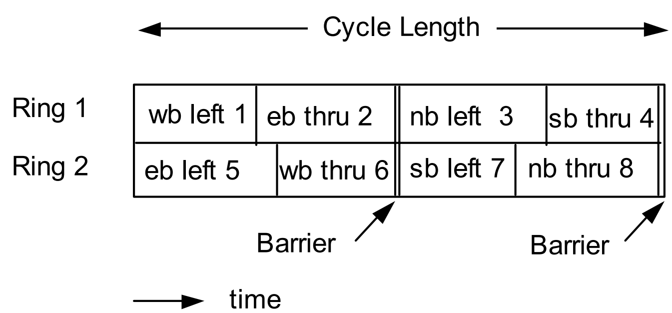


Figure 26. Ring structure for the example application.

The results of the analysis are shown in Figure 27. The trend line in this figure indicates that there is a strong correlation between intersection delay and the sum-of-critical-flow-ratios. Moreover, this correlation indicates that *intersection delay is largely unaffected by the turn movement patterns that underlie the sum-of-critical-flow-ratios.* Additional analyses for other lane configurations indicates that this finding is also true regardless of the number of lanes provided. It should be noted that, for a given sum-of-critical-flow-ratio, 70 percent of the individual movement delays were within ∀ 4.0 percent of the overall intersection delay.

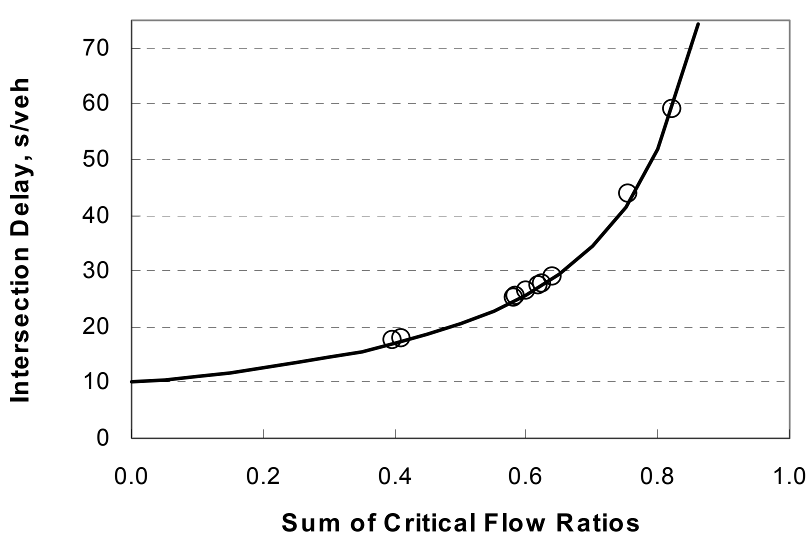


Figure 27. Effect of critical flow ratio on intersection delay.

Summary. Three points can now be made based on this analysis and discussion. First, the operation of any junction (intersection or interchange) controlled by a single controller can be modeled using the CMA approach. This approach defines the operation of the intersection in terms of one key parameter: the sum-of-critical-flow-rates.

Second, cycle length and delay are directly influenced by the sum-of-critical-flow-rates parameter. This one parameter "captures" the effect of signal phasing, volume pattern, volume level, saturation flow rate, and number of lanes. As a result, it can be used to overcome the problems noted in the Background section that have confounded researchers attempting to develop guidelines for identifying conditions where one interchange form will be more efficient than another.

Third, the findings suggest that two actuated intersections that have the same number of phases, phase lost time, and sum-of-critical-flow-rates will have the same cycle length and the same intersection delay, *regardless* of whether they have the same number of lanes, phase sequence, traffic volume level, or turn movement patterns. These three points have been exploited in subsequent sections of this paper to provide a uniform procedure for evaluating alternative interchange forms under a wide range of conditions.

Evaluation Model Analysis Procedures

The interchange evaluation model is based on the methodology described in Chapter 16 of the *HCM* (*13*) for evaluating signalized intersections. This methodology was extended such that the critical movement analysis approach can be used to estimate the sum-of-critical-flow-rates and the delay for both the SPUI/F and the TUDI. Ultimately, the relationship between these two variables will be used to form the basis for the proposed interchange selection and evaluation guidelines. To facilitate the analysis, the evaluation model was implemented in a spreadsheet.

This section provides an overview of the analysis procedures incorporated in the evaluation model. Initially, the signal phase sequences addressed in the procedures are described. Then, there is some discussion of the methods used to determine the change interval duration, phase duration, minimum green interval, maximum green interval, and delay for the various interchange forms and phase sequences. Finally, the assumptions used to define the interchange geometry are identified.

Signal Phase Sequence. Five phase sequences can be evaluated by the analysis tool. The phase sequence used for the SPUI/F is illustrated in Figure 26. The remaining four phase sequences are used for the TUDI. These sequences are illustrated in Figure 28. The phase numbering scheme associated with these sequences is shown in Figure 29. Nelson *et al.* (*23*) provide a thorough discussion of the settings needed to implement these phase sequences with an actuated controller.

The three-phase/single-ring sequence shown in Figure 28a serves two non-conflicting movements with each phase timing function. Each phase ends when both movements have gapped-out (i.e., the time between calls from all detectors on all approaches served exceeds the passage time setting).

The three-phase/dual-ring sequence shown in Figure 28b adds flexibility in phase duration by assigning only one movement to each timing function. The flexibility stems from the fact that phases 2 and 6 can each end at different times, as dictated by their individual traffic demands.   This operation can be more efficient than the sequence shown in Figure 28a by providing additional time to the cross street left-turn phase (1 or 5). This flexibility is not extended to the frontage-road phases (4 and 8) because barriers require that these phases begin and end together.

The four-phase/no-travel-time-interval sequence shown in Figure 28c takes its name from the sequential service it provides to the four external (or through) phases (i.e., 2, 4, 6, and 8). Each internal left-turn phase is served concurrently with two upstream external movements (e.g., 1 with 6 or 8).

a. TUDI - Three-Phase/Single Ring

2+6	1+5	3+7	4+8	Ring 1
				Ring 2

b. TUDI - Three-Phase/Dual Ring

2	1	3	4	Ring 1
6	5	7	8	Ring 2

c. TUDI - Four-Phase/No Travel Time Interval

2	4	1		Ring 1
5		6	8	Ring 2

d. TUDI - Four-Phase/With Travel Time Interval

Total green for southbound

2	4	3 = 4TT	1	Ring 1
7 = 8TT	5	6	8	Ring 2

Figure 28. Alternative ring structures for the TUDI.

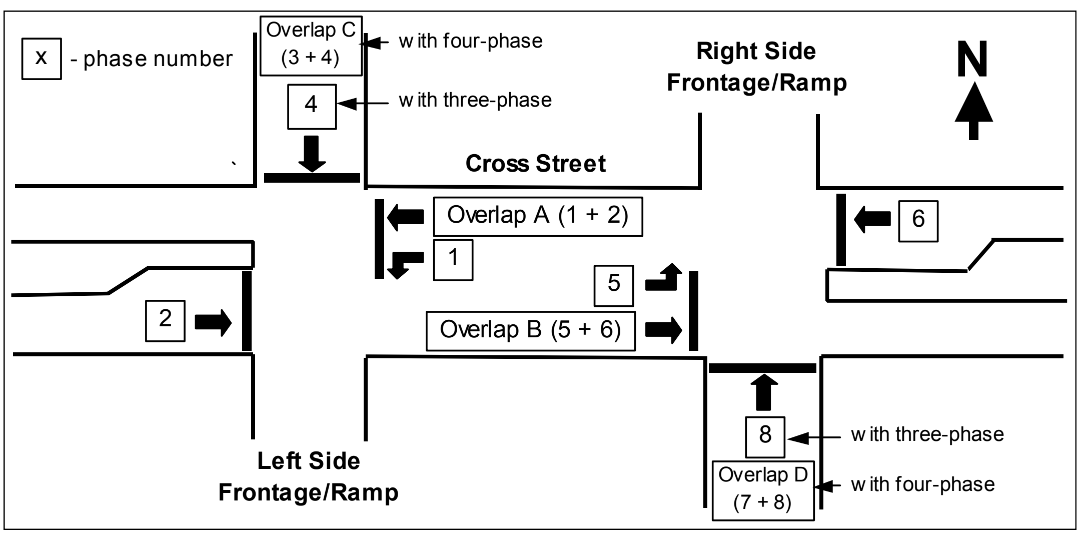


Figure 29. Phase numbering scheme for the TUDI.

The four-phase/with-travel-time-interval sequence shown in Figure 28d adds flexibility to the four-phase sequence shown in Figure 28c. Specifically, it allows for the concurrent service of one frontage-road phase and one upstream through phase (e.g., southbound frontage and westbound through). The concurrent phase (i.e., 3 or 7) follows the primary phase (i.e., 4 or 8) and has a fixed duration equal to the interchange travel time minus two seconds. In Figure 28d, phase 4 is a primary phase and phase 3 is shown as its concurrent phase. Together, these two intervals define the duration of the southbound frontage-road phase. Phases 8 and 7 share a similar relationship for the northbound frontage-road phase.

The four-phase/with-travel-time-interval sequence is devised to provide additional time to the upstream through phase, relative to that provided by the sequence shown in Figure 28c. This phase sequence can be provided with a single controller by using an overlap for each frontage-road phase (e.g., Overlap C for phase pairs 4 and 3; Overlap D for phase pairs 8 and 7) (23). Advance detection on the frontage-road approaches is also needed to effectively use the concurrent phase.

Clearance Interval Duration. The large size of the SPUI/F, relative to the TUDI, creates the need for a lengthy all-red interval at the end of each phase (with the possible exception of the frontage-road through phase). This all-red time translates directly into additional lost time for the corresponding phase. The procedure used to compute the all-red interval duration is described in Chapter 1.

Equilibrium Cycle Length. An actuated intersection will operate at an equilibrium cycle length if it is unconstrained by the maximum green settings. The duration of the equilibrium cycle length is also dictated by the passage time setting on the controller, as it relates to the green extension time following queue clearance. Research by Akcelik (*24*) indicates that the equilibrium cycle length at an intersection with a "snappy" detection design is about equal to the minimum delay cycle length (as predicted by Equation 2). Subsequent research by Bonneson (*25*) indicates that the equilibrium cycle length at a TUDI with a "typical" detection design is about 30 percent longer than that predicted by Equation 2.

The model developed for this research estimates the equilibrium cycle length by multiplying the minimum delay cycle length by a factor between 1.0 and 1.5. The precise value of this factor was determined during the model calibration activity (described in a subsequent section).

Actuated Phase Green Interval Duration. The model developed for this research estimates the green interval duration by applying the "equal-degree-of-saturation" approach, as described in Chapter 16, Appendix B of the *HCM*. This approach allocates a portion of the equilibrium cycle length to each critical phase based on the magnitude of its flow ratio relative to the sum-of-critical-flow-ratios. The green interval durations estimated by this approach are then checked (and adjusted if necessary) to accommodate any minimum or maximum green interval settings, as described in the next two sections.

Minimum Green Interval. The minimum green interval setting varies by interchange form and phasing. For the SPUI/F, a minimum green interval of 8.0 s is used for all phases. For the three-phase TUDIs, the minimum green intervals are defined using the techniques recommended by Bonneson (*25*). In this regard, the frontage-road minimums are set equal to 8.0 s; the cross-street through phase minimums are set equal to the larger of 8.0 s or the travel time minus 2.0 s; and the cross-street left-turn phase minimums are set equal to the larger of 8.0 s or the travel time minus 10 s.

The minimum green intervals for the four-phase TUDIs are defined using the "rules-of-thumb" coined by De Camp (*26*). For the four-phase/no-travel-time-interval sequence, the four external phases (i.e,. 2, 4, 6, and 8) have minimums equal to the larger of 8.0 s or the travel time minus 2.0 s. For the four-phase/with-travel-time-interval, the four external phases have minimums equal to the larger of 8.0 s or twice the travel time less 4.0 s. Both internal left-turn phases have minimums equal to 8.0 s.

Maximum Green Interval. The maximum green interval setting varies by interchange form and phasing. For the SPUI/F and the four-phase TUDIs, a maximum green interval of 50 s is used for all phases. For the three-phase TUID's, the maximum green intervals are defined using the techniques recommended by Bonneson (*25*). Specifically, the maximum green interval for the four external phases (i.e,. 2, 4, 6, and 8) is based on the time required to "fill" the internal storage area with vehicles such that any longer green would be inefficient or unused due to queue spillback. For practical reasons, this maximum value is not allowed to exceed 50 s. Also, the maximum green for the internal left-turn phases (i.e., 1 and 5) is set equal to 50 s.

Movement Delay. The procedure for estimating the average delay for each traffic movement also varies by interchange form and phasing. For all SPUI/F movements and all external TUDI movements (i.e., phases 2, 4, 6, and 8), delay is estimated using Equations 3, 4, and 5. If the phase associated with a given movement is limited by its maximum green interval setting, the incremental delay factor k in Equation 5 is set to 0.5; otherwise, it is set to 0.0. The k values in Exhibit 16-13 of the *HCM* (*13*) are not used because they lack a sensitivity to maximum green duration.

The internal movement delays are estimated from a time-space relationship of arrivals to the internal approaches. For the four-phase TUDIs, it is assumed that delay to the internal left-turn and through movements is negligible. For the three-phase TUDIs, delay to the left-turn movement is estimated as being equal to the opposing through phase duration minus the interchange travel time. For example, westbound left-turn vehicles served during phase 1 incur a delay equal to the eastbound through phase 2 green interval minus the travel time. Similarly, the delay to vehicles that turn left at an upstream frontage road and arrive as a through movement at an internal stop line is estimated as being equal to the concurrent frontage-road phase duration minus the travel time. For example, vehicles turning from the northbound frontage road during phase 8 are delayed the duration of phase 4 minus the travel time.

TUDI Geometry. Two assumptions regarding the TUDI geometry were formed when developing the evaluation model. First, it was assumed that the internal left-turn movements are provided an exclusive storage lane (or lanes) that extends back through the upstream frontage road. Second, on the frontage roads, it was assumed that the inside through lane adjacent to the left-turn lanes could be shared by left-turn and through drivers. The portion of left-turn vehicles using the shared lane is estimated by first converting left-turn vehicles into equivalent through vehicles and then distributing all vehicles (equivalent through and actual through vehicles) evenly among the available lanes.

DATA COLLECTION AND REDUCTION

This section describes the data collection and reduction activities that were undertaken for the purpose of developing a data set that could be used to calibrate the evaluation model. Two types of data were collected: field data and simulation data. Field data were collected at ten interchanges. These data were then supplemented with simulation data to provide a broader range of volume conditions.

The next three subsections provide details of the data collection and reduction activities. Initially, the details of the field study site selection and data collection methods are described.

Then, the field data are reduced and summarized to provide some insight into the overall operation of the interchanges studied. Finally, the simulation data are summarized.

Field Data Collection

Data collection activities were focused on ten interchanges in the Phoenix, Arizona metropolitan area. Five of the interchanges were of the SPUI/F configuration; the other five were of the TUDI configuration. A physical description of each interchange is provided in a subsequent section, as it relates to the geometric data collected during the field studies. Details on the site selection process are described in Chapter 2. The location of these interchanges is identified in Figure 10.

Several types of data were needed to calibrate the evaluation model described in the previous section. These data include the geometric, traffic flow, and traffic control characteristics that serve as "inputs" to the model. They also include the operational characteristics that represent the model "outputs." The specific types of data associated with these characteristics are listed in Table 18.

Table 18. Database elements.

Category	Data Type	Data Collection Method			
		Reduced from Videotape	Field Study	Site Survey	Agency Files
Geometric Characteristics	Number and width of traffic lanes			a	
	Ramp separation distance			a	
	Turn bay length			a	
	Photo log			a	
	Horizontal layout in plan view				a
Traffic Flow Characteristics	Traffic counts by movement	a			
	Platoon arrival type	a			
Traffic Control Characteristics	Speed limit			a	
	Phase sequence		a		a
	Yellow warning interval duration				a
	All-red clearance interval duration				a
Operational Characteristics	Saturation flow rate		a		
	Green interval duration	a			
	Cycle length	a			
	Average delay by movement	a			

Also listed in Table 18 is the method used to collect the corresponding data. Traffic flow, phase duration, and delay data were extracted from videotape recordings of each interchange approach. The methods described in Appendix A, Chapter 16 of the *HCM* (*13*) were followed for the delay study. All videotape-related data were measured during a common 15-minute time interval just following the morning peak period.

Saturation flow rates were sampled in the field. The methods described in Appendix H, Chapter 16 of the *HCM* (*13*) were followed for the saturation flow rate study. Saturation flow rate data were recorded at four interchanges (two SPUI/F and two TUDI). Four traffic movements were included in the study at each interchange, these movements include: frontage-road left-turn, frontage-road through, cross-street left-turn, and cross-street through movement at each interchange. For each movement, saturation flow rate data were collected for 12 to 18 signal cycles.

Prior to conducting field studies, each interchange study site was surveyed to gather relevant geometric characteristics. These data were supplemented with agency records to obtain a complete description of the interchange geometry and traffic control conditions.

Field Data Analysis

This section summarizes the data collected at the interchange study sites. Initially, the geometric characteristics of each interchange are described. Then, traffic flow, traffic control, and performance characteristics are summarized.

Geometric Characteristics

Table 19 lists the geometric characteristics of the study sites. As the information in the table indicates, both interchange types can be used in relatively narrow rights-of-way along the major-road, as reflected in the "ramp separation" data. Although, it should be noted that the TUDIs operate over a wider range of ramp separation distances (i.e., 80 to 150 m (260 to 490 ft)) relative to the SPUI/F (at 69 to 107 m (225 to 350 ft)).

Table 19. Study location geometric characteristics.

Type	Location	Geometry[1]	Ramp Separation[2] m (ft)	Number of Lanes[4]					
				Cross Road			Frontage Road[3]		
				L	T	R	L	T	R
TUDI	I-17/Peoria Avenue	Overpass	110 (360)	1	3	1/0	2/1	1/2	1
	Loop 101/Southern Ave.	Underpass	142 (465)	2	3	1	1	2	1
	Loop 101/Broadway Road	Underpass	149 (490)	2	3	1/0	1	2	1
	I-17/Indian School Road	Underpass	79 (260)	2	4	1	1	3	1
	I-17/Thomas Road	Underpass	94 (310)	1	3	0	1	3/2	0/1
SPUI/F	I-17/Dunlap Avenue	Underpass	91 (300)	2	4/3	2/1	2	3	1
	SR51/Thomas Road	Overpass	69 (225)	2	3	1	2	3	0
	I-17/Northern Avenue	Underpass	88 (290)	2	3/4	1	2	3	0
	Loop 101/Guadalupe	Underpass	107 (350)	2/1	2	1	2	2	1
	I-17/Camelback Road	Underpass	91 (300)	2	3	1	2	3	1

Notes:
1 - Overpass: major road passes over the cross street. Underpass: major road passes under the cross street.
2 - Distance between the two frontage road center lines, as measured along the cross street.
3 - All single-lane, frontage-road left-turn movements are paired with a shared through-and-left-turn lane. All single-lane, frontage-road right-turn movements are paired with a shared through-and-right-turn lane with the exception of I-17 & Thomas Road which does not include a shared lane.
4 - L: left-turn; T: through; R: right-turn. "x/y" notation denotes lane count on opposing approaches.

With regard to cross section, both interchange types tend to have six lanes on each cross street approach. These lanes tend to be distributed as two left-turn lanes, three through lanes (each way), and one right-turn lane on the cross street. In contrast, the TUDI tends to have only four lanes on each frontage-road approach compared to five lanes for the SPUI/F. This tendency is likely due to the fact that the TUDI phasing allows the frontage-road left-turn and through movements to share the inside through lane.

Traffic Flow Characteristics

As noted previously, calibration data were collected during a common 15-minute interval just following the morning peak period. These data included the traffic flow rates (expressed in units of "vehicles per hour"), green interval durations, and motorist delays. The time interval selected for study excluded the peak demand periods because traffic queues extended beyond the videotape recorder's field-of-view during these periods. Accurate estimates of delay require a continuous view of the back-of-queue on each interchange approach during the study interval. The flow rates and study time intervals for interchange study site are listed in Table 20.

Table 20. Study location traffic flow characteristics.

Location	Time Interval[1]	Flow Rate by Approach and Movement,[2,3] veh/h											
		Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
I-17 & Peoria Avenue	9:05 to 9:20	384	76	308	332	84	304	224	492	480	208	452	216
Loop 101 & Southern Ave.	9:15 to 9:30	152	484	96	296	324	260	196	480	88	148	856	416
Loop 101 & Broadway Rd.	9:15 to 9:30	244	224	192	432	304	252	232	412	136	260	336	444
I-17 & Indian School Road	9:10 to 9:25	332	112	148	440	68	308	280	788	244	216	580	204
I-17 & Thomas Road	9:45 to 10:00	276	116	148	380	84	180	172	496	208	164	472	300
I-17 & Dunlap Ave.	8:55 to 9:10	348	136	280	476	192	100	188	584	376	228	444	252
SR 51 & Thomas Road	8:50 to 9:05	340	152	188	280	68	372	148	548	172	160	748	200
I-17 & Northern Ave.	8:20 to 8:35	192	148	188	604	228	200	272	984	268	184	640	292
I-17 & Camelback Rd	9:10 to 9:25	224	64	236	336	100	136	144	632	240	176	460	224

Notes:
1 - Time interval for which volume, phase duration, and delay statistics were collected.
2 - L: left-turn; T: through; R: right-turn.
3 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.

It should be noted that the SPUI/F at Loop 101 and Guadalupe was not included in the operational evaluation because it was still under construction at the time of the field study and only 50 percent of its traffic movements were operational. However, data collected at the other

SPUI/Fs should reflect traffic flow conditions at this site when it is fully open to traffic. Moreover, exclusion of this site from the calibration database should not bias the model output nor limit the model's range of application because any error that may result by this exclusion will be small relative to the overall precision of the evaluation model.

Traffic Control Characteristics

Table 21 lists the traffic control characteristics at the ten study sites. These characteristics include the speed limit, phase sequence, yellow warning interval duration, and all-red clearance interval duration. Collectively, the speed limit ranges from 56 to 72 km/h (35 to 45 mph) at the study sites. Three phase sequences were observed during the field study. All SPUI/Fs used a four-phase dual-ring sequence, as depicted in Figure 26. The TUDIs on I-17 used a four-phase/no-travel-time-interval sequence, as depicted in Figure 28c. The TUDIs on Loop 101 used a three-phase/single-ring sequence, as depicted in Figure 28a.

Table 21. Study location traffic control characteristics.

<table>
<tr><th rowspan="3">Location</th><th rowspan="3">Speed Limit[1] km/h (mph)</th><th rowspan="3">Phase Sequence</th><th rowspan="3">Interval</th><th colspan="8">Interval Duration by Approach and Movement,[2] s</th></tr>
<tr><th colspan="2">Northbound</th><th colspan="2">Southbound</th><th colspan="2">Eastbound</th><th colspan="2">Westbound</th></tr>
<tr><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th></tr>
<tr><td rowspan="2">I-17 & Peoria Avenue</td><td rowspan="2">64 (40)</td><td rowspan="2">4-phase no travel time</td><td>Yellow</td><td colspan="2">3.9</td><td colspan="2">3.9</td><td>3.9</td><td>4.1</td><td>3.9</td><td>4.1</td></tr>
<tr><td>All-Red</td><td colspan="2">1.1</td><td colspan="2">2.1</td><td>1.1</td><td>1.1</td><td>1.1</td><td>1.1</td></tr>
<tr><td rowspan="2">Loop 101 & Southern Ave.</td><td rowspan="2">72 (45)</td><td rowspan="2">3-phase single ring</td><td>Yellow</td><td colspan="2">4.0</td><td colspan="2">4.0</td><td>4.5</td><td>4.5</td><td>4.5</td><td>4.5</td></tr>
<tr><td>All-Red</td><td colspan="2">2.0</td><td colspan="2">2.0</td><td>1.5</td><td>1.5</td><td>1.5</td><td>1.5</td></tr>
<tr><td rowspan="2">Loop 101 & Broadway Rd.</td><td rowspan="2">72 (45)</td><td rowspan="2">3-phase single ring</td><td>Yellow</td><td colspan="2">4.0</td><td colspan="2">4.0</td><td>4.5</td><td>4.5</td><td>4.5</td><td>4.5</td></tr>
<tr><td>All-Red</td><td colspan="2">2.0</td><td colspan="2">2.0</td><td>1.5</td><td>1.5</td><td>1.5</td><td>1.5</td></tr>
<tr><td rowspan="2">I-17 & Indian School Road</td><td rowspan="2">56 (35)</td><td rowspan="2">4-phase no travel time</td><td>Yellow</td><td colspan="2">3.9</td><td colspan="2">3.9</td><td>3.9</td><td>3.9</td><td>3.6</td><td>3.6</td></tr>
<tr><td>All-Red</td><td colspan="2">3.0</td><td colspan="2">3.0</td><td>1.4</td><td>1.4</td><td>1.6</td><td>1.6</td></tr>
<tr><td rowspan="2">I-17 & Thomas Road</td><td rowspan="2">56 (35)</td><td rowspan="2">4-phase no travel time</td><td>Yellow</td><td colspan="2">3.9</td><td colspan="2">3.9</td><td>3.0</td><td>3.6</td><td>3.0</td><td>3.6</td></tr>
<tr><td>All-Red</td><td colspan="2">1.9</td><td colspan="2">2.0</td><td>1.0</td><td>1.3</td><td>1.0</td><td>1.3</td></tr>
<tr><td rowspan="2">I-17 & Dunlap Ave.</td><td rowspan="2">64 (40)</td><td rowspan="2">4-phase dual ring</td><td>Yellow</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td></tr>
<tr><td>All-Red</td><td>4.7</td><td>2.3</td><td>4.7</td><td>2.5</td><td>4.7</td><td>6.5</td><td>4.7</td><td>6.5</td></tr>
<tr><td rowspan="2">SR 51 & Thomas Road</td><td rowspan="2">56 (35)</td><td rowspan="2">4-phase dual ring</td><td>Yellow</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td></tr>
<tr><td>All-Red</td><td>5.0</td><td>2.0</td><td>5.0</td><td>2.0</td><td>4.8</td><td>7.6</td><td>4.8</td><td>7.6</td></tr>
<tr><td rowspan="2">I-17 & Northern Ave.</td><td rowspan="2">64 (40)</td><td rowspan="2">4-phase dual ring</td><td>Yellow</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td></tr>
<tr><td>All-Red</td><td>4.7</td><td>2.5</td><td>4.7</td><td>2.5</td><td>4.7</td><td>6.5</td><td>4.7</td><td>6.5</td></tr>
<tr><td rowspan="2">Loop 101 & Guadalupe</td><td rowspan="2">72 (45)</td><td rowspan="2">4-phase dual ring</td><td>Yellow</td><td>3.6</td><td>3.6</td><td>3.6</td><td>3.6</td><td>4.0</td><td>4.0</td><td>4.0</td><td>4.0</td></tr>
<tr><td>All-Red</td><td>6.4</td><td>2.5</td><td>5.0</td><td>2.5</td><td>6.0</td><td>7.8</td><td>5.0</td><td>7.8</td></tr>
<tr><td rowspan="2">I-17 & Camelback Rd</td><td rowspan="2">56 (35)</td><td rowspan="2">4-phase dual ring</td><td>Yellow</td><td>4.3</td><td>4.3</td><td>4.3</td><td>4.3</td><td>3.9</td><td>3.9</td><td>3.9</td><td>3.9</td></tr>
<tr><td>All-Red</td><td>4.1</td><td>1.9</td><td>4.1</td><td>1.9</td><td>4.5</td><td>5.9</td><td>4.5</td><td>5.9</td></tr>
</table>

Notes:
1 - Speed limit on the cross street.
2 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.

The yellow interval durations listed in Table 21 represent typical values and are fairly consistent across both interchange types. In contrast, the all-red interval durations are quite different among interchange types with the SPUI/F tending to have much larger all-red intervals. The all-red intervals at the TUDI range from 1.0 to 3.0 s. In contrast, the all-red intervals for the SPUI/F (excluding the frontage-road through movement) range from 4.1 to 7.8 s. The all-red intervals for the frontage-road through movement at the SPUI/F are very similar to those for the TUDI. In general, the all-red interval durations for all interchanges are consistent with those obtained from the procedure described in Chapter 1.

Operational Characteristics

The operational characteristics of the study sites are summarized in Tables 22 and 23. Table 22 lists the saturation flow rates measured at two SPUI/Fs and two TUDIs. Table 23 lists the average cycle length, average phase duration, and average delay measured at nine interchanges.

Table 22. Saturation flow rates at selected interchanges.

Type	Location	Saturation Flow Rate,[1] veh/h/ln			
		Cross Street		Frontage Road	
		Left	Thru	Left	Thru
TUDI	I-17 & Peoria Avenue	–	2100	1910	2020
	I-17 & Indian School Rd	1920	1830	1880	2090
SPUI/F	I-17 & Dunlap Ave.	2090	1970	2110	2050
	SR 51 & Thomas Road	2240	1890	2220	–

Notes: 1- "–": data not collected.

The saturation flow rates shown in Table 22 represent averages from a small sample of discharge rates (an average of 15 cycles per movement) at four interchanges. This data was collected to provide a general sense of the saturation flow rates at the study interchanges and to confirm that the interchanges studied had saturation flow rates consistent with those reported by Hook and Upchurch (*12*) and by Poppe *et al.* (*8*).

The saturation flow rates shown in Table 22 indicate that traffic operations in interchange areas are very efficient. In fact, they suggest that the base saturation flow rate at an interchange is larger than 1,900 veh/h/ln (which is the base rate recommended in Chapter 16 of the *HCM* (*13*) for signalized intersection approaches). The data also confirm the trend noted in Chapter 1, that the SPUI/F left-turn movements tend to have base saturation flow rates of 2100 to 2200 veh/h/ln.

The data in Table 23 provide some insight into the operational character of the interchanges studied. In general, the range of cycle lengths and phase durations are similar between the two interchange types. With regard to the delays, it can be seen that the through movement delays are very similar for both interchange types. Although, the TUDIs on Loop 101 have a three-phase sequence and tend to have lower through movement delays than either the TUDIs on I-17 or the SPUI/Fs.

The left-turn movement delays varied significantly among interchange types. The TUDIs on I-17 use the four-phase sequence that all but eliminates delay to the left-turn movements on the internal approaches. However, the left-turns at the TUDIs do experience delay on the external approaches. This delay is the same as that experienced by the adjacent through movements because they are served during a common external phase. The data in Table 23 suggest that the left-turns at TUDIs with a four-phase sequence experience less delay than the left-turn movements at the SPUI/Fs.

Table 23. Study location operational characteristics.

Location	Ave. Cycle Length, s	Characteristic	Approach and Movement[1, 2, 3]							
			Northbound		Southbound		Eastbound		Westbound	
			Left	Thru	Left	Thru	Left	Thru	Left	Thru
I-17 & Peoria Avenue	108	Green Interval, s		—		—	—	21.4	—	22.5
		Delay, s/veh		**31.2**		**42.0**	**0.0**	**43.2**	**0.0**	**34.4**
Loop 101 & Southern Ave.	94	Green Interval, s		—		—	—	36.2	—	36.6
		Delay, s/veh		**24.4**		**23.4**	**29.4**	**12.2**	**25.0**	**18.8**
Loop 101 & Broadway Rd.	94	Green Interval, s		—		—	—	36.6	—	35.7
		Delay, s/veh		**25.1**		**28.6**	**27.3**	**14.8**	**31.6**	**23.1**
I-17 & Indian School Road	105	Green Interval, s		—		—	—	25.3	—	17.6
		Delay, s/veh		**37.2**		**36.5**	**0.0**	**27.9**	**0.0**	**40.9**
I-17 & Thomas Road	105	Green Interval, s		—		—	—	21.8	—	22.8
		Delay, s/veh		**36.5**		**36.8**	**0.0**	**36.0**	**0.0**	**37.9**
I-17 & Dunlap Ave.	104	Green Interval, s	—	—	—	—	10.8	19.4	13.8	22.6
		Delay, s/veh	**40.7**	**34.4**	**34.2**	**34.8**	**38.6**	**24.7**	**47.0**	**34.9**
SR 51 & Thomas Road	90	Green Interval, s	—	—	—	—	8.2	16.3	7.8	18.1
		Delay, s/veh	**39.0**	**45.5**	**33.9**	**33.6**	**56.5**	**26.3**	**58.3**	**33.5**
I-17 & Northern Ave.	124	Green Interval, s	—	—	—	—	15.1	33.5	12.6	30.4
		Delay, s/veh	**61.5**	**44.0**	**39.6**	**36.0**	**48.1**	**36.8**	**57.5**	**39.3**
I-17 & Camelback Rd	104	Green Interval, s	—	—	—	—	10.2	26.3	11.0	30.0
		Delay, s/veh	**51.3**	**29.5**	**36.7**	**36.3**	**47.8**	**46.3**	**36.6**	**25.3**

Notes: 1 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.
2 - "—": data not collected.
3 - Internal delays for Loop 101 & Southern: WB thru - 22.4 s/veh; EB thru - 13.4 s/veh.
Internal delays for Loop 101 & Broadway: WB thru - 12.3 s/veh; EB thru - 11.3 s/veh.

The TUDIs on Loop 101 use a three-phase sequence that creates the potential for two increments of delay to the left-turn movements. The first delay is incurred on the external approaches, similar to that described in the preceding paragraph. The second delay is incurred at the second frontage road junction encountered. The second delay to the cross-street left-turn movements at the Loop 101 TUDIs is noted in Table 23 in the column labeled "Eastbound Left" and "Westbound Left." The second delay to the frontage-road left-turn movements is identified in the table footnote. The data in Table 23 suggest that, the left-turns at the TUDIs with three-phase operation experience delays (both increments combined) at a similar level to those found for the left-turn movements at the SPUI/Fs.

Simulation Data Summary

The CORSIM traffic simulation model (Version 4.3) was used to simulate traffic conditions at three interchanges. The geometry of these interchanges was patterned after three interchanges in the Phoenix area; their locations are identified in Table 24. Collectively, the three simulated interchanges use the three phase sequences observed during the field studies.

Table 24. Simulated interchange traffic flow characteristics.

Type	Location	Flow Rate by Approach and Movement,[1,2,3] veh/h											
		Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
TUDI 3 - phase	Loop 101 & Broadway Rd.	244	224	192	432	304	252	232	412	136	260	336	444
		285	389	213	539	445	403	289	377	140	130	787	684
TUDI 4 - phase	I-17 & Indian School Road	332	112	148	440	68	308	280	788	244	216	580	204
		361	110	168	525	247	140	291	1536	502	243	677	209
SPUI/F	I-17 & Dunlap Ave.	348	136	280	476	192	100	188	584	376	228	444	252
		335	205	383	706	629	123	156	1188	453	68	189	111

Notes: 1 - For each location, the first row of volumes represents an off-peak hour; the second row represents the morning peak hour.
2 - L: left-turn; T: through; R: right-turn.
3 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.

The volumes selected for simulation were chosen to represent typical off-peak and peak hour flow rates at the three interchanges. The volumes used are listed in Table 24. They are based on actual counts obtained during the field studies and reflect morning traffic patterns.

Each interchange was simulated for one hour. An analysis of the delay variability indicated that one hour of simulation was sufficient to estimate the mean delay with an error range of 10 percent (at 90% confidence) for traffic movements of 250 veh/h or more. The error increases to 20 percent for movements of 100 veh/h. The average delays and green interval durations obtained from the simulation are listed in Table 25.

The delay obtained from CORSIM was identified as "queue delay." This delay reflects time spent in queue or moving-up while in a queue. Data published by Zhang *et al.* (*27*) were used to develop a relationship between queue delay and control delay, the latter being the delay computed by the evaluation model. This analysis indicated that control delay could be estimated using the following relationship:

$$d = 0.74 + 1.04 \ d_q \tag{9}$$

where:

d = average control delay, s/veh; and

d_q = average queue delay, s/veh.

Equation 6 is based on 52 delay observations and has a coefficient of determination R^2 of 0.998.

Table 25. Simulated interchange operational characteristics.

<table>
<tr><th rowspan="3">Location</th><th rowspan="3">Ave. Cycle Length, s</th><th rowspan="3">Characteristic</th><th colspan="8">Approach and Movement [1, 2, 3]</th></tr>
<tr><th colspan="2">Northbound</th><th colspan="2">Southbound</th><th colspan="2">Eastbound</th><th colspan="2">Westbound</th></tr>
<tr><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th><th>Left</th><th>Thru</th></tr>
<tr><td rowspan="4">Loop 101 & Broadway Rd.</td><td rowspan="2">54</td><td>Green Interval, s</td><td colspan="2">14.6</td><td colspan="2">14.6</td><td>7.8</td><td>13.1</td><td>7.8</td><td>13.1</td></tr>
<tr><td>Delay, s/veh</td><td colspan="2">15.8</td><td colspan="2">17.8</td><td>4.5</td><td>15.8</td><td>3.7</td><td>16.1</td></tr>
<tr><td rowspan="2">80</td><td>Green Interval, s</td><td colspan="2">23.8</td><td colspan="2">23.8</td><td>10.6</td><td>27.6</td><td>10.6</td><td>27.6</td></tr>
<tr><td>Delay, s/veh</td><td colspan="2">23.3</td><td colspan="2">26.4</td><td>17.5</td><td>19.8</td><td>15.8</td><td>20.9</td></tr>
<tr><td rowspan="4">I-17 & Indian School Road</td><td rowspan="2">80</td><td>Green Interval, s</td><td colspan="2">13.4</td><td colspan="2">16.0</td><td>—</td><td>14.0</td><td>—</td><td>12.6</td></tr>
<tr><td>Delay, s/veh</td><td colspan="2">32.3</td><td colspan="2">31.9</td><td>0.0</td><td>31.1</td><td>0.0</td><td>31.6</td></tr>
<tr><td rowspan="2">104</td><td>Green Interval, s</td><td colspan="2">17.2</td><td colspan="2">22.3</td><td>—</td><td>25.9</td><td>—</td><td>14.9</td></tr>
<tr><td>Delay, s/veh</td><td colspan="2">39.7</td><td colspan="2">37.5</td><td>0.0</td><td>35.1</td><td>0.0</td><td>41.6</td></tr>
<tr><td rowspan="4">I-17 & Dunlap Ave.</td><td rowspan="2">81</td><td>Green Interval, s</td><td>12.1</td><td>10.0</td><td>13.9</td><td>11.9</td><td>9.2</td><td>13.8</td><td>9.9</td><td>15.3</td></tr>
<tr><td>Delay, s/veh</td><td>32.4</td><td>33.7</td><td>33.1</td><td>30.4</td><td>35.7</td><td>30.4</td><td>36.1</td><td>29.6</td></tr>
<tr><td rowspan="2">96</td><td>Green Interval, s</td><td>12.5</td><td>11.5</td><td>22.7</td><td>21.7</td><td>9.7</td><td>23.2</td><td>8.5</td><td>19.4</td></tr>
<tr><td>Delay, s/veh</td><td>40.3</td><td>35.6</td><td>33.9</td><td>31.5</td><td>43.8</td><td>32.0</td><td>41.6</td><td>31.3</td></tr>
</table>

Notes:
1 - For each location, the first row of volumes represents an off-peak hour; the second row represents the morning peak hour.
2 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.
3 - Internal delays for Loop 101, off-peak: WB thru - 5.0 s/veh; EB thru - 4.6 s/veh.
Internal delays for Loop 101, peak: WB thru - 13.0 s/veh; EB thru - 10.3 s/veh.

ANALYSIS

This section describes an analysis of interchange operations using the evaluation model. Initially, the process of calibrating the evaluation model is described. Then, the calibrated model is used to evaluate the operation of several alternative interchange forms and phase sequences.

Model Calibration

Calibration Based on Field Data

The evaluation model was calibrated by comparing the predicted phase durations and delays with those observed during the field study. The calibration consisted solely of adjusting the multiplier used to inflate the minimum-delay cycle length C_o. This cycle length is then distributed among the signal phases using the "equal-degree-of-saturation" approach with the resulting phase durations used to estimate movement and interchange delay. As noted previously, research by Akcelik (*24*) and Bonneson (*25*) indicates that the equilibrium cycle length for an actuated intersection or interchange tends to be slightly larger than the minimum-delay cycle length.

The calibration process was based on a "trial-and-error" approach of adjusting the cycle-length multiplier. With this approach, the cycle-length-multiplier was adjusted until the "best fit" was obtained between the measured and predicted green interval durations as well as the measured and predicted delays. This process was repeated for each interchange site.

The results of this process indicated that a multiplier of 1.2 (i.e., $C = 1.2\ C_o$) was appropriate for the SPUI/F. In contrast, a multiplier of 1.5 was found to yield the best fit for the TUDIs on I-17. The larger value for these TUDIs is rationalized by the additional detectors on the internal interchange approaches, relative to the SPUI/F. These detectors allow interchange vehicles to extend the green a second time (i.e., once on the external approach and then again on the internal approach). These added extensions tend to result in longer phase durations and thus longer cycle lengths for the TUDI, relative to the SPUI/F.

The calibration effort for the Loop 101 TUDIs did not include a search for an equilibrium cycle length because these locations were part of a coordinated signal system and have fixed cycle lengths. Thus, the calibration effort for these two interchanges was simplified by substituting the observed cycle length for the computed minimum-delay cycle length.

The quality of fit obtained by the calibrated model to the field data is shown in Figures 30 and 31 for the SPUI/Fs and the TUDIs, respectively. The trends shown in these figures indicates that the model is able to predict the green interval duration with negligible error. The delay predictions are also relatively good in light of the fact that the measured delays are based on 15-minute intervals (which tend to have about twice the random error of 1-hour data).

Calibration Based on Simulation Data

A second calibration effort was undertaken using simulation data. All interchanges were simulated in isolation with other signalized intersections and, thus, were allowed to converge to their equilibrium cycle length during the simulation. The cycle-length multiplier was again varied on an interchange-by-interchange basis. It was found that the best-fit multipliers were 0.9 and 1.2 for the SPUI/F and TUDIs, respectively. The tendency for the SPUI/F multiplier to be lower than that for the TUDI is consistent with the finding from the field data calibration.

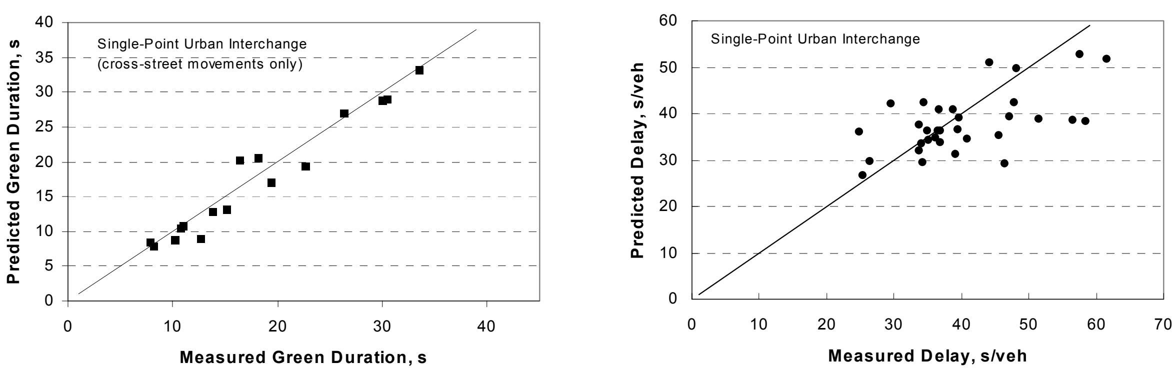


Figure 30. Comparison of model predictions with field data from the SPUI/F sites.

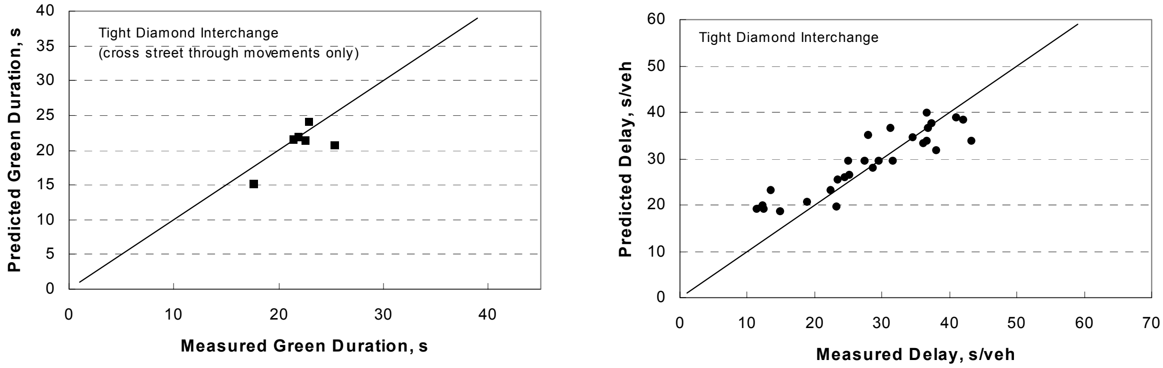


Figure 31. Comparison of model predictions with field data for the TUDI sites.

Both of the best-fit multipliers were found to be smaller than their field-data-based counterpart by a magnitude of 0.3. It is believed that this trend is due to the more efficient nature of the simulated detectors. While the simulated detection design was patterned after that used at the interchanges, it was noted that the simulated detector operation was more precise in recording vehicle presence. Specifically, the simulated detector placed (and dropped) its call immediately before (and after) the vehicle crossed (and cleared) the detector whereas actual loop detectors have a sensing zone that extends about a meter before and after the loop location, effectively extending the call duration and, thus, the phase duration.

The quality of fit obtained by the calibrated model to the simulation data is shown in Figures 32 and 33 for the SPUI/F and TUDIs, respectively. The trends in these two figures indicate that the calibrated model is able to accurately predict the average green interval duration and the average delay. The variability of the simulation data is reduced, relative to the field-data, primarily because there are about four times as many observations underlying each data point.

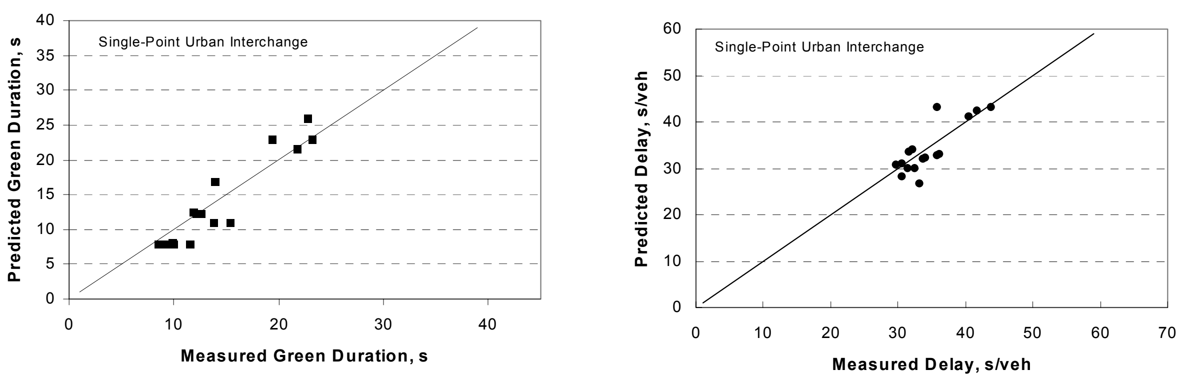


Figure 32. Comparison of model predictions with simulation data for the SPUI/F.

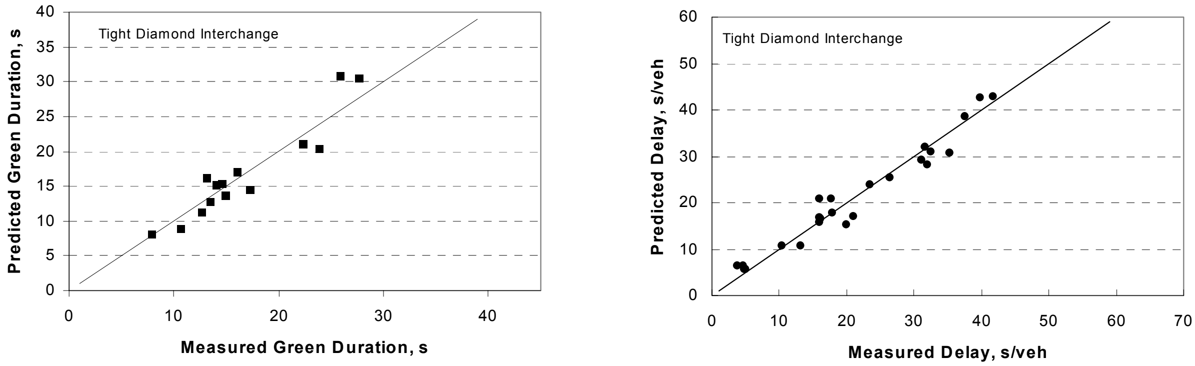


Figure 33. Comparison of model predictions with simulation data for the TUDI.

Interchange Evaluation

Evaluation Scenarios

This section describes an application of the calibrated evaluation model to the evaluation of alternative interchange forms and phase sequences. This evaluation was based on a factorial design of various combinations of volume, cross section, and cross-street speed. For each combination, the sum-of-critical-flow-ratios and movement delay were recorded. These data were then used to develop interchange-form-and-phasing characteristic curves.

The delay statistic computed for this evaluation is analogous to "intersection delay" in that it represents an overall average delay for the interchange. It is computed using the following equation:

$$d_I = \frac{\sum (d_i \; v_i) + \sum (d_j \; v_j)}{\sum v_i} \tag{10}$$

where:

d_I = interchange delay, s/veh;
d_i = average control delay for external movement i, s/veh;
d_j = average control delay for internal movement j, s/veh;
v_i = flow rate for external movement i, s/veh; and
v_j = flow rate for internal movement j, s/veh.

External movements represent all movements that enter the interchange for the first time. Internal movements represent all movements that encounter a second stop line within the interchange. SPUI/Fs have only external movements. TUDIs have internal left-turn and through movements on both of the cross-street approaches located between the frontage road junctions. The equation for interchange delay is defined such that its denominator is constant for all interchange forms and represents the total volume entering the interchange. As a result, this delay statistic can be used to compare alternative interchange forms without bias.

To facilitate the examination of critical flow ratio on interchange delay, a total of 30 volume scenarios were developed. These volumes are listed in Table 26. They were patterned after the volumes used by Garber and Smith (*3*) in their comparison of SPUI and TUDI operations. The set of volumes consists of three groups of ten volume patterns. One group represents low volumes; another represents moderate volumes, and a third represents high volumes. Within each group, there are five volume patterns for the cross-street combined with two-volume patterns for the frontage road. The cross-street volume patterns have the following attributes:

- Scenarios 1 & 2: equal through volumes and equal left-turn volumes.
- Scenarios 3 & 4: unequal left-turn volumes and unequal through volumes where the heavier through volume opposes the heavier left-turn volume.
- Scenarios 5 & 6: unequal left-turn volumes and unequal through volumes where the heavier through volume opposes the lighter left-turn volume.
- Scenarios 7 & 8: equal left-turn volumes and unequal through volumes.
- Scenarios 9 & 10: unequal left-turn volumes and equal through volumes.

The frontage road volumes within each cross-street scenario pair were also varied. The first scenario of a pair has frontage road volumes that reflect unequal left-turn and unequal through volumes where the heavier through volume opposes the lighter left-turn volume. The second scenario of a pair has the frontage road volumes that reflect unequal left-turn volumes and unequal through volumes where the heavier through volume opposes the heavier left-turn volume.

In addition to volume level and volume pattern, interchange geometry was varied for the evaluation. Two geometry scenarios were developed based on typical interchange geometrics in the Phoenix area. These two scenarios are identified in Table 27. One scenario represents a "large" or high-type interchange; the other has fewer cross-street through lanes and fewer frontage-road left-turn lanes.

Finally, the evaluation was conducted for a range of ramp separation distances. The distances considered include 60, 90, and 150 m (200, 300, and 500 ft). These distances were selected to explore the effect of interchange travel time on overall operations.

Table 26. Turn movement volume scenarios.

Scenario Level	Scenario No.	Northbound L	Northbound T	Northbound R	Southbound L	Southbound T	Southbound R	Eastbound L	Eastbound T	Eastbound R	Westbound L	Westbound T	Westbound R	Critical Sum, veh/h
		Flow Rate by Approach and Movement, [1, 2] veh/h												
High	1	400	400	350	600	600	400	800	1000	300	800	1000	300	2800
	2	300	600	350	700	300	400	800	1000	300	800	1000	300	3100
	3	400	400	350	600	600	400	400	1200	300	1000	800	300	3200
	4	300	600	350	700	300	400	400	1200	300	1000	800	300	3500
	5	400	400	350	600	600	400	400	800	300	1000	1200	300	2800
	6	300	600	350	700	300	400	400	800	300	1000	1200	300	3100
	7	400	400	350	600	600	400	800	800	300	800	1200	300	3000
	8	300	600	350	700	300	400	800	800	300	800	1200	300	3300
	9	400	400	350	600	600	400	400	1000	300	1000	1000	300	3000
	10	300	600	350	700	300	400	400	1000	300	1000	1000	300	3300
Moderate	1	350	350	225	450	450	300	475	795	215	475	795	245	2070
	2	250	475	225	550	225	300	475	795	215	475	795	245	2295
	3	350	350	225	450	450	300	350	825	215	550	850	245	2175
	4	250	475	225	550	225	300	350	825	215	550	850	245	2400
	5	350	350	225	450	450	300	350	850	215	550	825	245	2200
	6	250	475	225	550	225	300	350	850	215	550	825	245	2425
	7	350	350	225	450	450	300	475	625	215	475	1050	245	2325
	8	250	475	225	550	225	300	475	625	215	475	1050	245	2550
	9	350	350	225	450	450	300	350	800	215	575	800	245	2175
	10	250	475	225	550	225	300	350	800	215	575	800	245	2400
Low	1	300	300	100	300	300	200	150	590	130	150	590	190	1340
	2	200	350	100	400	150	200	150	590	130	150	590	190	1490
	3	300	300	100	300	300	200	300	450	130	100	900	190	1800
	4	200	350	100	400	150	200	300	450	130	100	900	190	1950
	5	300	300	100	300	300	200	300	900	130	100	450	190	1600
	6	200	350	100	400	150	200	300	900	130	100	450	190	1750
	7	300	300	100	300	300	200	150	450	130	150	900	190	1650
	8	200	350	100	400	150	200	150	450	130	150	900	190	1800
	9	300	300	100	300	300	200	300	600	130	150	600	190	1500
	10	200	350	100	400	150	200	300	600	130	150	600	190	1650

Notes:
1 - L: left-turn; T: through; R: right-turn.
2 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.

Table 27. Interchange geometry scenarios.

Scenario		Number of Lanes by Approach and Movement [1, 2, 3]												Total
Level	No.	Northbound			Southbound			Eastbound			Westbound			
		L	T	R	L	T	R	L	T	R	L	T	R	
High	1	2	2	0	2	2	0	2	3	1	2	3	1	20
Low	2	1	2	0	1	2	0	2	2	1	2	2	1	16

Notes:
1 - L: left-turn; T: through; R: right-turn.
2 - East and westbound approaches are on the cross street; north and southbound approaches are on the frontage roads.
3 - Inside through lane on each TUDI frontage-road approach is shared with the left-turn movement.

Evaluation Results

The evaluation consisted of recording the sum-of-critical-flow-ratios and interchange delay for each volume scenario, geometry, and ramp separation distance. These data were then used to develop characteristic curves for each interchange type and phase sequence. The results of this development are shown in Figure 34 for a 60 m (200-ft) ramp separation distance.

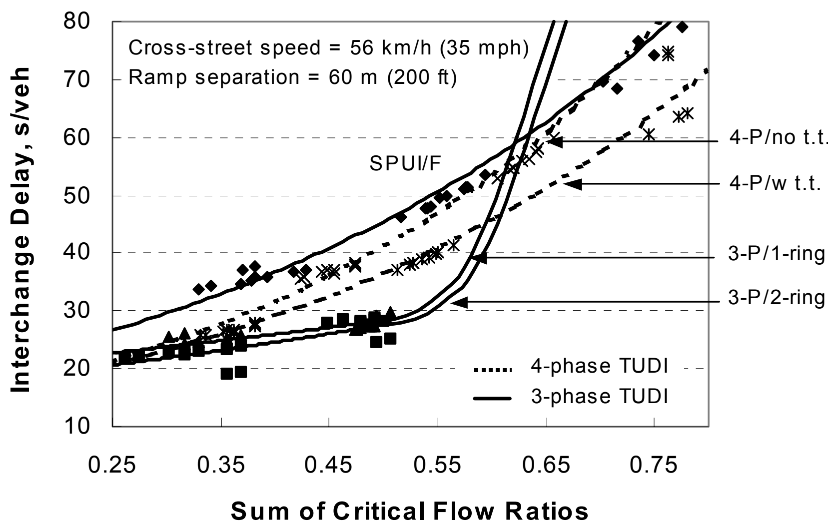


Figure 34. Effect of critical flow ratio on interchange delay for 60 m (200-ft) ramp separation.

The trends in this figure offer strong evidence that a unique characteristic curve exists for every interchange form and phase sequence combination. The trends lines represent the best-fit to the corresponding data points shown. Five curves are shown in the figure; one for each of the following interchange forms and phase sequences:

i Single point urban interchange with frontage roads and dal-ring control (SPUI/F).
i TUDI with four-phase/no-travel-time-interval control (4-P/no t.t.).
i TUDI with four-phase/with-travel-time-interval control (4-P/w t.t.).
i TUDI with three-phase/single-ring control (3-P/1-ring).
i TUDI with three-phase/dual-ring control (3-P/2-ring).

Much of the discussion that follows will compare the five alternatives in terms of the delay incurred for a given sum-of-critical-flow-ratio. However, it must be remembered that the same entering volume will yield slightly different sum-of-critical-flow-ratios for different interchange forms or for the same interchange but with different phase sequences. For example, Scenario 10 for the Ahigh-volume@ condition in Table 26 results in the following critical flow ratios: 0.58, 0.62, 0.52, 0.49, and 0.48 for the five alternatives (in the order listed above).

The fact that one alternative has a trend line above another in Figure 34 does not guarantee that the associated interchange will operate with higher delay, given the same volume level. However, experience with the data indicate that significant delay differences (say, 10-s or more) among trend lines at a given sum-of-critical-flow-ratio are strong indicators that one alternative is likely to operate less efficiently than another, regardless of any subtle difference in their critical flow sum. In short, the best method of interpreting the trends in Figure 34 would be to determine the sum-of-critical-flow-ratios for each alternative being considered and find its corresponding delay using the appropriate trend line. However, general tendencies can be discerned about the relative performance of any two alternatives if their characteristic curves differ by more than 10 s.

The trends in Figure 34 indicate that the SPUI/F and TUDI with four-phase/no-travel-time-interval control (4-P/no t.t.) operate with similar levels of delay when ramp separation distances are about 60 m (200 ft). The TUDI with four-phase/with-travel-time-interval control (4-P/w t.t.) will likely operate with less delay over the full range of volumes. For low volume levels, the trends indicate that the TUDI with three-phase control will operate with equal (or less) delay than the TUDI or SPUI/F. However, the queue spillback associated with the short internal storage area will cause oversaturation and significant delay for the higher volume levels. For this reason, the TUDI with three-phase operation is typically avoided for short ramp separation distances.

The effect of increasing ramp separation distance was also examined. The results of this examination are shown in Figures 34, 35, and 36. This comparison indicates that the differences between the SPUI/F and TUDI become more distinct as the distance between the frontage roads increases. For the SPUI/F, it can be seen that delays increase with increasing distance.

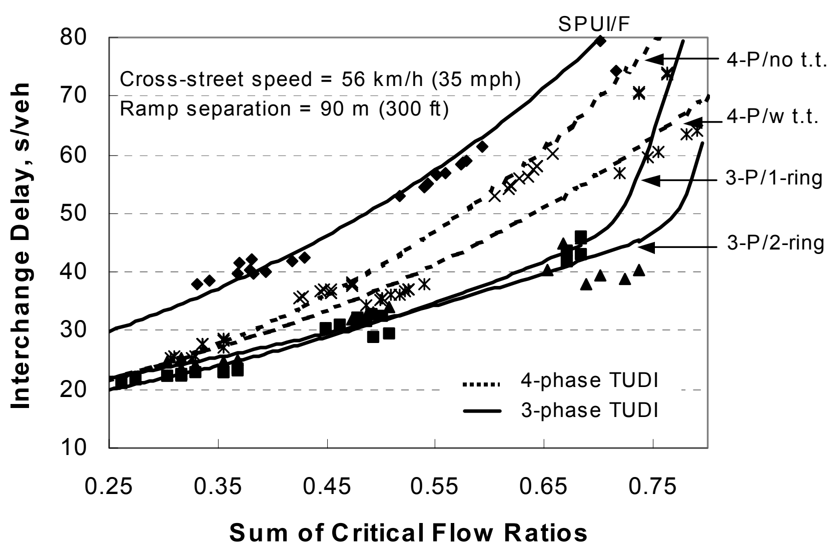


Figure 35. Effect of critical flow ratio on interchange delay for 90 m (300 ft) ramp separation

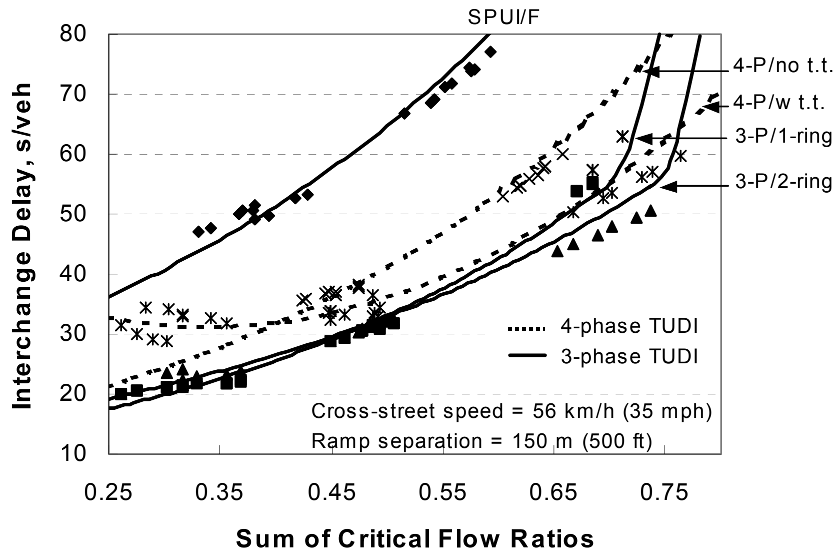


Figure 36. Effect of critical flow ratio on interchange delay for 150 m (500 ft) ramp separation

For the TUDIs with four-phase operation, there is negligible effect of ramp separation distance on delay. One exception is that the TUDI with four-phase/with-travel-time-interval control increases delay for the lower volume scenarios with increasing distance. This increase in delay is due to the lengthy minimum green intervals inherent to this control mode.

For the TUDIs with three-phase operation, the primary effect of distance is that the critical flow ratio associated with a transition from under to oversaturated operation increases. Specifically, a critical flow ratio of 0.5 or less is necessary to prevent oversaturation when the ramp separation distance is 60 m (200 ft). However, this limiting ratio increases to about 0.7 for distances of 150 m (500 ft). It is also noted that the delay at TUDIs with three-phase operation and higher volume increases with increasing ramp separation distance. Although, three-phase operation always has lower delay than four-phase operation, provided that the sum-of-critical-flow-ratios is less than the oversaturation level.

The effect of geometry, in terms of the number of lanes provided, was also investigated during this evaluation. Specifically, the effect of volume and ramp separation distance on delay was evaluated for both lane configurations described previously in Table 27. The results of the analysis indicated that geometry had no effect on the characteristic curves. In fact, a regression analysis indicated that the difference in delay between the two geometric variations, for a given volume and distance combination, was not significantly different from zero (with a 95 percent confidence level). This conclusion adds further support to the use of characteristic curves to identify efficient interchange forms and phasing alternatives for a wide range of volume conditions and lane configurations.

SUMMARY

The findings from the interchange evaluation indicate that a sound, rational approach to interchange form selection and operational evaluation is feasible using the characteristic relationship between interchange delay and the sum-of-critical-flow-ratios. The use of these curves can provide a solution to the challenging question of, "Which interchange form is most efficient?" Previous research projects directed at answering this question have produced guideline statements that can be characterized as vague, subjectively based, or difficult to apply.

The sum-of-critical-flow-ratios is a unique parameter that can combine an infinite number of interchange volume level, volume pattern, and geometry combinations into a single value. Furthermore, the analysis presented in the previous section indicates that this parameter has a unique delay relationship based on interchange type and phase sequence. These attributes can be exploited to develop a family of characteristic curves for a range of ramp separation distances that collectively can be used to identify the most efficient interchange alternative.

The characteristic curves could be used for planning-level and operations-level evaluations. At the planning level, it would be sufficient to identify and sum the critical movement lane volumes and then divide this total by a representative saturation flow rate to obtain the sum-of-critical-flow-ratios. At the operations level, the critical movement flow ratios would be computed and summed. This latter application would incorporate more detail regarding the saturation flow rate of the individual movements.

The only limitation of this approach is that it assumes that a single, actuated controller is used to control the interchange phase sequence. The use of two controllers (i.e., one for each frontage road junction) or the use of pretimed phases would violate key assumptions related to phase time allocation. Such deviations may blur the relationship between the sum-of-critical-flow-ratios and delay.

CHAPTER 5
A COST COMPARISON BETWEEN
THE SINGLE POINT URBAN INTERCHANGE WITH FRONTAGE ROADS
AND THE TIGHT URBAN DIAMOND INTERCHANGE

INTRODUCTION

This chapter compares cost components of the single-point urban interchange with frontage roads (SPUI/F) with that of the tight urban diamond interchange (TUDI). The primary focus is to compare the planned cost of each type with the actual construction cost. Additionally, road user costs are included to supplement the construction and right-of-way costs.

This information is particularly relevant because the current ADOT process for selecting an interchange type is to generally select the least costly alternative from among those that provide an acceptable operational level.

COST COMPONENTS

This chapter will present and evaluate cost components of the SPUI/F and TUDI. It was originally intended to document right-of-way, construction and road user costs for each of the ten interchanges being studied in this research. These ten interchanges are listed in Table 28 with the interchange type, whether overpass or underpass and ramp separation distance. This was to be accomplished by comparing the following cost components:

- Right-of-Way Costs
- Construction Costs, and
- User Costs.

Table 28. Study Locations

Interchange	Type	Geometry	Width[1]
I-17/Peoria Avenue	TUDI	Overpass	110 m (360’)
Loop 101/Southern Avenue	TUDI	Underpass	140 m (465’)
Loop 101/Broadway Road	TUDI	Underpass	150 m (490’)
I-17/Indian School Road	TUDI	Underpass	80 m (260’)
I-17/Thomas Road	TUDI	Underpass	95 m (310’)
I-17/Dunlap Avenue	SPUI/F	Underpass	90 m (300’)
SR 51/Thomas Road	SPUI/F	Overpass	70 m (225’)
I-17/Northern Avenue	SPUI/F	Underpass	88 m (290’)
Loop 101/Guadalupe Road	SPUI/F	Underpass	107 m (350’)
I-17/Camelback Road	SPUI/F	Underpass	90 m (300’)

[1]distance measured between center of frontage roads

Cost Data Perceived to be Available

Four types of information were believed to be readily available for most, if not all, of the ten interchanges being evaluated.

1. Planning documents (design concept reports, feasibility studies, preliminary engineering reports) are developed for most projects and were expected to provide the estimated project costs during the preliminary design phase of the projects.

2. Engineer's estimates are prepared for all projects before they are advertised for bidding, and are based on the quantities determined during the design process. The engineer's estimates were expected to provide a more refined estimate of the project cost than the planning documents.

3. Bid prices received from contractors indicate the price for which the project will be constructed. Bid prices can vary from the engineer's estimate based on the contractor's bid unit prices.

4. Cost records are maintained by ADOT based upon the payments for the work completed. These cost records are kept by project and indicate the type of work (design engineering, right-of-way, utilities, construction) and the date payment was made. The project cost can increase or decrease as the result of change orders issued during the construction of the project.

Cost Data Found to be Available

Although numerous efforts were made by those involved in the project (both the client and the consultant), the only cost information obtained was the following:

Planning documents. Two design concept reports and one preliminary engineering report were located.

- *Design Concept Report (1995) Dunlap Avenue Traffic Interchange, (28)*
- *Design Concept Report (1995) Northern Avenue Traffic Interchange, (29)*
- *Preliminary Engineering Report, Phoenix-Cordes Junction Highway, Indian School Road Traffic Interchange, (30)*

Engineers Estimates and Bid Prices

Bid prices were obtained for three projects, which also included the engineer's estimates. The bid prices for all three projects have the same heading – *Tabulation of Bids, ADOT, Intermodal Transportation Division, Contracts & Specifications Section.* The three project were:

- Project No: IM*17-1-(334), Contract No: 1998079, Location: I-17, Thomas Road - Peoria Avenue. (This project was let as a design-build contract.)

- Project No: RAM 600-1-545, Contract No: 1996130, Location: Price, Baseline-Guadalupe Road. (This project included the work from Baseline south for 1.2 miles.)

- Project No: AC NH 17-1-(320), Contract No. 1996135, Location: Northern AV TI & Dunlap AV TI. (This project combined the work at two interchanges.)

Cost Records

ADOT provided spreadsheets indicating cost information broken down by phase and fiscal year. The database went to 1989, so charges could not be provided before that year. Information was provided for charges in four basic categories – construction, right-of-way, utilities, and design.

Information was provided for seven project numbers.

HO186	Thomas Road Underpass
HO189	Peoria Avenue TI (FY 89, 90) Total Amount $99.54
HO 836	Squaw Peak, Glendale Ave. to Northern Ave., (FY 89 - 99)
HO195	Indian School Road TI, (FY 90 - 00)
H4478	Thomas Rd. - Peoria Ave., (FY 98 - 01)
	Glendale Ave. - Dunlap Ave. (FY 97 - 01)
H2402	Thomas Rd. - Thunderbird Rd., Ph 1, (FY 91-95)
	Dunlap-Beardsley, (FY 94 - 99)
	McDowell Rd. - Dunlap Ave. (FY 95 - 00)
	Northern Ave. & Dunlap Ave. (FY 95 - 01)
	Northern Ave. TI & Dunlap Ave. TI (FY 97 - 01)
	Thomas Rd. - Dunlap Ave. (FY 94 - 00)
H2224	Price, Baseline-Guadalupe Rd., (FY 96 - 01)
	Price, Guadalupe Rd. - Elliot Rd. (FY 96 - 98)
	SR 101L, Baseline - Guadalupe (FY 95 - 00)

All of the information is computerized and charges are made against the project numbers. Unfortunately, many of the project descriptions to which the project numbers are assigned are not for individual interchanges.

For example, the project descriptions, particularly along I-17, do not specifically relate to the interchanges. The largest project, with construction charges of $90,148,880, was design-build, and the location was I-17, Thomas Road to Peoria Avenue. Another project with construction charges of $9,391,866 was from McDowell to Dunlap. A project from Dunlap to Beardsley had construction charges of $8,632,848. It can be noted that all of these projects have termini that are several miles apart.

There were some projects that had right of way and construction charges that were within the limits of some of the above projects. These included:

H2402: Northern Ave. TI & Dunlap TI $19,875,009.88
H0195: Indian School $7,288,873.24 (R/W); $8,802,328.37 (Construction)
H0186: Thomas Underpass $879,379.19

Other projects appeared to have good cost information, although the scope of work was not clear. For the project identified as Price Freeway, Baseline – Guadalupe, the construction charges were $20,028,004.24. This would appear to be the project which the bid tab identified as beginning south of Baseline Road and extending south for 1.2 miles, and for which the low bid was $17,130,199.17. However, it appears the project was for more than the interchange.

After reviewing the information, it was decided that the best approach would be to work from the information included in the design concept reports, and compare the DCR planned cost with the engineer's estimate (where available) and the actual cost.

Comparing Costs of Interchanges

It should be noted that there are numerous unknown factors which can make cost comparisons of constructing interchanges at different locations difficult, if not impractical. The following are some of the factors which should be considered related to cost comparisons.

A. Utilities – Were utility relocations included in the construction contract?
What utilities were involved in the project?
What was the scope of the utility relocation work?

B. Right-of-Way – How much R/W was required?
What type of R/W was it? (Commercial or Residential)
What was the cost of the R/W?
Were relocations involved?
Did the project have an impact on driveways?

C. New Construction or Reconstruction?
Was some of the R/W already owned by ADOT?
Was it possible to reuse some of the existing roadway elements?

D. Did the project include provisions for the future?
Example -- Did the structure width provide for future lanes?

E. Did different environmental factors require roadway elements such as sound walls?

F. Did traffic considerations such as the time the road is closed result in the selection of more costly but less time consuming roadway elements?

G. Were geotechnical conditions similar or did one location require more expensive footings; or a thicker pavement section?

H. Were drainage conditions similar, including such items as pumping stations, or even the amount of pipe required for drainage?

I. Were the years of construction comparable? Although cost indices can help -- the variations in the prices of steel, concrete, and asphalt are sometimes based more on competition than on inflation.

J. Were the bidding conditions imposed on the contractor similar, including such items as insurance requirements, time allowed for construction, and who performed the construction surveying?

K. Were specific advantages or disadvantages considered in the project. For example, it is indicated in the planning reports that the wider bridge required for the SPUI provides for better handling of traffic during both construction and maintenance operations.

It is believed that the approach used acknowledges these issues by comparing planning level cost estimates with actual construction cost at the same interchange. This is the primary intent of the comparison rather than the total construction cost of one type as compared with another.

PLANNING LEVEL COST ESTIMATE COMPARISON WITH ACTUAL COSTS

It was desired to compare the planning level cost estimates with the actual construction cost of the two interchange types. For this comparison, Design Concept Reports (DCR) or Preliminary Engineering (PE) Reports were obtained for the following interchanges:

- I-17 and Dunlap (SPUI/F)
- I-17 and Northern (SPUI/F)
- I-17 and Indian School (TUDI)

DCR Cost Estimates: I-17 at Dunlap and Northern Avenues.

Separate Design Concept Reports were prepared for the Dunlap Avenue and Northern Avenue interchanges on I-17. Both reports include thorough analyses of alternate interchange types. The reports include cost estimates of right-of-way and construction, and indicate the reasons for recommending the SPUI/F.

The following summaries of the itemized cost estimates in the reports reflect the major anticipated cost differences between TUDI and SPUI/F interchanges at the two locations. At both interchanges, the estimated right-of-way cost of the TUDI was over $2,000,000 higher than for the SPUI/F, which was the primary factor in the selection of the SPUI/F. The lower right-of-way cost was the result of the narrower approach width with the SPUI/F resulting from its alignment of cross road left turn lanes. It can be noted that the wider bridge required for the SPUI/F causes the estimated structure cost to be higher, although the smaller costs of some other items (earthwork, pavement) tend to reduce the overall cost differential.

Item	Northern Avenue TUDI	Northern Avenue SPUI/F	Dunlap Avenue TUDI	Dunlap Avenue SPUI/F
Earthwork	$212,000	$184,200	$206,400	$172,000
Pavement	605,000	445,000	562,650	480,300
Drainage	1,475,000	1,475,000	1,475,000	1,475.000
Structures	1,112,500	1,593,800	1,087,000	1,570,000
Traffic Control/Lighting	610,000	610,000	690,000	610,000
Landscaping	150,000	150,000	150,000	150,000
Utility Relocation	800,000	800,000	800,000	800,000
Subtotal	4,964,600	5,258,000	5,171,050	5,257,300
Miscellaneous*	1,539,026	1,629,980	1,603,026	1,629,763
Roadway and Structures Subtotal	6,503,626	6,887,980	6,774,076	6,887,063
Contingency (15%)	975,544	1,033,197	1,016,111	1,033,059
Right-of-Way	6,440,000	4,370,000	4,531,500	2,305,000
Total	13,919,170	12,291,177	12,321,687	10,225,122

*Mobilization, Quality Control, Water Supply and Dust Palliative, Erosion Control, Maintenance/ Protection of Traffic

Cost Comparison: Dunlap & Northern Interchanges Bid Price and Actual Cost

The description of the construction project of the work was "The proposed (*sic*) Replace Bridges at Two T.I.'s work is located in Maricopa County on Interstate 17 in the City of Phoenix, at Northern Avenue (Milepost 206.88) and Dunlap Avenue (Milepost 207.93). The proposed work includes reconstruction of existing traffic interchanges at Northern Ave. and Dunlap Ave., and construction of a new pump station at Northern Ave. The work consists of removal of existing and construction (*sic*) new bridges at Northern Avenue and Dunlap Avenue: cross-road, frontage-road and ramp paving (concrete and asphalt); installation of a drainage system including the new pump station; decommissioning and removal of two existing pump stations; construction of retaining walls; miscellaneous utility relocation; installation of traffic signals, pavement markings, signing and lighting; landscaping; and other incidental work."

The DCR cost estimate for Roadway and Structures subtotal for the two interchanges was $13,775,043. The engineer's estimate for the work was $15,194,000, and the low bid was $16,594,691.40, which was 9.22% ($1,400,691.40) over the Department's estimate. Bids were made on approximately 206 items, and there is no indication on the bid tabulation of whether an item is related to an individual interchange or to both locations. Some of the major bid items (for both locations) which might not be directly comparable to other locations, with the low bidder's amount are:

Removal of structures and obstructions	$500,000
Remove Bridge	350,000
Removal of pump station	50,000
Pipe, Reinforced Concrete (various classes & sizes)	1,555,850
Generator & Control Bldg & Wet Well & Pump Station Site	400,000
Pump Station Equipment and Controls	400,000
Continuous Pump Test	10,000
Pump Station Piping and Piping Accessories	30,000
Pump Station Electrical System	200,000
Changeable Message Sign	288,000
Flagging Services (Uniformed Officer)	212,160
Mobilization	1,746,000
Construction Surveying and Layout	170,000
Lump Sum Structure	1,955,702
Lump Sum Structure	2,266,850

As indicated earlier, the interchanges on I-17 at Northern and Dunlap Avenues were combined into one project for bidding and construction and the total costs are indicated below. The engineer's estimate and the low bid include the same items and should be comparable. However, the actual cost from the records includes additional expenditures such as the direct charges by ADOT personnel against the project for engineering and administration, equipment rental, overruns or underruns on the project, and change orders.

Engineer's estimate	$15,194,000.00
Low bid	$16,594,691.40
Actual cost from records	$19,875,009.88 (Construction)
Actual cost from records	$ 4,831,504.18 (Right-of-Way)

Interchange at I-17 and Indian School Road

A Preliminary Engineering Report (*30*) was prepared in May 1988 which evaluated various interchange types at this location. This report recommended an urban compressed diamond interchange (what we are now calling a SPUI/F) over the conventional diamond (which is now referred to as the TUDI) and a three-level diamond. According to this report ". . . it is concluded that although the three-level diamond is preferable to the other two designs from a traffic operations viewpoint, fund limitations preclude recommending its adoption as the "preferred" design. Since the urban and conventional diamond offer comparable traffic service, but the urban design should provide somewhat better traffic operation when completed, <u>it is recommended that the urban (compressed diamond) be utilized for the improvement of the Indian School Road, T.I."</u> (5, p. 148) It is unclear why this recommendation was not followed and the TUDI was constructed, however a comparison of the Preliminary Engineering cost estimates for the TUDI contained in the report was used for comparison with the actual construction cost.

Item	Preliminary Engineering Report Cost	Actual Cost*
Design	----------	$1,223,716
Right-of-Way	$7,491,000	7,284,922
Utilities	1,000,000	494,212
Construction	8,028,000	6,585,671
Pump Station	1,136,000	2,216,658
Cost before E&C (excluding design)	17,655,000	16,581,463
Engineering and Contingencies	1,524,600	----------.
Total cost (excluding design)	$19,179,600	$16,581,463
Total cost (excluding R/W & design)	$11,688,600	$9,296,541

*Actual cost from ADOT records, rounded to nearest dollar

Summary of Planned Costs Compared to Actual Costs

Table 29 summarizes the preliminary engineering cost estimates as compared to the engineer's estimate (where available), the bid price (where available) and the actual cost.

Table 29. Cost Comparisons for Specific Interchanges

INTERCHANGE	DCR/PE COST ESTIMATE*	ENGINEER'S ESTIMATE	ACTUAL COST
I-17 / Indian School (TUDI)	$11,688,000	N/A	$9,296,541
I-17 / Dunlap (SPUI/F)	7,920,122	$15,194,000*	$19,875,010*
I-17 / Northern (SPUI/F)	7,921,177	*	*

*Cost shown under I-17 / Dunlap includes both that interchange and I-17 / Northern

The Preliminary Engineering cost estimate for the I-17 / Indian School TUDI did not include an estimate for ADOT Engineering and Administration (15%) nor for Construction Surveys (1%), which were included in the Design Concept Report estimates for the I-17 / Dunlap and I-17 / Northern interchanges. In the interest of comparing like estimates the DCR / PE Cost Estimates shown in Table 29 do not include the 15% ADOT Engineering and Administration nor the 1% Construction Surveys in any of the three estimates. All three estimates include the 15% Contingency.

Care must be exercised in drawing definite conclusions from this data due to the small sample size, however the information is provided as the best available. It should be noted that the cost estimate for the TUDI at I-17 / Indian School was finalized in 1988 while those for the two SPUI/F were finalized in 1995. Because of the increase in costs during those years a comparison of TUDI cost to SPUI/F cost should be done with caution. It does appear that the comparison of estimated cost to actual cost is valid, because of a similar time period. Table 30 summarizes the actual cost as a percentage of the Design Concept Report or Preliminary Engineering cost estimate for the TUDI (one sample) and the SPUI/F (two samples).

Table 30. Cost Comparison for Interchange Type

INTERCHANGE TYPE	DCR/PE COST ESTIMATE	ACTUAL COST	ACTUAL COST AS PERCENTAGE OF ESTIMATED COST
TUDI (one sample)	$11,688,000	$9,296,541	80%
SPUI/F (two samples)	$15,841,299	$19,875,010	125%

Life Cycle Costs

A previous study (*31)* prepared a sample life cycle cost computation comparing a SPUI/n with a TUDI. The computation assumed a 7% amortization rate, and assigned a service life to each road element. It should be noted that this comparison was for a SPUI without frontage roads, however the evaluation method has relevance in this study.

Bridge	50 years
Roadway items	15 years
Earthwork	100 years
Retaining systems	50 years
Signals	10 years
Lighting	25 years
Signing	15 years
Engineering	25 years

The annualized differential in construction costs between the two types of interchanges was computed with the SPUI/n having an annual construction cost that was $233,824 more than the TUDI. The Utah Report was prepared in 1991, and the assumed costs are quite different than recent estimated costs for Arizona. As an example, the Utah study estimated the SPUI/n bridge costs to be $1,500,000 more than the TUDI The DCRs for both the I-17 / Dunlap and I-17 / Northern estimated the bridge cost to be $500,000 more for the SPUI/F than for the TUDI.

The Utah study then compared the differential in the annual cost of construction ($233,824) with the annual differential in road user costs ($400,000) and indicated the results favored the SPUI/n. (The SPUI/n had the higher construction costs and the lower road user costs.)

The Utah study did not include several costs which were indicated to be negligible or have an undetermined differential including mobilization, traffic control, drainage, right-of-way and utility relocations. (In Arizona, the deciding factor in selecting the SPUI/F at Northern and Dunlap Avenues was the savings in right-of-way costs.)

The final conclusions of the Utah study were:

"Confidence in the validity of the nearly 2:1 cost advantage of the SPUI[/n] is sufficiently low that other factors (i.e., geometry, maintenance of traffic during construction, drainage, utility relocations, access, right-of-way, available financing, traffic mix, peak flows, and related matters) may tilt the balance in favor of the CDI. [A CDI, compressed diamond interchange, is similar to a TUDI]. On the other hand, safety, reduced delays, and other features described in the preceding report can be used to favor the SPUI/n. What the preceding exercise does show is that user cost savings may justify a higher initial investment in an interchange reconstruction."

Road User Costs

Typically, road user costs are based on the 1977 *Manual on User Benefit* Analysis *(32)* published by AASHTO. However, this data has become outdated and for this project we looked to other sources for data pertaining to road user costs with respect to delay. References pertaining to the subject of comparing cost effectiveness of SPUI interchange versus an at-grade intersection include NCHRP 345 *(5)* and an article in the Journal of Transportation Engineering *(33)*. Both sources detailed the user cost aspect of interchange evaluation between a SPUI option and an at-grade intersection. However, more up-to-date information was provided in NCHRP 345 pertaining to cost of delay, operating costs at idle, operating costs at stops, operating costs at operating speed, and accident costs.

Cost of delay data was given in NCHRP 345 as cost per vehicle-hour of delay. The figures of $12.69 per vehicle-hour for passenger cars and $23.02 per vehicle-hour for trucks were presented as 1990 dollars in the article. These values were based on research presented in NCHRP Project 7-12, which was an update to the information initially provided in the AASHTO *Manual on User Benefit Analysis*. In order to apply these delay cost figures to the calculated delay results from the project, the figures were converted to 2001 dollars. Consumer price indices (CPI) for Transportation and All Items were used in order to perform this calculation. The proportion of the Transportation CPI from 1982-2001 to the All Items CPI from 1982-2001 was calculated from CPI data provided by the Bureau of Labor Statistics. This proportion was then applied to the ratio of the All Items CPI for 2001 to the All Items CPI for 1990. The result was a factor of 1.18, which was applied to the user delay costs of $12.69 and $23.02 to obtain $14.97 and $27.16 in terms of 2001 dollars. The new user delay cost figures were applied to the calculated delay values for each interchange type. An assumption of 10% trucks was used in the calculation and the user costs were weighted accordingly. The result was converted from user costs per hour to annual user costs of delay for each interchange type.

User cost of delay was selected as the point of comparison between SPUI/F and TUDI interchanges due to its substantial contribution to the overall road user cost. User costs of idling were assumed to be in proportion to the user costs associated with delay. However, this proportion was considered to be relatively insignificant with respect to the total user costs from delay. The weighted cost for idling used in NCHRP 345 was $0.943 per vehicle-hour. This figure, in 2001 dollars ($1.11), is only 6.8% of the weighted user cost of delay ($16.19 per vehicle-hour). Another user cost associated with interchange operations pertains to the number of vehicle stops incurred. Although this data was not available in the scope of this project, it is assumed that the number of stops incurred at SPUI/F and TUDI interchanges are equal. Operating speeds at SPUI/F and TUDI interchanges are assumed to be equal which negates the difference in user costs of operating speed. User costs associated with crashes were also regarded as equal based on the results presented in Chapter 3, which concluded that there is no significant difference in the crash rates associated with SPUI/Fs as compared to TUDIs.

Using the volumes predicted in the Design Concept Report or Preliminary Engineering Report, a life cycle cost evaluation of the three interchanges was made. Table 31 shows the annual user cost in 2001 dollars for the SPUI/F and four phasing variations of the TUDI based on an assumed 2-hour AM peak and 2-hour PM peak for a total of four peak hours. Each of these peak hours was assumed to be 8% of the total daily volume. Table 32 shows the same information based on the following highest 8 hours of the day. This is based on the median volume of this eight-hour period being the 8^{th} highest hour of the day, using the hourly adjustment factor contained in ADOT Traffic Engineering Policies, Guidelines, and Procedures Manual *(34)*. Table 33 shows the total user costs of delay based on the peak and non-peak conditions for the design year interchange ADT per the planning documents.

Table 31 Design Year Peak Hours Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$4,644,000	$7,968,000	$2,910,000	$3,330,000	$2,477,000
Northern	$2,129,000	$1,076,000	$1,102,000	$1,828,000	$1,214,000
Dunlap	$1,566,000	$867,000	$893,000	$1,277,000	$987,000

Table 32 Design Year Non-Peak Hours Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$3,378,000	$1,871,000	$1,949,000	$3,064,000	$2,151,000
Northern	$2,241,000	$1,299,000	$1,351,000	$1,831,000	$1,418,000
Dunlap	$1,815,000	$1,075,000	$1,112,000	$1,443,000	$1,237,000

Table 33 Design Year Total Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$8,022,000	$9,839,000	$4,859,000	$6,394,000	$4,628,000
Northern	$4,370,000	$2,375,000	$2,453,000	$3,659,000	$2,632,000
Dunlap	$3,381,000	$1,942,000	$2,005,000	$2,720,000	$2,224,000

Table 34 through 36 shows the same information as above except it is based on an opening year interchange ADT. The opening year interchange ADT was determined by assuming the design year ADT was a result of the opening year interchange ADT growing at an average annual rate of 2% for 20 years.

Table 34 Opening Year Peak Hours Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$1,503,000	$830,000	$885,000	$1,348,000	$959,000
Northern	$1,017,000	$595,000	$619,000	$827,000	$652,000
Dunlap	$832,000	$496,000	$507,000	$661,000	$571,000

Table 35 Opening Year Non-Peak Hours Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$1,720,000	$1,035,000	$1,106,000	$1,486,000	$1,158,000
Northern	$1,249,000	$760,000	$755,000	$997,000	$848,000
Dunlap	$1,072,000	$651,000	$638,000	$853,000	$765,000

Table 36 Opening Year Total Annual User Costs of Delay

Interchange	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Indian School	$3,223,000	$1,865,000	$1,991,000	$2,834,000	$2,117,000
Northern	$2,266,000	$1,355,000	$1,374,000	$1,824,000	$1,500,000
Dunlap	$1,904,000	$1,147,000	$1,145,000	$1,514,000	$1,336,000

The total annual user costs of delay for each interchange are incorporated into a present worth calculation. From the road user cost of delay standpoint, the TUDI provides the greater user cost benefit when comparing the SPUI/F and TUDI costs in Table 33 and Table 36. For the design year, the user cost benefits of a TUDI are shown below for the interchange locations examined:

Table 37 Design Year User Cost Benefits of a TUDI

Interchange	SPUI/F Total User Cost of Delay	Lowest TUDI Total User Cost of Delay	User Cost of Delay Benefit for TUDI
Indian School	$8,022,000	$4,628,000	$3,394,000
Northern	$4,370,000	$2,375,000	$1,995,000
Dunlap	$3,381,000	$1,942,000	$1,439,000

For the opening year, the user cost benefits of a TUDI are shown in the following table for the interchange locations examined:

Table 38 Opening Year User Cost Benefits of a TUDI

Interchange	SPUI/F Total User Cost of Delay	Lowest TUDI Total User Cost of Delay	User Cost of Delay Benefit for TUDI
Indian School	$3,223,000	$1,865,000	$1,358,000
Northern	$2,266,000	$1,355,000	$911,000
Dunlap	$1,904,000	$1,145,000	$759,000

In order to assess the present worth of these benefits, the following equation *(5)* is used:

$$f = (e^{(r-i)*n} - 1) / (r - i) \qquad \textbf{(1)}$$

where *f* represents the factor that adjusts the opening year's benefits to estimate benefits for the service life of the interchange; *n* equals service life (20 years); *r* = ln (*a*) / *n*, where *a* equals the ratio of the n-year's benefit to the opening year's benefit; and i is equal to the discount rate, which is assumed to be 0.04 (4% per year).

Applying Equation 1 to the user cost benefits of a TUDI gives a present worth (at the interchange opening year) of the benefits equal to:

$28,797,748 (f = 21.206) for the Indian School interchange
$18,073,329 (f = 19.839) for the Northern interchange
$14,025,561 (f = 18.479) for the Dunlap interchange

This present worth of benefits would be discounted by the capital required to obtain the additional right of way needed for a TUDI (if this amount exceeded the difference in construction costs which favor a TUDI construction). However, the additional expense to obtain the necessary right of way would be insignificant when compared to the present worth of the benefits.

CONCLUSIONS

A cost evaluation of interchanges can be made in various manners. If one evaluates only right-of-way and construction costs as was done in the Design Concept Reports an alternative may be selected which will provide the least initial cost, however which may result in a higher life cycle cost. This is especially true if one considers the cost to the motoring public. This evaluation has compared the planning level cost estimates of two interchange types with the actual cost when they were finally built. Although the sample sizes do not permit definitive conclusions, the cost estimates for the SPUI/F appear to have been underestimated.

When one considers the road user cost, the life cycle cost of the TUDI for all three interchanges is considerably less than that of the SPUI/F. The primary reason for this is the additional delay at the SPUI/F for interchanges with these ramp separation distances as discussed in Chapter 4.

CHAPTER 6
RECOMMENDED DESIGN GUIDELINES FOR THE SINGLE POINT URBAN INTERCHANGE WITH FRONTAGE ROADS

INTRODUCTION

This chapter describes geometric and traffic control device guidelines for the single point urban interchange with frontage roads (SPUI/F). These guidelines are not comprehensive in their treatment of geometric or control device design for the interchange area. As such, they are intended to be used in conjunction with existing guideline documents, including *A Policy on Geometric Design of Highways and Streets* (*35*) (i.e., the *Green Book*), the Arizona Department of Transportation's *Roadway Design Guidelines* (*36*), and the *Manual on Uniform Traffic Control Devices* (*37*). Additional guidance for SPUI design can also be found in *NCHRP Report 345 - Single Point Urban Interchange Design and Operations Analysis* (*5*).

The geometric guidelines in this document address design element controls and criteria for which guidelines are either not specifically provided in the *Green Book* or are provided in the *Green Book* but are not updated to reflect the findings of current research. Topics addressed include: sight distance, left-turn path design, ramp-interchange spacing, cross street crest curvature, and cross section components. The traffic control device guidelines address signing, pavement markings and signalization elements not contained in the *Manual on Uniform Traffic Control Devices (37).*

GEOMETRIC DESIGN GUIDELINES

Design Objectives

The SPUI/F requires a systems-based design process to insure that all functional elements are coordinated and reflect a thoughtful balance between driver needs and interchange cost. This requirement reflects a complex interaction between the size of the SPUI/F conflict area and the orientation of the intersecting travel paths. No one design component can be designed in isolation of the others; to do so may result in unsafe or inefficient operation. These complications stem from the SPUI/F multiple and lengthy travel paths through the conflict area, the limited number of locations where island channelization can be used in the conflict area, and the potential for sight line obstructions.

A systems-based approach to SPUI/F design can be facilitated by adherence to the following design objectives (these objectives were previously offered by Messer *et al.* (*5*)):

- Design the left-turn paths to support a design speed that is consistent with that of the interchange approaches and consistent with driver expectancy.

- Locate bridge abutments, retaining walls, roadside barriers and other potential sight obstructions so that they do not compromise the left-turn driver's stopping sight distance when traveling along the left-turn path.

- Provide sufficient separation distance between the interchange and adjacent cross street intersections to allow turn-related weaving maneuvers to occur freely, even when a downstream queue is present.

- When crest curvature is present on the cross street, insure that it is sufficiently flat within the conflict area as to provide drivers a clear view of all applicable pavement markings and conflicting vehicles.

- Size the interchange to provide adequate capacity to satisfy vehicular demand throughout a 20-year design life.

- Whenever possible, chose a cross section that allows a second left-turn lane to be added at a later time (if not provided in the initial design) without having to modify to the bridge structure.

- Provide design element sizes that exceed the minimum control values whenever possible.

The remaining sections describe guidelines that can be used to achieve these objectives.

Design Controls

This section describes two fundamental controls for the geometric design of a SPUI/F. The control described first is Sight Distance; the second is Design Speed. The discussion of sight distance is a summary of guidance provided in the *Green Book* (*35*), as it relates to all interchange traffic movements. The discussion of design speed focuses on the design speed of the left-turn paths. The content of this discussion is based on a synthesis of information provided in the *Green Book* and in *NCHRP Report 439 - Superelevation Distribution Methods and Transition Design* (*38*).

Sight Distance

Several types of sight distance are needed by drivers when approaching and traveling through a SPUI/F. Messer *et al.* (*5*) discuss the need for stopping sight distance, decision sight distance, and intersection sight distance in a SPUI/F design. They recommended that: (1) stopping sight distance should be provided everywhere along the left-turn paths; (2) decision sight distance should be used as a control for the horizontal and vertical alignments of the exit-ramp approaches to the SPUI/F; and (3) that intersection sight distance should be provided for the left and right-turning drivers at the interchange. They also recommend that the SPUI/F design elements be sized such that the available sight distance exceeds the "desirable minimum" distance. This additional distance is justified because of the likelihood that many drivers are unfamiliar with the SPUI/F configuration.

Sight distance control values cited in the *Green Book* are listed in Table 39. Values of decision sight distance are defined according to area type (i.e., suburban or urban) and apply to maneuvers that result in a speed, path, or direction change. The "suburban" values correspond to about 12 s travel time; those for "urban" conditions correspond to 14 s travel time.

The intersection sight distances shown in Table 39 were derived from two sources. The "absolute minimum" values are based on 7.5-s travel time, as recommended by Harwood *et al.* (*39*) following a comprehensive investigation of intersection sight distances. The "desirable minimum" values are based on the procedures described in Chapter IX of the *Green Book* (*35*) (for Case IIIC - Turning Right into a Major Road).

The proposed guidelines and controls described in subsequent sections of this document are based on the sight distances listed in Table 39.

Design Speed

This section describes a procedure for defining the design speed of the SPUI/F left-turn paths. The procedure is based on guidance provided in Chapters III and X of the *Green Book*; however, additional detail has been added to provide greater clarity and consistency in the left-turn path design process.

As a desirable goal, the design speed of the subject left-turn path should equal that of the roadway exited *and* that of the roadway entered. However, in some instances, the design speeds of the exited and entered roadways are different. Also, it is sometimes found that bridge length (and cost) can be significantly reduced if a sharper curve (with lower design speed) is used. If either of these situations is present, it is acceptable to use a design speed that is 10 to 20 km/h (5 to 15 mph) below that of the roadway exited or entered. Guidance to assist with the selection of curve design speed is summarized in Table 40.

The SPUI/F left-turn path can consist of either a simple radius or a compound curve design. The compound curve would consist of two or three simple curves. If it is determined that the left-turn path design speed should be equal to that of the exited and entered roadways, the left-turn path can consist of a simple (or single) radius; otherwise, a compound design is required. As noted by Messer *et al.* (*5*), the simple curve is preferable to a compound curve design because the simple curve is easier to design and is probably more consistent with driver steering behavior.

The following approach can be used to determine the appropriate design speed combination and curve design type for a SPUI/F. First, a 3-centered, compound curve should be posed as the initial curve design type with each component curve (i.e., entry curve, central curve, exit curve) potentially having a different design speed and radius. The Entry curve's design speed should match that of the roadway segment just in advance of the curve. The Exit curve's design speed should match that of the roadway segment just following the curve. For example, in the design of the cross-street left-turn path, the Entry curve would have the same design speed as the cross street and the Exit curve would have the same design speed as the frontage road.

Second, the Central curve's design speed is selected using Table 40. The speed selected should satisfy either the "desirable" or "acceptable" speeds associated with both the Entry curve design speed and the Exit curve design speed. The full range of design speed combinations that satisfy these conditions are listed in Table 41.

Table 39. Minimum sight distance requirements.

Metric						
Design Speed, km/h	Minimum Sight Distance, m					
	Stopping Sight Distance [1]		Decision Sight Distance [2]		Intersection Sight Distance [3]	
	Absolute Minimum	Desirable Minimum	Suburban Minimum	Urban Minimum	Absolute Minimum	Desirable Minimum
30	29.6	29.6	100	130	65	65
40	44.4	44.4	135	165	85	85
50	57.4	62.8	160	200	105	110
60	74.3	84.6	205	235	125	145
70	94.1	110.8	240	275	145	185
80	112.8	139.4	275	315	165	235
90	131.2	168.7	320	360	190	290
100	157.0	205.0	365	405	210	355
110	179.5	246.4	390	435	230	430

U.S. Customary						
Design Speed, mph	Minimum Sight Distance, ft					
	Stopping Sight Distance		Decision Sight Distance		Intersection Sight Distance	
	Absolute Minimum	Desirable Minimum	Suburban Minimum	Urban Minimum	Absolute Minimum	Desirable Minimum
20	97	97	330	430	210	210
25	146	146	440	540	280	280
30	188	206	520	660	340	360
35	244	278	670	770	410	480
45	309	364	790	900	480	610
50	370	457	900	1030	540	770
55	430	553	1050	1180	620	950
60	515	673	1200	1330	690	1160
70	589	808	1280	1430	750	1410

Notes:

1 - *Green Book* (*35*), Table III-1.

2 - *Green Book* (*35*), Table III-3. Values apply to speed, path, or direction changes. Values for 30 and 40 km/h (20 and 25 mph) are estimated by extrapolation.

3 - Sight distance to the left, as needed by a frontage-road right-turn movement. Absolute Minimum: Harwood *et. al.* (*39*), Table 28 (based on 7.5-s travel time); Desirable Minimum: *Green Book* (*35*), Figure IX-41.

Table 40. Left-turn alignment design speed as related to through roadway design speed.

Metric							
Facility		**Design Speed, km/h**					
Frontage road or cross street		40	50	60	70	80	90
SPUI/F Left-turn path	Desirable:	40	50	60	70	80	90
	Acceptable:	30	40	50	60	60	70

U.S. Customary							
Facility		**Design Speed, mph**					
Frontage road or cross street		25	30	35	45	50	55
SPUI/F Left-turn path	Desirable:	25	30	35	45	50	55
	Acceptable:	20	25	30	35	35	45

Finally, the curve design type is selected using Table 41 based on the selected design speeds of the three curves. If the design speed of the Central curve is different from that of the Entry and Exit curves, then a 3-centered curve is appropriate. If two consecutive curves have the same speed, then they can be considered to represent one curve segment of common radius (i.e., a 2-centered curve). If all three curves have the same speed, then they can be considered to represent one curve having a simple radius.

For example, consider a SPUI/F left-turn path that departs from a frontage road that has a 70 km/h (45 mph) design speed and that intersects with a cross street that also has a 70-km/h design speed. Table 41 indicates that a simple curve design and a 3-centered compound curve design are available (see the underlined values). If design simplicity is a priority, the simple-radius curve design with a design speed of 70 km/h (45 mph) can be selected. If there are indications that bridge costs will be significantly reduced by using compound curvature, then a 3-centered curve design is available. For the 3-centered design type, the Entry, Central, and Exit curves would have design speeds of 70, 60, and 70 km/h (45, 35, and 45 mph), respectively.

Horizontal Alignment

General

The horizontal alignments of two intersecting roadways typically do not intersect at a 90-degree angle. This deviation from a 90-degree intersection is referred to as "skew." A clockwise rotation of the cross street (relative to the major road) is defined herein as a positive skew angle. Skew has been found to have a negative impact on the SPUI's construction cost and traffic operation (*5, 40*).

Messer *et al.* (*5*) noted that skew (positive or negative) has a significant affect on the SPUI bridge design. They found that skew tends to increase the size of the SPUI bridge deck, especially for the overpass SPUI design. This increase indirectly stems from the larger radii needed for two of the left-turn paths. The effect of skew on overpass SPUI/F bridge length and left-turn radius is illustrated in Figure 37. The radii reported in this figure reflect a simple radius design (or "equivalent" simple radius for a compound curve design).

Table 41. Design speed combinations and corresponding curve design type.

Metric [1, 2, 3]							
Design Speed for Curve Segments, km/h				**Design Speed for Curve Segments, km/h**			
Entry	**Central**	**Exit**	**Curve Type**	**Entry**	**Central**	**Exit**	**Curve Type**
40	40	40	Simple	70	70	70	Simple
		50	2-centered			90	2-centered
	30	40	3-centered		60	60	2-centered
50	50	50	Simple			70	3-centered
		60	2-centered			80	3-centered
	40	40	2-centered	80	80	80	Simple
		50	3-centered		60	60	2-centered
60	60	60	Simple			70	3-centered
		70	2-centered			80	3-centered
		80	2-centered	90	90	90	Simple
	50	50	2-centered		70	70	2-centered
		60	3-centered			90	3-centered

U.S. Customary [1, 2, 3]							
Design Speed for Curve Segments, mph				**Design Speed for Curve Segments, mph**			
Entry	**Central**	**Exit**	**Curve Type**	**Entry**	**Central**	**Exit**	**Curve Type**
25	25	25	Simple	45	45	45	Simple
		30	2-centered			55	2-centered
	20	25	3-centered		35	35	2-centered
30	30	30	Simple			45	3-centered
		35	2-centered			50	3-centered
	25	25	2-centered	50	50	50	Simple
		30	3-centered		35	35	2-centered
35	35	35	Simple			45	3-centered
		45	2-centered			50	3-centered
		50	2-centered	55	55	55	Simple
	30	30	2-centered		45	45	2-centered
		35	3-centered			55	3-centered

Note:

1 - "Simple" denotes a constant radius throughout the left-turn path; it represents a "desirable" design.

2 - "2-centered" or "3-centered" denote compound curves with two or three radii, respectively; they represent "acceptable" designs.

3 - Values used in the example application described in the text are underlined.

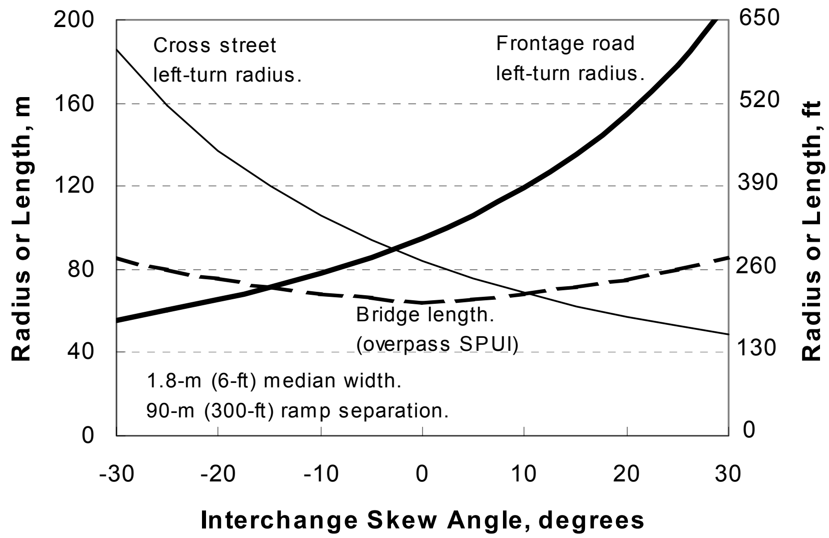


Figure 37. Effect of skew angle on left-turn radius and bridge length.

The trends in Figure 37 indicate that the effect of skew is more pronounced on the left-turn radius values than on bridge length. Radii tend to range from 50 to 200 m (160 to 650 ft) over the range of skew angles shown and change in length by about 7 percent for every degree of skew. In contrast, bridge length ranges from 65 to 85 m (210 to 280 ft) and increase about 1.0 percent for each degree of skew. It should be noted that bridge length and radius are sensitive to the distance between the ramps (or frontage roads, if they exist). For example, a 60 m (200 ft) ramp separation distance will increase bridge length by about 40 percent relative to that shown in Figure 37. The same 60 m ramp separation will reduce the left-turn radii by about 23 percent.

Bonneson (*40*) has reported that skew can also have an adverse effect on SPUI traffic operation. Specifically, skew tends to increase the all-red clearance interval of most SPUI signal phases. This increase stems from increased travel distances through the conflict area and, for two left-turn movements, decreased clearance speeds (via smaller radii). The effect of skew on the "total" clearance time for a SPUI/F (i.e., the sum of the all-red clearance interval for each of the signal phases) is shown in Figure 38.

The trends shown in Figure 38 indicate that total clearance time increases with skew and ramp separation distance. The clearance times shown reflect a cross-street speed limit of 64 km/h (40 mph). Additional investigation indicates that each 8-km/h (5-mph) increase in cross street speed limit reduces the total times shown in Figure 38 by about 0.7 s.

Left-Turn Geometry

The benefits of the SPUI/F design are largely a result of its large, sweeping left-turn paths. However, realization of any operational or safety benefits requires careful attention to the design of these paths. Design controls that need to be considered include: design speed, minimum curve radius, minimum stopping sight distance, and minimum lateral clearance. Guidelines for identifying values for these controls and selecting appropriate design element dimensions are provided in this section.

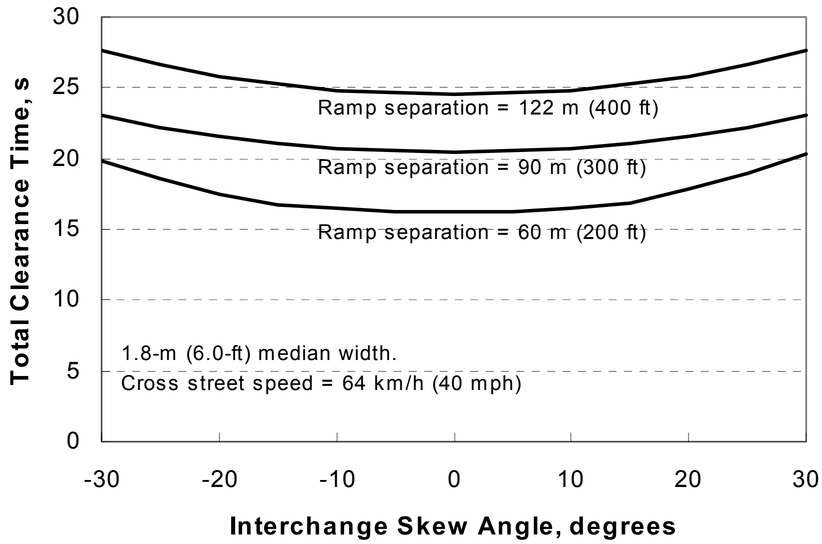


Figure 38. *Effect of skew angle on total clearance time.*

Radius. As noted in the section titled Design Speed, simple or compound curvature can be used for the SPUI/F left-turn paths. The use of a simple curve design type is preferable to compound curvature because it is easier for the turning driver to navigate. However, considerations of bridge cost or conflict area size may sometimes justify the use of compound curvature. Both design types are described in this document.

Table 42 defines the minimum left-turn radius for a given design speed. Larger radii are encouraged when conditions allow. For compound curve design types, each component curve will have a separate design speed and a separate minimum radius. The lateral clearance information listed in Table 42 is discussed in a subsequent section.

The minimum radii listed in Table 42 are associated with a mid-curve speed that is 10 km/h lower than the design speed for design speeds of 30 through 70 km/h. The mid-curve speed is 20-km/h lower for design speeds of 80 and 90 km/h. Mid-curve speeds in U.S. Customary units will be from 5 to 15 mph lower, depending on the design speed.

Messer *et al.* (*5*) evaluated the need for superelevation on the SPUI left-turn paths. They noted that the use of superelevation is complicated by the fact that a significant portion of the path is located within the interchange conflict area. Because it is impractical to provide superelevation within this area and because of frequent stopping on the ramp-portion of the left-turn path, they recommended that superelevation not be provided on the left-turn path (beyond the nominal amount that may be needed for drainage). Accordingly, the radii listed in Table 42 do not reflect the use of superelevation.

The minimum radii listed in Table 42 are based on the following relationship:

$$R = \frac{V_c^2}{127\ f} \tag{11}$$

with,

$$f = 0.243 - 0.00187\ V + 0.0068\ (V\ -\ V_c) \qquad \textbf{(12)}$$

where:

V = design speed of the subject curve, km/h;
V_c = speed reached along the subject curve, km/h;
f = side friction demand factor, and
R = radius of the inside edge of the inside lane of the subject curve, meters.

Equation 2 was developed by Bonneson (*38*) and is based on measurements of speed on turning roadways at several intersections and interchanges throughout the United States, including Arizona.

The mid-curve speed V_c can be computed through the use of Equations 1 and 2 or determined graphically from Figure 39. In this figure, one trend line is provided for each design speed in the range of 30 to 90 km/h (20 to 55 mph). The left-most endpoint of each trend line coincides with the minimum radius for the corresponding design speed. The curve speed associated with the minimum radius represents the minimum curve speed. Radii larger than the minimum radius will yield larger speeds along the curve, up to the point at which the curve speed equals the design speed and the trend line becomes horizontal.

To illustrate the use of Figure 39, consider the situation where an Entry curve is identified as having a design speed of 70 km/h (45 mph). Table 42 indicates that the minimum radius associated with this design speed is 155 m (510 ft). Figure 39 indicates that, if a 155 m radius is used for a 70 km/h design speed, the curve speed will be 60 km/h (i.e., a 10-km/h speed reduction, as noted in the discussion associated with Table 42). However, if the designer decides to use a 200 m (650 ft) radius, Figure 39 indicates that the curve speed will be about 63 km/h (i.e,. a 7-km/h speed reduction).

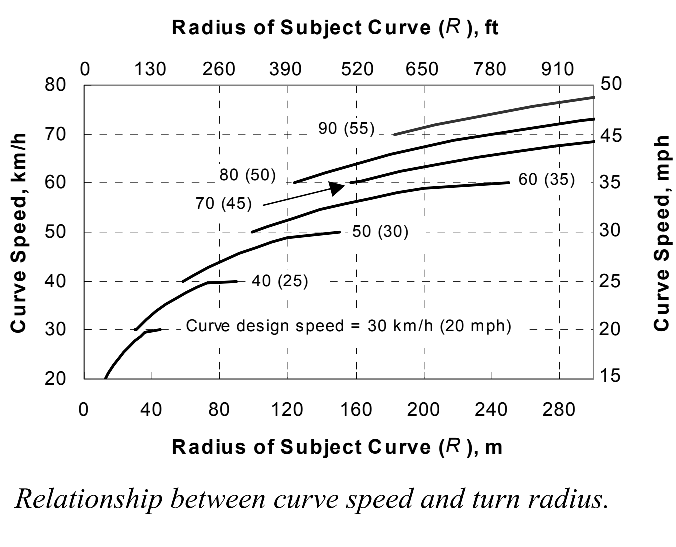


Figure 39. Relationship between curve speed and turn radius.

Table 42. Minimum left-turn radius and lateral clearance.

Metric					
Design Speed,[1] km/h	Minimum Left-Turn[2] Radius, m	Stopping Sight Distance,[3] m		Lateral Clearance,[4] m	
		Absolute Minimum	Desirable Minimum	Absolute Minimum	Desirable Minimum
30	15	29.6	29.6	6.1	6.1
40	30	44.4	44.4	7.4	7.4
50	60	57.4	62.8	6.6	7.8
60	100	74.3	84.6	6.7	8.7
70	155	94.1	110.8	7.0	9.7
80	125	112.8	139.4	12.3	18.7
90	185	131.2	168.7	11.4	18.7

U.S. Customary					
Design Speed,[1] mph	Minimum Left-Turn[2] Radius, ft	Stopping Sight Distance, ft		Lateral Clearance, ft	
		Absolute Minimum	Desirable Minimum	Absolute Minimum	Desirable Minimum
20	50	97	97	20	20
25	100	146	146	24	24
30	200	188	206	22	26
35	330	244	278	22	29
45	510	309	364	23	32
50	410	370	457	40	61
55	610	430	553	37	61

Notes:

1 - Design speed is that of the subject curve. For design speeds of 30 through 70 km/h, the subject curve will have, at its midpoint, a 95th percentile speed that is 10 km/h below the design speed. For design speeds of 80 and 90 km/h, the midpoint 95th percentile speed is 20 km/h below the design speed. Mid-point speeds in U.S. Customary units will be from 5 to 15 mph lower, depending on design speed.

2 - Radius is measured to the inside edge of the left-turn lane (inside lane of a dual-lane curve).

3 - Absolute minimum and desirable minimum stopping sight distance values are obtained from the *Green Book* (*35*), Table III-1.

4 - Lateral clearance values are obtained from the *Green Book* (*35*), Figures III-24(A) and III-24(B) and correspond to the minimum radii listed (smaller clearances are needed for larger radii). Lateral clearance is measured from the centerline of the left-most left-turn path to the nearest vertical sight obstruction on the inside of the curve.

Simple Curve Design. This section describes a procedure for selecting the radius for a simple curve design type (compound curve radius selection is described in the next section). As noted in a previous section, the simple left-turn radius that "fits" the SPUI/F can be computed as a function of skew angle and ramp separation distance. In general, radii between 75 and 105 m (250 and 345 ft) should be used with skew angles less than 5 degrees and ramp separations of 90 m (300 ft). Figure 37 can be used to identify appropriate radii for other skew angles. For SPUI/F with a ramp separation other than 90 m (300 ft), the following equation can be used to estimate the best-fit radius:

$$R = R_2\,[1 + 0.0078\ (X_r - 90)] \tag{13}$$

where:

R = radius of the inside edge of the inside lane of the subject curve, m;
R_2 = radius obtained from Figure 37, m; and
X_r = distance between ramp (or frontage road) centerlines, as measured along the cross street (i.e., ramp separation distance), m.

Regardless of the radius value obtained from Figure 37 or Equation 3, the radius used in design should equal or exceed the minimum listed in Table 42 for the selected design speed.

Compound Curve Design. The use of compound curvature in the SPUI/F left-turn path offers more flexibility in the design; however, the added flexibility comes at the expense of additional design and construction effort. This section describes a general procedure for selecting the radii used in each of the component curves of a 2 or 3-centered design type (i.e., Entry curve, Central curve, and Exit curve). The procedure consists of following a set of radius selection "rules" and then defining the minimum length of each curve. The procedure is consistent with the guidance provided in the *Green Book* (*35*).

There are three rules for radius selection for compound curve design, they are:

1. The radius of each curve should equal or exceed the minimum values listed in Table 42.

2. The ratio of each adjacent curve radius pair should follow same trend as the ratio of their corresponding design speeds. For example, if the design speed of the first curve is larger than that of the second curve, then so should the radius of the first curve exceed that of the second curve.

3. The ratio of the larger-to-smaller radius should not exceed 2:1.

Adherence to these three rules will insure a balanced curve design that provides: (1) an acceptable level of speed reduction into the curve and acceleration (if needed) when exiting the curve and (2) a radius change sequence that is consistent with the intended speed change.

The *Green Book* (*35*) advises that the length of each curve component should be sufficient to accommodate the intended speed change with a comfortable deceleration or acceleration rate. It offers that an acceptable maximum deceleration rate is 5 km/h/s (3 mph/s). This guidance translates into the lengths shown in Table 43 when the minimum radii and design speed combinations listed in Table 42 are used. In application, the length of each of the component curves is selected from Table 43, *if* its radius is equal to the minimum value listed in Table 42.

If the radius used for a particular curve exceeds the minimum value, then lengths shorter than those listed in Table 43 would apply. This deviation is a result of the lower speed reduction incurred on curves whose radii exceed the minimum values. For this situation, Figure 39 should be used to determine the corresponding curve speed V_c for the subject curve. Next, the speed change is computed as the difference between the design speed V of the subject curve and the curve speed. Finally, Figure 40 should be used to determine the minimum length of the subject curve. The trend lines shown are based on a 5 km/h/s (3 mph/s) deceleration (and acceleration) rate. This process is repeated for each of the curves that comprise the compound curve (i.e., Entry, Central, and Exit) to determine their respective minimum lengths.

Inside Lateral Clearance. The *Green Book* (*35, p. 118*) states that "...sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop in this distance." This guidance is equally applicable to the drivers negotiating the left-turn paths at SPUI/F. A satisfactory left-turn path design requires that bridge abutments, barrier walls, and pedestrian fences on the inside of the left-turn path do not compromise the driver's stopping sight distance. This distance was previously identified in Table 42.

Table 43. Minimum curve length for a curve of minimum radius.

Metric [1, 2]						
Minimum Curve Length	**Design Speed of Subject Curve, km/h**					
	40	**50**	**60**	**70**	**80**	**90**
Absolute Minimum, m:	19	25	31	36	78	89

U.S. Customary [1, 2]						
Minimum Curve Length	**Design Speed of Subject Curve, mph**					
	25	**30**	**35**	**45**	**50**	**55**
Absolute Minimum, ft:	60	80	100	120	255	290

Notes:
1 - Lengths based on a 10-km/h speed reduction along the length of the subject curve for design speeds of 40 to 70 km/h and a 20-km/h speed reduction for speeds of 80 and 90 km/h. The speed reduction ranges from 5 to 15 mph for U.S. Customary units, depending on design speed.
2 - Minimum length based on 5 km/h/s (3 mph/s) deceleration.

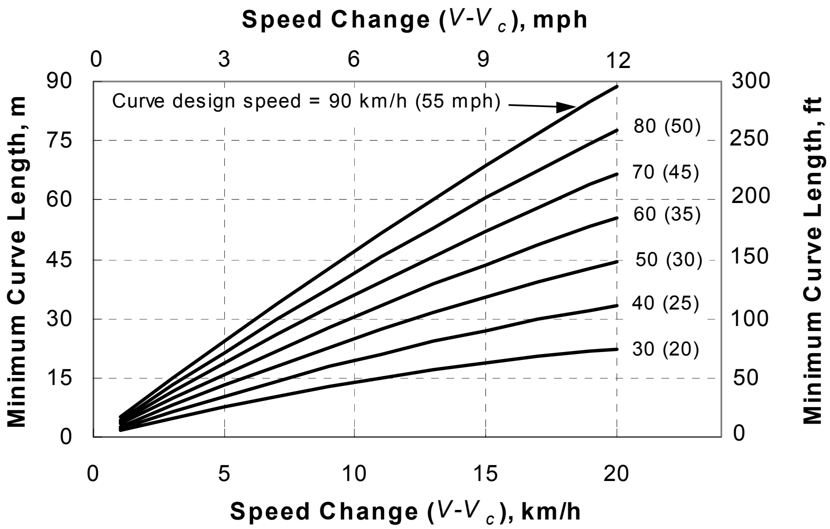


Figure 40. Minimum curve length for compound curve design.

Figure 41 illustrates the sight lines needed by the driver along the inside of the left-turn path at various points along the turn path. The length of each sight line is equal to the stopping sight distance, as measured along the path centerline. A "sight line boundary" is formed by the collection of sight lines along a given turn path. It represents the outer edge of an area that must be kept free of sight obstructions (e.g., columns, walls, fences, etc.). In design applications, this boundary can be located by determining the minimum lateral clearance *LC* for each left-turn path. Lateral clearance values for curves with radii at the minimum value are listed in Table 42.

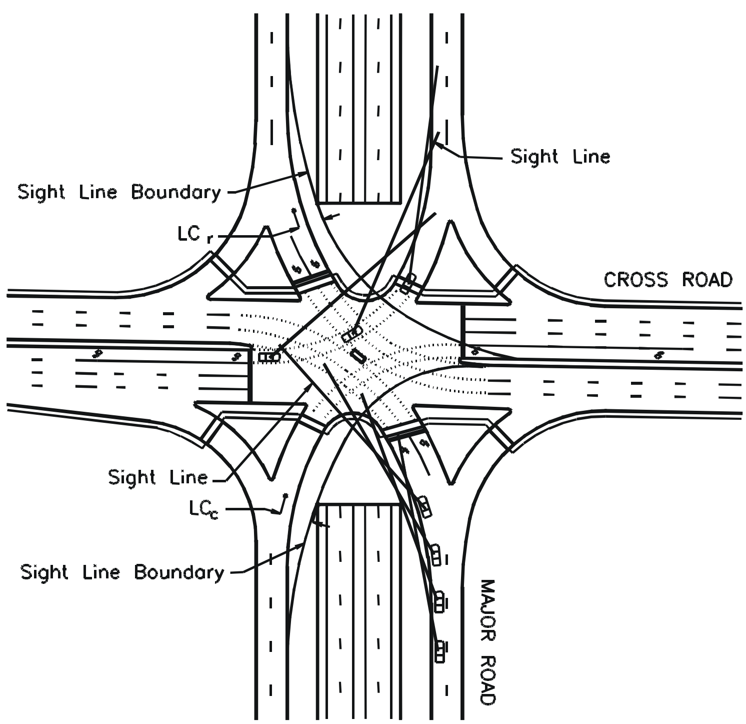


Figure 41. Sight line boundaries and associated lateral clearances on left-turning roadways.

The lateral clearance values listed in Table 42 tend to range from 6.1 to 9.7 m (20 to 32 ft) for design speeds of 30 to 70 km/h (20 to 45 mph) and from 11 to 19 m (37 to 61 ft) for designs speeds of 80 or 90 km/h (50 or 55 mph). Values in excess of 9.7 m (32 ft) are difficult to obtain with some SPUI/F designs. In these situations, a 3-centered curve design type with a Central curve design speed of 70 km/h (45 mph) or less may be appropriate.

If above-minimum-radii are used for a curve, then the required lateral clearance should be determined using Figures III-24(A) or III-24(B) in the *Green Book* (*35, p. 220*). Alternatively,

(14)

$$LC = R - R\,x\,\mathrm{Cos}\left(\frac{28.65\ S}{R}\right)$$

the following equation can be used to compute the necessary lateral clearance:

where:

- LC = lateral clearance (measured from the centerline of the left-most left-turn path to the nearest vertical sight obstruction on the inside of the curve), m;
- S = stopping sight distance (see Table 42), m; and
- R = radius of the inside edge of the inside lane of the subject curve, m.

Outside Lateral Clearance. The safe and efficient operation of the left-turn path is also dependent on the lateral clearance provided between two opposing left-turn movements that are served simultaneously during a signal phase. Following a review of the literature and current SPUI design practice, Messer *et al.* (*5*) recommended the provision of 1.8 m (6 ft) between the near side edge lines of the opposing left-turn paths. This recommendation is extended herein to SPUI/F design.

Cross Street Geometry

A key element of the cross street geometry is the distance between the center lines of the two ramps (or frontage roads, if present). This distance has been found to affect both the safety and operational efficiency of the SPUI. With regard to safety, Dorothy *et al.* (*4*), in an extensive review of SPUI design practice, noted that there was "significant driver confusion" when the distance between ramps is "very large." Messer *et al.* (*5*) also noted the increased potential for driver confusion with increasing distance between ramps.

The effect of SPUI/F size on driver confusion is likely due to the increasing similarity between the SPUI and the TUDI as ramp separation distance increases. While small SPUI/F tend to look and operate much like a large at-grade intersection, large SPUI/F look more like a TUDI but still operate like an at-grade intersection. Specific points of confusion at larger SPUI/F may pertain to driver selection of: (1) the correct lane from which to complete a left-turn, (2) the location of the stop line (both near-side and far-side), and (3) the location of the controlling signal head. This confusion may be alleviated to some degree as more drivers become familiar with the SPUI/F design; however, for now, SPUI/F should be designed for the unfamiliar driver with larger SPUI/F given extra attention in terms of their signing and marking plan.

The efficiency of SPUI/F operation is also affected by ramp separation distance. This effect is illustrated in Figure 42. The trends in this figure apply to a SPUI/F with no appreciable skew. They indicate that total clearance time increases with increasing ramp separation distance and with lower cross-street speeds. Clearance time is shown to increase from 15 s at 50-m separation to 25 s at 120-m separation, an increase of about 10 s. To add perspective on the implications of clearance time on SPUI/F performance, it is reasonable to assume that 1.0 s of clearance time translates into an equal amount of additional delay to each left-turn and through driver at the SPUI/F.

In summary, there is evidence that SPUI/F safety and operational efficiency degrade with increasing ramp separation distance. Therefore, ramp separation distance should be fully evaluated during design with the goal being to keep it as short as is reasonably possible.

Frontage Road Geometry

The guidelines in this section have been developed for the SPUI/F. However, some of these guidelines can also be used for SPUIs that do not have frontage roads; such extensions will be noted where applicable. The guidelines in this section address the following design controls:

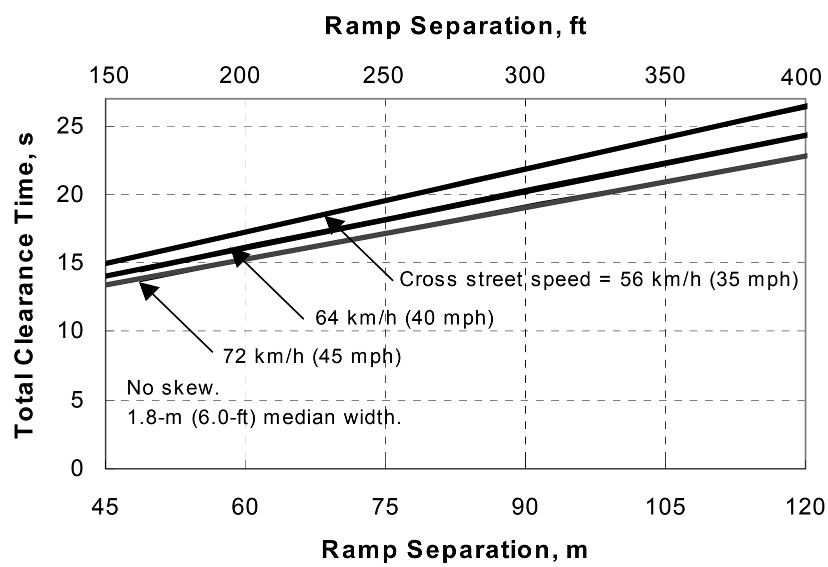


Figure 42. Effect of ramp separation distance on total clearance time.

- Left-turn Storage Length
- Ramp Meter Storage Length
- Weave Zone Length
- Exit-Ramp-to-Interchange Spacing
- Auxiliary Lane Length
- Interchange-to-Entrance-Ramp Spacing

The design elements that correspond to these six controls are shown schematically in Figure 43. Of the controls listed above, the first two (i.e., left-turn storage length and ramp meter storage length) are not unique to the SPUI/F. Existing ADOT guidelines will apply for these two controls. The last four controls are described in the next several sections.

Ramp Location. In general, the location of the exit and entrance ramps, relative to the cross street, is controlled by consideration of both queue storage and weaving. As shown in Figure 43, exit ramp location is dictated by the exit-ramp-to-interchange spacing and the combined weave zone length plus left-turn storage length. The larger of these two distances will control the location of the exit ramp. The weave zone length and exit-ramp-to-interchange spacing are measured from the exit ramp nose. The storage length and exit-ramp-to-interchange spacing are measured to the near edge of the cross street.

The entrance ramp location is dictated by the interchange-to-entrance-ramp spacing and the combined auxiliary lane length plus ramp meter storage length. The larger of these two distances will control the location of the entrance ramp relative to the cross street. The auxiliary lane length and interchange-to-entrance-ramp spacing are measured from the channelizing island nose. The storage length is measured to the ramp meter stop line and the interchange-to-entrance-ramp spacing is measured to the ramp nose. It should be noted that the ramp nose and island nose referred to herein is identified as the point where the two pavement edge lines intersect.

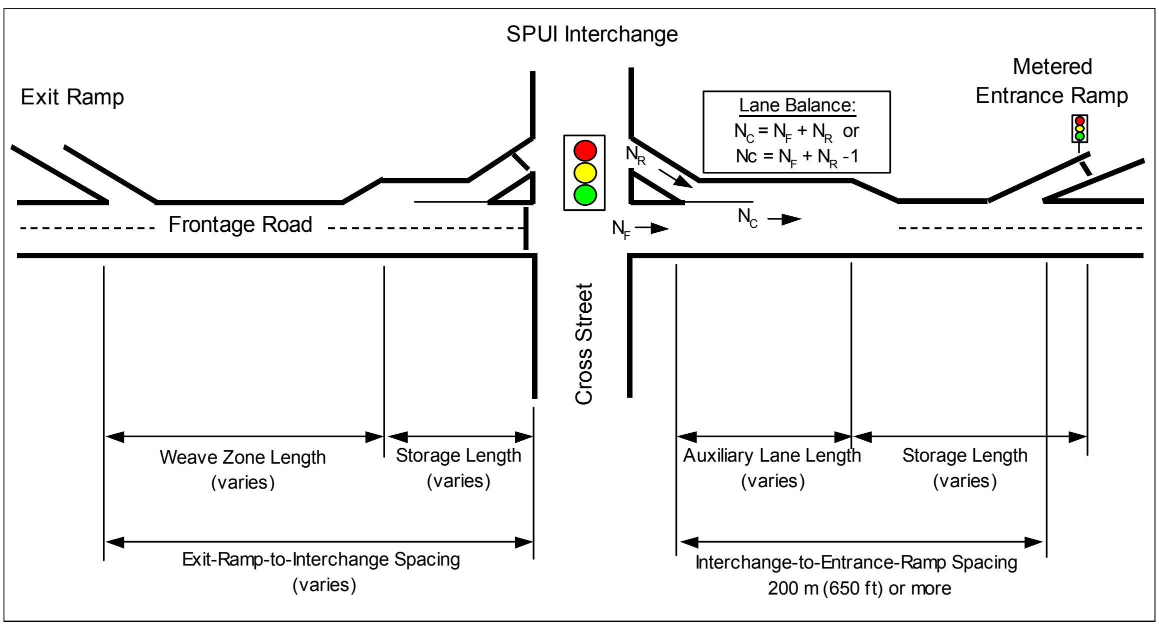


Figure 43. Design elements associated with frontage road geometry at SPUI/F.

Lane Balance. The need for an auxiliary lane on the frontage road, downstream of the cross street, is based on consideration of the principle of lane balance, as described in the *Green Book* (*35 p. 902*). Lane balance was initially developed for freeway merge-area applications; however, the concept is also applicable to the SPUI/F because its geometry and signalization require the cross-street left and right-turn movements to merge on the frontage road. The problem is heightened at the SPUI/F because the left-turn drivers merge to the *right* which is a relatively uncommon and difficult maneuver. Experience indicates that SPUI/F designs that do not provide lane balance tend to experience frequent conflicts at the merge point.

The applicable lane balance rule is summarized in Figure 43. In words, it states that "...the number of lanes beyond the merging of two traffic streams should not be less than the sum of all traffic lanes on the merging roadways minus one." To comply with this rule, a dual-lane left-turning roadway will typically require an auxiliary lane along the frontage road, as shown in Figure 43. In contrast, an auxiliary lane is optional for a single-lane left-turning roadway. If it is determined that an auxiliary lane is not needed, the entrance ramp location should be based on the longer of the ramp meter storage length and the interchange-to-entrance-ramp spacing.

Weave Zone Length. The length of this zone is dictated by the distance an exit-ramp driver needs to weave across the frontage road lanes, reach the right-hand (or outside) lane and decelerate to a stop (or near-stop) condition. Observational studies of this maneuver by Fitzpatrick *et al.*(*41*) reveal that a majority of drivers use between 60 and 120 m (200 and 390 ft) for this maneuver. A more recent study of exit-ramp weaving by Jacobson *et al.* (*42*) examined the effect of frontage road cross section width on the weave zone length. They found that the average weave maneuver required 80 m (260 ft) for a two-lane frontage road and that this distance increased 30 m (100 ft) for each additional lane. The findings by Jacobson *et al.* are offered in Table 44 as the recommended minimum Weave Zone Length for SPUI/F design.

Table 44. Minimum weave zone length.

Metric			
Weaving Distance [2, 3]	**Number of Lanes in Weaving Section [1]**		
	2	**3**	**4**
Absolute Minimum, m:	80	110	140
U.S. Customary			
Weaving Distance [2, 3]	**Number of Lanes in Weaving Section [1]**		
	2	**3**	**4**
Absolute Minimum, ft:	260	360	460

Notes:
1 - Equates to the number of lane changes required to reach the right-hand (or outside) lane on the frontage road from the exit ramp.
2 - Distance is measured from the exit-ramp nose to the design queue storage limit (see Figure 43).
3 - Distances are based on the 50th percentile weaving and deceleration distances.

Exit-Ramp-to-Interchange Spacing. Fitzpatrick *et al.*(*41*) conducted a comprehensive evaluation of the spacing required for weaving and queue storage between the exit-ramp and a TUDI. This examination considered the combined effects of weaving volume and distance on frontage road level-of-service. They found that the minimum spacing needed for acceptable operation increased with frontage road volume, exit-ramp volume, and the percentage of ramp drivers turning right at the downstream intersection. Their findings are offered in Table 45 as the recommended minimum exit-ramp-to-interchange spacing for SPUI/F design.

Additional guidance regarding the exit-ramp-to-interchange spacing for SPUI/F is provided in the *Green Book* (*35, p. 858*). The *Green Book* authors recommend an absolute minimum spacing of 200 m (660 ft) and a desirable minimum spacing of 300 m (980 ft). These minimums have been incorporated in Table 45 such that they supercede any shorter length recommended by Fitzpatrick *et al.*(*41*). Through this combination of guidance, the values in Table 45 are rationalized to be appropriate for SPUI/F designs with "typical" signalization.

Auxiliary Lane Length. The length of auxiliary lane needed is dependent on the time required for the left-turn driver to: (1) detect the need to change lanes, (2) accelerate to the frontage road speed while selecting an appropriate gap, and (3) complete the lane-change maneuver. The *Green Book* (*35*) guidance in this regard is directed to freeway lanes and right-hand entrance ramps, the latter being more appropriate for SPUI/F design.

For freeway applications, the *Green Book* (*35, p. 944*) guidance recognizes that most of the detection time noted previously is available prior to the auxiliary lane and that the actual lane-change can take place within the taper at the end of the auxiliary lane. Hence, the *Green Book* indicates that the minimum length of the auxiliary lane is dictated by the larger of two distances: (1) that needed for gap acceptance, and (2) that needed to accelerate to the frontage road speed. This guidance is repeated in Table 46 as the recommended minimum auxiliary lane length for SPUI/F design.

Interchange-to-Entrance-Ramp Spacing. The critical maneuver that will likely dictate this distance is the weaving that occurs between the left-turn, right-turn, and frontage-road through vehicles as each driver maneuvers to position his or her vehicle in an appropriate lane before reaching the entrance ramp.

There is no formal guidance in the literature that describes the minimum interchange-to-entrance-ramp spacing for frontage road applications. For TUDIs, the Arizona DOT's *Roadway Design Guidelines* (*36*) indicates that the entrance ramp nose should be about 150 m (490 ft) downstream from the cross street centerline. However, the potential for intense weaving on the frontage road is far less at the TUDI relative to the SPUI/F because of differences in how vehicles arrive to the weaving section and how the signal affects the timing of their arrival. This characteristic of the SPUI/F suggests that the spacing should exceed that provided for the TUDI. In support of this observation, the *Green Book* (*35, p. 943*) indicates that a minimum spacing of 300 m (980 ft) is needed between successive ramp terminals along a collector-distributor roadway.

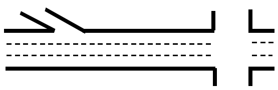
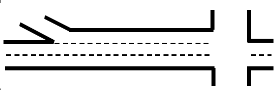
Based on this limited guidance and first-hand observation of the intense weaving activity that can take place along this section of frontage road, it is recommended that 200 m (660 ft) be considered as the absolute minimum interchange-to-entrance-ramp spacing and that 300 m (980 ft) be considered the desirable minimum spacing for SPUI/F design.

Vertical Alignment

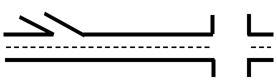
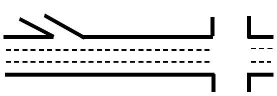
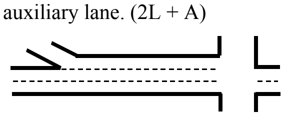
The vertical alignment of the cross street and ramp (or frontage road, if applicable) should be sufficiently flat as to allow the turning driver an unobstructed view of all conflicting traffic streams. This guidance is most appropriate for SPUI/F where the cross street or ramp has crest curvature through the intersection conflict area as such curvature can effectively block the driver's line of sight.

Table 45. Minimum exit-ramp-to-interchange spacing for frontage roads.

Metric

Frontage Road Geometry	Ramp Volume, veh/h	Frontage Road Volume, veh/h[3] 500		1000		1500		2000	
		Minimum Spacing between ramp and interchange,[1,2] m							
		Abs.	Des.	Abs.	Des.	Abs.	Des.	Abs.	Des.
Two continuous through lanes. (2L)	250	200	300	200	300	200	300	200	400
	500	200	300	200	325	200	455	200	585
	750	200	385	200	510	200	640	315	770
	1000	200	570	240	695	370	825	500	955
	1250	300	755	430	885	555	1010	685	1140
Three continuous through lanes. (3L)	250	200	300	200	300	200	315	200	450
	500	200	300	200	300	200	415	250	550
	750	200	300	200	375	215	515	350	650
	1000	200	340	200	475	315	615	450	750
	1250	200	440	275	575	415	715	550	850
Two through lanes plus an auxiliary lane. (2L + A)	250	200	300	200	300	200	300	200	300
	500	200	300	200	300	200	300	200	300
	750	200	300	200	300	200	330	200	400
	1000	200	315	200	385	200	455	200	525
	1250	200	445	200	515	200	585	255	655

U.S. Customary

Frontage Road Geometry	Ramp Volume, veh/h	Frontage Road Volume, veh/h[3] 500		1000		1500		2000	
		Minimum Spacing between ramp and interchange,[1,2] ft							
		Abs.	Des.	Abs.	Des.	Abs.	Des.	Abs.	Des.
Two continuous through lanes. (2L)	250	660	980	660	980	660	980	660	1310
	500	660	980	660	1070	660	1490	660	1920
	750	660	1260	660	1670	660	2100	1030	2530
	1000	660	1870	790	2280	1210	2710	1640	3130
	1250	980	2480	1410	2900	1820	3310	2250	3740
Three continuous through lanes.	250	660	980	660	980	660	1030	660	1480
	500	660	980	660	980	660	1360	820	1800
	750	660	980	660	1230	710	1690	1150	2130
	1000	660	1120	660	1560	1030	2020	1480	2460
(3L)	1250	660	1440	900	1890	1360	2350	1800	2790
Two through lanes plus an auxiliary lane. (2L + A)	250	660	980	660	980	660	980	660	980
	500	660	980	660	980	660	980	660	980
	750	660	980	660	980	660	1080	660	1310
	1000	660	1030	660	1260	660	1490	660	1720
	1250	660	1460	660	1690	660	1920	840	2150

Notes:

1 - "Abs.:" Absolute minimum spacing. "Des.:" Desirable minimum spacing.

2 - Add the following distance to any underlined value in the table if the percent of exit ramp vehicles that turn right at the downstream intersection exceeds 50 percent, *2L*: 70 m (230 ft); *3L*: 135 m (445 ft); *2L+A*: 155 m (510 ft).

3 - Frontage road volume is measured *prior* to the exit ramp.

Table 46. Minimum auxiliary lane length.

Metric						
Frontage Road Design Speed, km/h	Curve Design Speed, km/h					
	40	50	60	70	80	90
	Minimum Auxiliary Lane Length, m					
50	90	90	—	—	—	—
60	90	90	90	—	—	—
70	90	90	90	90	—	—
80	135	100	90	90	90	—
90	210	175	135	90	90	90

U.S. Customary						
Frontage Road Design Speed, mph	Curve Design Speed, mph					
	25	30	35	45	50	55
	Minimum Auxiliary Lane Length, ft					
30	300	300	—	—	—	—
35	300	300	300	—	—	—
45	300	300	300	300	—	—
50	440	330	300	300	300	—
55	690	570	440	300	300	300

Several types of sight distance are needed within the SPUI/F conflict area. Given the complexity of the SPUI, Messer *et al.* (*5*) recommended that the decision sight distance be used for vertical alignment control. Stopping sight distance is also needed as is intersection sight distance. The magnitude of all three sight distance types were previously shown in Table 39.

As applied to crest curve design, two factors must be considered when determining the minimum curve length. The first is the controlling sight distance and the second is the corresponding height of object. For intersection sight distance, the appropriate object height is that of another vehicle (i.e., 1.3 m [4 ft]). In contrast, the *Green Book* recommends the use of a 0.15 m (0.5 ft) object height for stopping sight distance and decision sight distance.

Based on an examination of the three sight distance types and their associated object heights, decision sight distance requires the flattest curvature and, therefore, controls the design. Minimum rates of crest curvature (*K*) that provide this sight distance are listed in Table 47. The values shown apply to both the cross street and ramp (or frontage road, if applicable) alignments within the interchange conflict area.

It should be noted that the rates listed in Table 47 are quite large and may require special consideration of longitudinal drainage for the Type I crest curve (i.e., "plus" grade followed by a "minus" grade). It should also be noted that the resulting curve lengths will provide stopping sight distance to a 0.0 m object height. This capability will allow drivers to see and react to the pavement markings within the interchange conflict area (which is especially important at the

SPUI/F due to its unusual markings and stop line locations). The resulting curve lengths will also yield sufficient sight distance for frontage road, right-turn drivers to verify the safety of a yielding right-turn entry to the cross street.

Table 47. Minimum rate of vertical curvature for crest curves at SPUI/F.

Metric							
Rate of Vertical Curvature (K),[1] length (m) per % of A	**Design Speed, km/h**						
	30	**40**	**50**	**60**	**70**	**80**	**90**
Minimum Rate for Suburban:	25	46	64	104	143	188	254
Minimum Rate for Urban:	42	68	99	137	188	246	231

U.S. Customary							
Rate of Vertical Curvature (K),[1] length (ft) per % of A	**Design Speed, mph**						
	20	**25**	**30**	**35**	**45**	**50**	**55**
Minimum Rate for Suburban:	80	150	210	340	470	615	835
Minimum Rate for Urban:	140	225	325	450	615	805	760

Notes:
1 - Rates are based on decision sight distance and a 0.15 m (0.5 ft) object height.

Cross Section

General

This section describes a procedure that can be used to estimate the number of lanes in the cross street and frontage road cross sections. It is based on the "critical movement analysis" (CMA) approach that forms the basis for the signalized intersection analysis procedure in the *Highway Capacity Manual* (*13*). The CMA approach, and its applicability to the SPUI/F, is described in Chapter 4.

The procedure consists of three steps that are completed in sequence. Steps 2 and 3 are repeated as needed, until an acceptable agreement is reached between the number of lanes provided and the maximum service volume. The steps are described as follows:

Step 1. Identify Movement Volumes and Lane Assignments. For this step, the design hourly volumes v are identified for the left-turn and through-plus-right-turn movements. These movements are numbered 1 through 8 using the convention identified in Figure 44.

Also identified in this step are the "trial" number-of-lanes n for each of the eight movements. This number will be checked and possibly revised in subsequent steps. As a preliminary estimate, one lane can be assumed to serve a maximum of 400 veh/h/ln. Thus, a movement with 350 veh/h would be assumed to need one lane; a second movement with 450 veh/h would be assumed to need two lanes.

In general, the number of lanes selected for a particular movement should be "balanced" with its complementary movement (e.g., v_1 is complementary with v_5, v_2 is complementary with v_6). In this regard, complementary movements should have an equal number of lanes; that movement of a pair that requires the largest number of lanes should determine the number of lanes for both movements. Also, the number of left-turn lanes used should never exceed the number of lanes provided on the receiving (or departure) leg of the interchange.

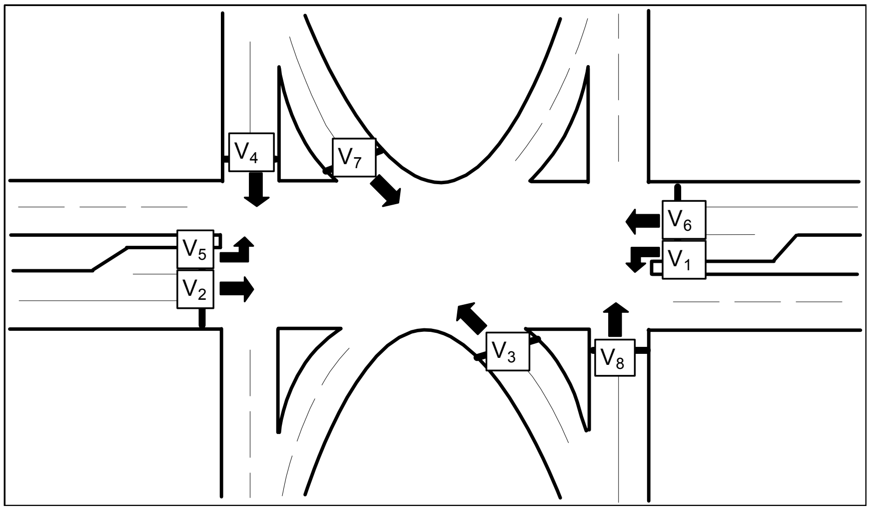


Figure 44. Movement numbers for critical volume summation.

Step 2. Determine the Critical Lane Volume. During this step, the movement volume and lane estimates from Step 1 are used with Equations 5, 6 and 7 to estimate the sum-of-critical-volumes.

(15)

$$A = Larger\ of: \left[\frac{v_1}{n_1} + \frac{v_2}{n_2}\ ;\ \frac{v_5}{n_5} + \frac{v_6}{n_6}\right]$$

(16)

$$B = Larger\ of: \left[\frac{v_3}{n_3} + \frac{v_4}{n_4}\ ;\ \frac{v_7}{n_7} + \frac{v_8}{n_8}\right]$$

(17)

$$v_c = A + B$$

where:

v_i = volume of movement i (i = 1, 2, ... 8), veh/h;
n_i = number of lanes serving movement i (i = 1, 2, ... 8);
A = critical volume for the cross street movements, veh/h/ln;
B = critical volume for the frontage road movements, veh/h/ln; and
v_c = sum of critical volumes, veh/h/ln.

Step 3. Evaluate Service Volume and Adjust Lanes. For this step, the sum-of-critical-volumes v_c from Step 2 is compared with the maximum service volume from Table 48. These service volumes correspond to level-of-service "D" based on the delay guidelines provided in the *Highway Capacity Manual* (*13*). They were computed using the analysis procedure described in Chapter 4.

If the sum-of-critical-volumes exceeds the maximum service volume, then the number of lanes needs to be increased for one or more movements and Steps 2 and 3 repeated. If the sum-of-critical-volumes is much less than the maximum service volume (say, less than 80 percent of the maximum service volume), then the number of lanes should be reduced and Steps 2 and 3 repeated. When the sum-of-critical-volumes is less than the maximum service volume but more than 80 percent of the maximum service volume, then the procedure can stop and the number-of-lanes should form the basis for the SPUI/F cross section design.

Table 48. Maximum service volumes for level-of-service D operation.

Metric								
Variable	**Ramp Separation[1], m**							
	60	**70**	**80**	**90**	**100**	**110**	**120**	**130**
Max. Service Volume, veh/h/ln [2,3,4]	1200	<u>1200</u>	1100	1100	1000	1000	900	900

U.S. Customary								
Variable	**Ramp Separation[1], ft**							
	200	**230**	**260**	**300**	**330**	**360**	**390**	**430**
Max. Service Volume, veh/h/ln [2,3,4]	1200	<u>1200</u>	1100	1100	1000	1000	900	900

Notes:
1 - Distance between the two frontage road center lines, as measured along the cross street.
2 - Based on an assumed saturation flow rate of 2,000 veh/h/ln.
3 - Volumes shown correspond to an average control delay of 55 s/veh which is the upper limit of level-of-service "D."
4 - The <u>underlined</u> value is referenced in the example application.

Example Application. Consider a SPUI/F that has a proposed 70 m (230 ft) ramp separation and the volumes listed in Table 49. Applying the "400-veh/h/ln" rule yields the number lanes shown in Column 4 of this table. One exception is the northbound left-turn. Its volume does not justify two lanes; however, the complementary left-turn movement (i.e, the southbound left-turn) does require two lanes. Thus, both the north and southbound left-turn movements are assigned two lanes.

Equation 5 requires finding the larger of two volume pairs (e.g., $v_1/n_1 + v_2/n_2$ and $v_5/n_5 + v_6/n_6$). The total for each pair is listed in Column 5, with that for Movements 5 and 6 representing the larger pair (i.e., 584 veh/h/ln) for the cross street approach. Equation 6 can be used to find that Movements 3 and 4 represent the larger pair with a critical sum of 550 veh/h/ln. From Equation 7, the sum-of-critical-volumes is computed as 1,134 veh/h/ln. Because this sum is less than the maximum service volume of 1,200 veh/h/ln (from Table 48) but not less than 960 veh/h/ln (= 0.8 x 1,200), the number-of-lanes identified in Column 4 of Table 49 will be used for cross section design.

Table 49. Example critical volume computation.

Approach	Movement	Volume, veh/h	Lanes	Volume Pairs	Critical Volume veh/h/ln
Cross Street	1. Westbound left-turn	475	2	574	584
	2. Eastbound through + right	1010	3		
	5. Eastbound left-turn	475	2	<u>584</u>	
	6. Westbound through + right	1040	3		
Frontage road	3. Northbound left-turn	350	2	<u>550</u>	550
	4. Southbound through + right	750	2		
	7. Southbound left-turn	450	2	513	
	8. Northbound through + right	575	2		
Sum of Critical Volumes, veh/h/ln:					1134

If the SPUI/F were to have a 90 m (300 ft) ramp separation distance, the maximum service volume would be 1,100 veh/h/ln. As the sum-of-critical-volumes from Table 49 exceeds this amount, additional lanes would be needed for a wider interchange.

<u>Cross Street Geometry</u>

The *Green Book* (*35, p. 858*) notes that it may be beneficial to have a nominal median width in the cross street cross section. The benefit stems from the fact that a median can allow the cross street stop lines to be brought into the interchange conflict area further than they could otherwise if no median were provided. This ability to locate the opposing stop lines closer together provides an operational benefit by minimizing clearance distances for the left-turn and through movements and their corresponding all-red interval durations. The manner in which this benefit is achieved is illustrated in Figure 45.

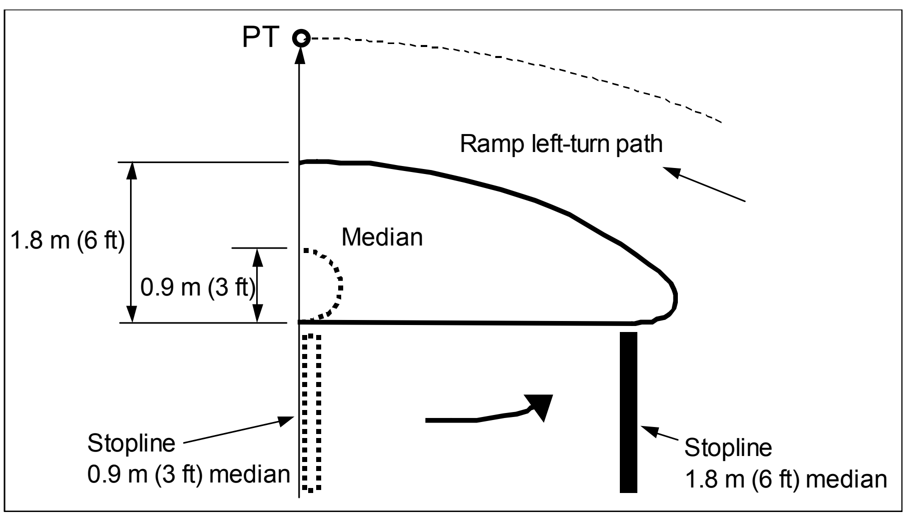


Figure 45. Effect of median width on cross street stop line location.

As indicated in Figure 45, the 1.8 m (6 ft) median width allows an asymmetric median nose to be used to contour the ramp left-turn path and, thereby, move the cross street stop line closer to the center of the conflict area. In contrast, a 0.9 m (3 ft) median width would likely be associated with a narrower cross street width which would require a more distant location for the stop line (see dashed lines).

The benefit of providing a cross street median is illustrated in Figure 46. The trend line for the 60 m (200 ft) ramp separation indicates that the total clearance time is at a minimum when a 1.8 to 2.5 m (6 to 8 ft) median width is provided. Larger median widths tend to widen the cross street and increase the clearance time. Smaller median widths tend to push the cross street stop lines excessively far away from the center of the conflict area (and increase the clearance time).

The trends in Figure 46 indicate that the benefit of providing a median is noticeably reduced with wider SPUI/F. Nevertheless, a 1.5 to 2-m (5 to 6.5 ft) median width is appropriate and consistent with typical design practice for all SPUI/F designs.

Frontage Road Geometry

The *Green Book* (*35, p. 858*) suggests that the right-turn movement on the SPUI/F frontage road approach should be provided an exclusive turn lane. This lane removes the right-turn vehicles from the interchange's signalized movements and thereby, improves its operation.

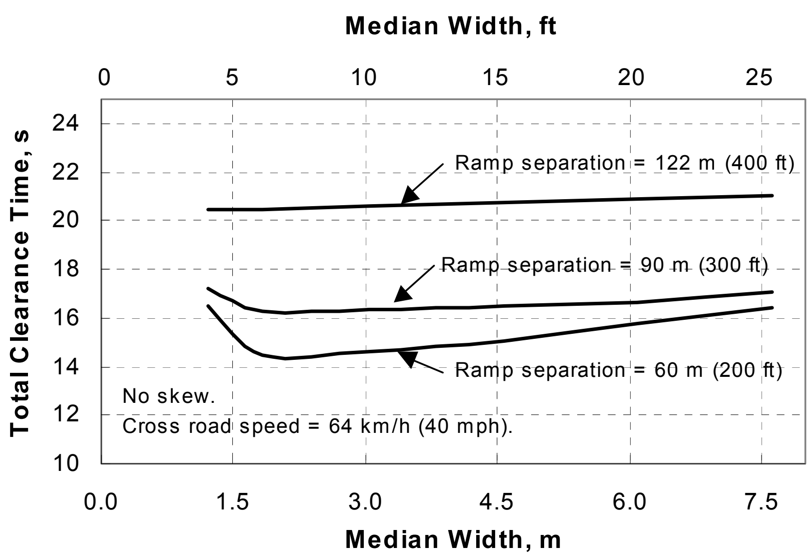


Figure 46. Effect of median width on total clearance time.

The right-turn movement can be free (i.e., uncontrolled), yield controlled, or signal controlled. A free right turn requires a turning roadway design with a radius of 50 m (160 ft) or more and an auxiliary lane on the cross street. The auxiliary lane would be 60 m (200 ft) or more in length. An adjacent signalized intersection on the cross street should be 150 m (490 ft) or more downstream of the frontage road to insure that there will be adequate distance for the right turn-related weaving activity on the cross street.

A yield or signal controlled right turn should have an exclusive turn lane in the frontage road cross section. At its intersection with the cross street, the exclusive lane should have a curb return with a 3-centered-curve design to facilitate low speed turns by heavy vehicles. If space permits, a triangular island separating the right-turn and adjacent through lanes at the cross street is desirable to accommodate pedestrian traffic. The gap-selection process associated with the right-turn maneuver is made more difficult when the “entry angle” is less than 60 degrees. Entry angle is that angle between the center line of the cross street and the turn vehicle’s longitudinal axis (when at the stop line). An acceptable angle is achieved when the right-turn lane is designed to yield an effective travel-path radius of 25 m (80 ft) or less (as measured at the center of the path).

A signal controlled right-turn lane should be served concurrently with the frontage road through movement phase. Additional service can be provided during the cross street left-turn phase (via a phase overlap function). Signal control requires detection in the right-turn lane which effectively extends the boundaries of the intersection conflict area. In this manner, signal control has the potential to reduce interchange capacity (relative to yield or free right-turn operation) by taking cycle time from other movements and by increasing the total clearance time (via an increased conflict area).

U-Turn Lanes

The *Green Book* (*35, p. 858*) indicates that an exclusive U-turn lane for frontage road traffic may be desirable to expedite traffic movement from one side of the major road to the other. The operational efficiency of a U-turn lane would logically be maximized at SPUI/F s in continuous frontage road systems and dense commercial development. The disadvantage of the U-turn lane is that it adds to the cost of the bridge structure, regardless of whether the SPUI/F has an underpass or overpass design.

The provision of a U-turn lane requires the provision of an exclusive lane in advance of (and beyond) the cross street along the frontage-road approach (and departure) leg. The exclusive lane in advance of the cross street is needed to separate the left-turn and u-turn vehicles. Without this separation, the U-turn vehicles would incur as much delay as the left-turn vehicles. The exclusive auxiliary lane on the departure leg provides a length of roadway for the U-turn driver to merge safely into the frontage-road traffic stream.

Figure 47 illustrates a SPUI/F with U-turn lanes. Exclusive lanes are provided in advance of (and beyond) the cross street along the frontage road. It should also be noted that this design provides lane balance on the frontage roads at the merge point of the cross street left-turn and frontage road through movements.

TRAFFIC CONTROL DEVICE DESIGN GUIDELINES

Traffic control design guidelines for a SPUI/F is presented under three separate categories:

- Signing
- Pavement markings
- Signalization

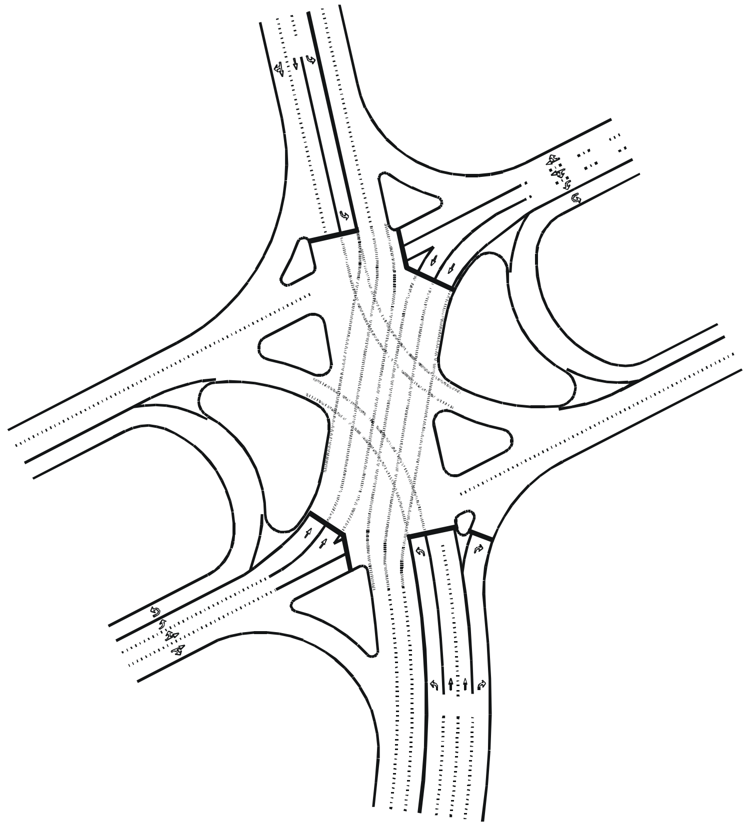


Figure 47. Interchange layout with frontage-road U-turn lanes.

The overall design of traffic control is heavily dependent on and based upon information contained in the *MUTCD (37)*. The Arizona Department of Transportation has published additional guidelines for use in their jurisdiction.

Definitions

The *MUTCD (37)* uses verbs “shall”, “should” and “may” to describe conditions for traffic devices. The meanings are defined under the following text headings contained in the *MUTCD (37):*

1. Standard – Statements under this heading are required, mandatory, or specifically prohibitive practices regarding traffic control devices. The verb “shall” is typically used to describe the mandatory practices in this heading.

2. Guidance – Statements under this heading are recommended, but not mandatory, practices. The verb “should” is typically used under this heading to recommend advised practices.

3. Option – Statements under this heading are permissive conditions that carry no requirements or recommendations. The verb "may" is typically used under this heading.
4. Support – Statements under this heading are for information purposes only, and do not convey any degree of recommendations, prohibitions, authorizations, or other guidance. The verbs "shall", "should", and "may" are not used in Support statements.

Signing

This section describes the signing layout for a SPUI/F. Merritt *et al. (43)* noted that signing at SPUIs is generally similar to that of diamond interchanges, with the exception that SPUIs require larger legends and advance signing due to the higher turning speeds in the interchange. Due to the complex nature of the SPUI design, it is crucial that signing clearly relays travel directions to motorists.

Signing can be categorized as being regulatory, warning, or guide signs. Regulatory signs at a SPUI typically include: One-Way, Do Not Enter, lane-use controls, and turn prohibitions. Some regulatory signs unique to SPUIs are discussed below:

<u>Advanced Overhead Signing</u>

Advance overhead signing is regularly used on both crossroads and off-ramp approaches at SPUIs to provide guidance for approaching vehicles. Merritt *et al. (43)* recommends advanced overhead signing which provides lane-use control for both the off-ramp and crossroad approaches at SPUI interchanges. Although *MUTCD (37)* language does not identify overhead sign installations as a "shall" condition at off-ramp and crossroad approaches, it does provide the option of consideration for these signs based on criteria that the SPUI design meets:

> Option:
>
> The following conditions (not in priority order) may be considered in an engineering study to determine if overhead signs should be used:
>
> A. Traffic volume at or near capacity
> B. Complex interchange design
> C. Three or more lanes in each direction
> D. Restricted sight distance
> E. Closely-spaced interchanges
> F. Multi-lane exits
> G. Large percentage of trucks
> H. Street lighting background
> I. High-speed traffic
> J. Consistency of sign message location through a series of interchanges
> K. Insufficient space for ground-mounted signs
> L. Junction of two freeways
> M. Left exit ramps

There are several conditions for determining overhead sign usage which are met by the design of a SPUI/F. SPUI designs could be classified as complex interchanges (see B), approaches typically contain three or more lanes in each direction (see C), and sight distance is often restricted (see D). Therefore, overhead signing is recommended when practical for use at both the off-ramp and crossroad approaches to SPUI/Fs. Figure 48 displays typical overhead signing for an off-ramp approach at a SPUI/F. When the overhead signing is used to direct each individual lane, individual signs should be centered over respective lanes.

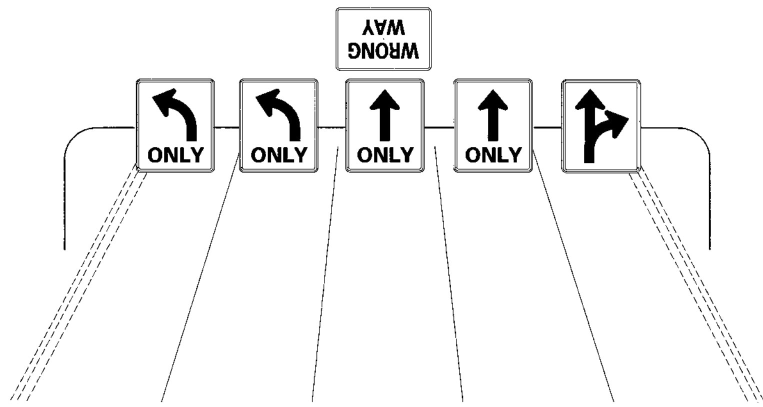


Figure 48. Typical overhead signing at SPUI off-ramp.

The *MUTCD (37)* states that overhead signing shall provide a vertical clearance of not less than 5.1 m (17 ft) from the roadway to the bottom of the signing, light fixture, or any other part of the overhead portion of the structure.

Overhead signing on the cross street may not be prudent at locations where a narrow median or other conditions may prevent the placement of posts at their optimal locations. *The Green Book (35)* recommends a minimum 0.5 m (1.5 ft) clearance from curb to face of object. Assuming a structural support width of 0.5 m, the minimum recommended median width is therefore 1.5 m (5 ft). Based on *MUTCD (37)* guidelines, the lateral offset from the edge of the shoulder (or if no shoulder exists, from the edge of the pavement) must be at least 1.8 m (5.9 ft).

Merritt *et al. (43)* states that decision sight distance for the crossroad approach to the SPUI should be used as the minimum signing design criteria. Based on this guidance, Green Book values for decision sight distance are shown previously in Table 39. When appropriate, decision sight distance should be considered in the placement of overhead signs on off-ramps. When decision sight distance is used to locate signing, it should be the distance from the point of application to where the sign can be read.

Signing at both overpass and underpass SPUIs are generally the same. However, sign placement at overpass SPUIs presents a potential problem due to the shadowing that occurs under the bridge during daylight hours. Signing placed under the bridge structure (or close enough to provide shadowing from the sun) is difficult to read for drivers entering the interior portion of the interchange. This is due in part to the geometric differences between a SPUI and a TUDI. The presence of separate left-turn channelization on SPUI/Fs often provides a need for additional signing on their adjacent islands. These islands are often close enough to the bridge structure to "shade" the sign in daylight, significantly reducing its visibility.

The *MUTCD (37)* states the following with regards to wrong-way signing at interchange ramps:

Standard:

At interchange exit ramp terminals where the ramp intersects a crossroad in such a manner that wrong-way entry could inadvertently be made, the following signs shall be used (see Figure 2E-31):

A. At least one ONE WAY sign for each direction of travel on the crossroad shall be placed where the exit ramp intersects the crossroad.

B. At least one DO NOT ENTER sign shall be conspicuously placed near the end of the exit ramp in positions appropriate for full view of a road user starting to enter wrongly.

C. At least one WRONG WAY sign shall be placed on the exit ramp facing a road user traveling in the wrong direction.

As an option, additional ONE WAY and/or WRONG WAY signs may be placed on the off-ramp or crossroad to further alert the wrong-way driver. Messer *et al*. *(5)* advises placing one DO NOT ENTER sign on each end of the off-ramp left-turn lane at the stop line. Figure 49 shows typical signing at a SPUI/F.

Warning signs at SPUIs may include Signal Ahead signs and Merge signs. A merge sign along the departing frontage road is shown in Figure 49.

Guide signs at SPUIs are an important tool for conveying travel directions to motorists, especially for those who are unfamiliar with the area roadways. Guide signs at a SPUI, as illustrated in Figure 49 include street names signs, route markers with turn arrows, and overhead guide signs where needed.

Pavement Markings

General

This section describes design considerations for pavement markings at a SPUI/F where compliance with the *MUTCD (37)* is critical. Merritt *et al. (43)* noted that pavement markings at SPUIs are similar to that of diamond interchanges, with the exception of striping requirements for left turns at SPUIs. Figure 50 presents typical pavement markings at a SPUI/F.

It should be noted that no attempt is made with pavement markings in Figure 50 to comply with the AASHTO lane balance recommendations summarized in Figure 43. Consideration could be given to changing the frontage road through-right lane to a right turn only. This would leave only two through frontage road lanes departing the cross road intersection which would merge with the two left turn lanes from the cross road. If this were done, the left-most through lane beyond the cross road would be marked out and the inside through lane moved to the right to occupy what is now the center lane beyond the intersection. Figure 51 schematically depicts this design. According to the *MUTCD (37),* the taper length for this transition should be as shown in Table 50.

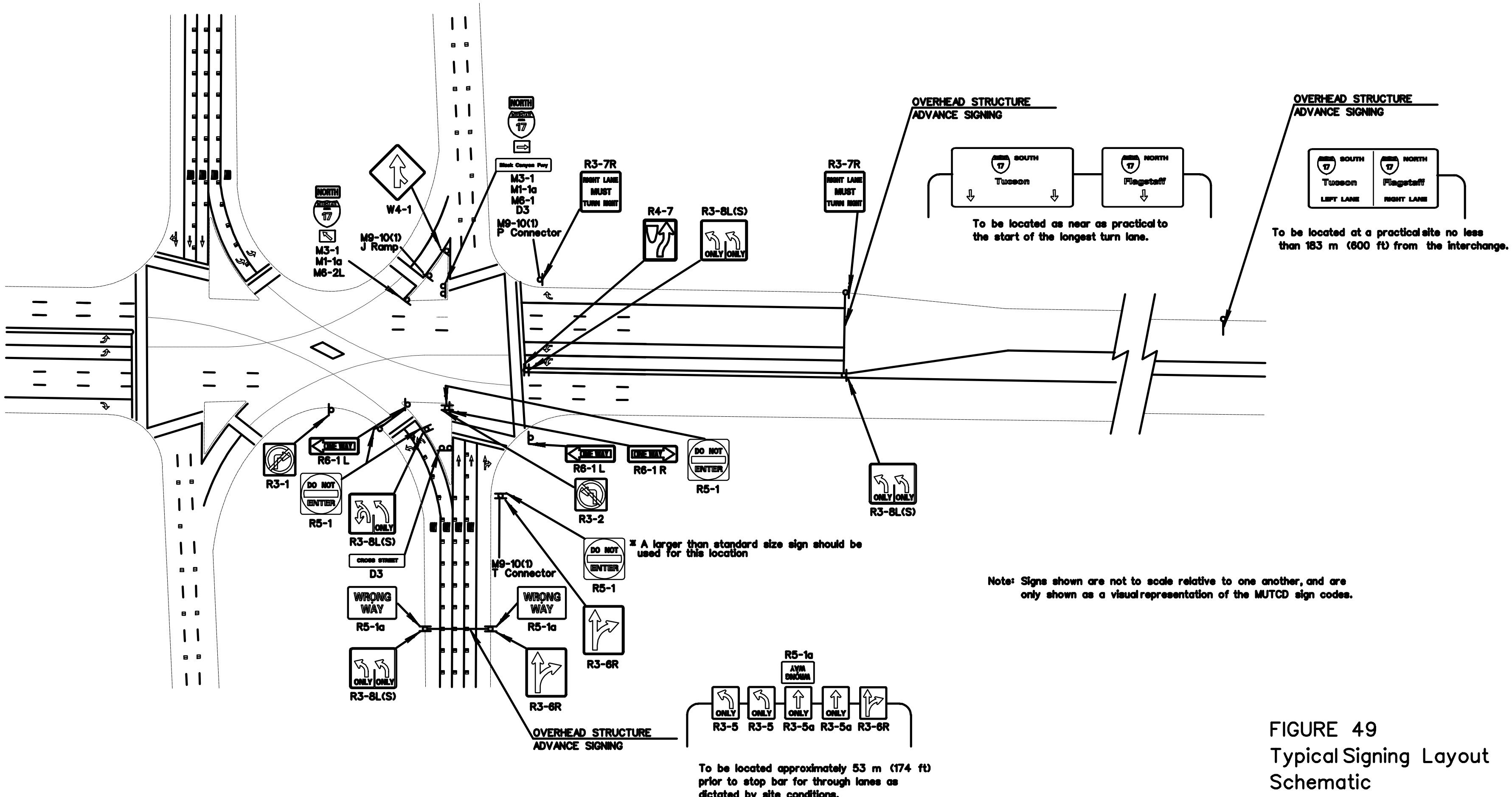


FIGURE 49
Typical Signing Layout Schematic
(Underpass SPUI/F)

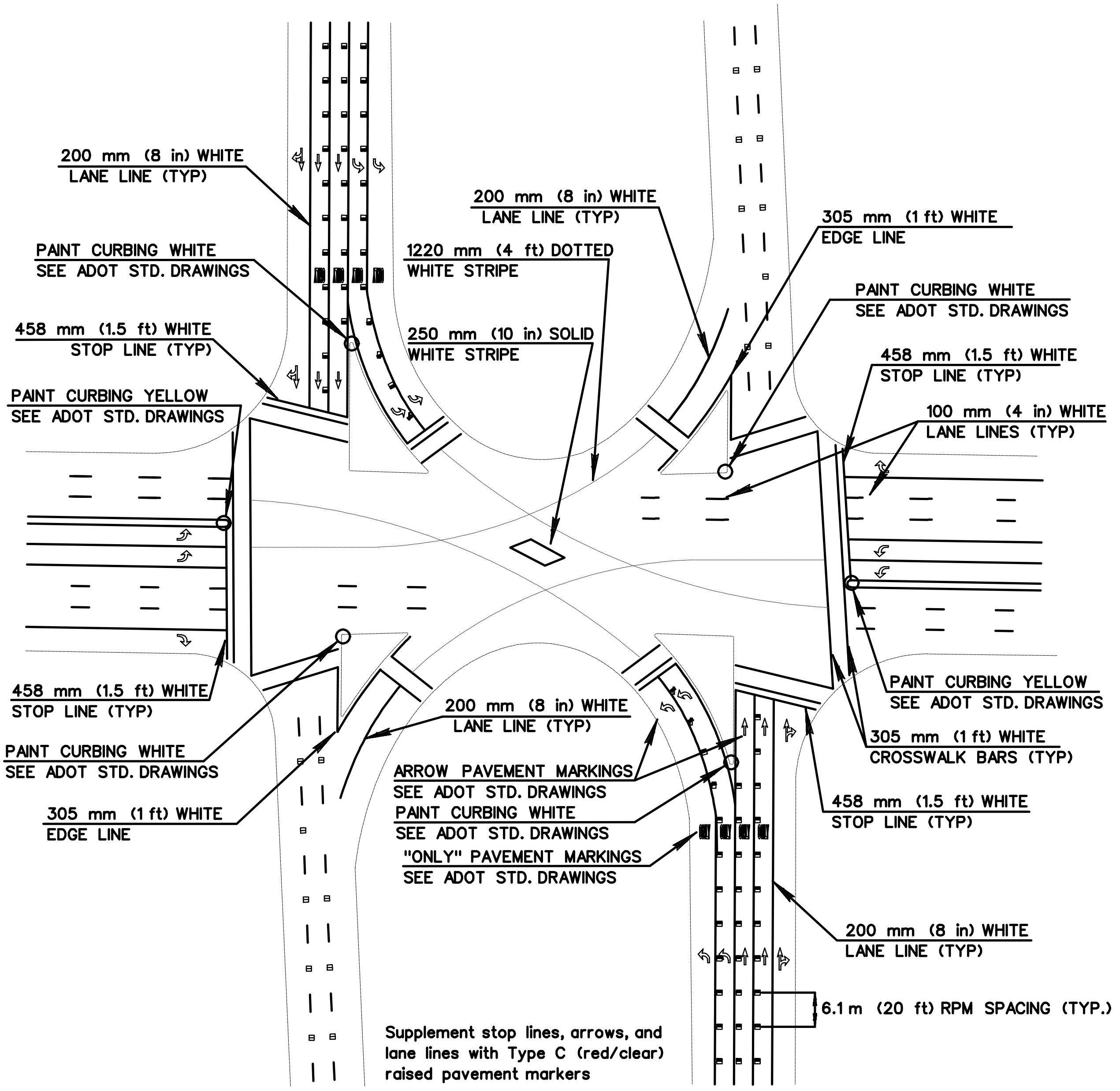


FIGURE 50
Typical Pavement Markings Schematic (Underpass SPUI/F)

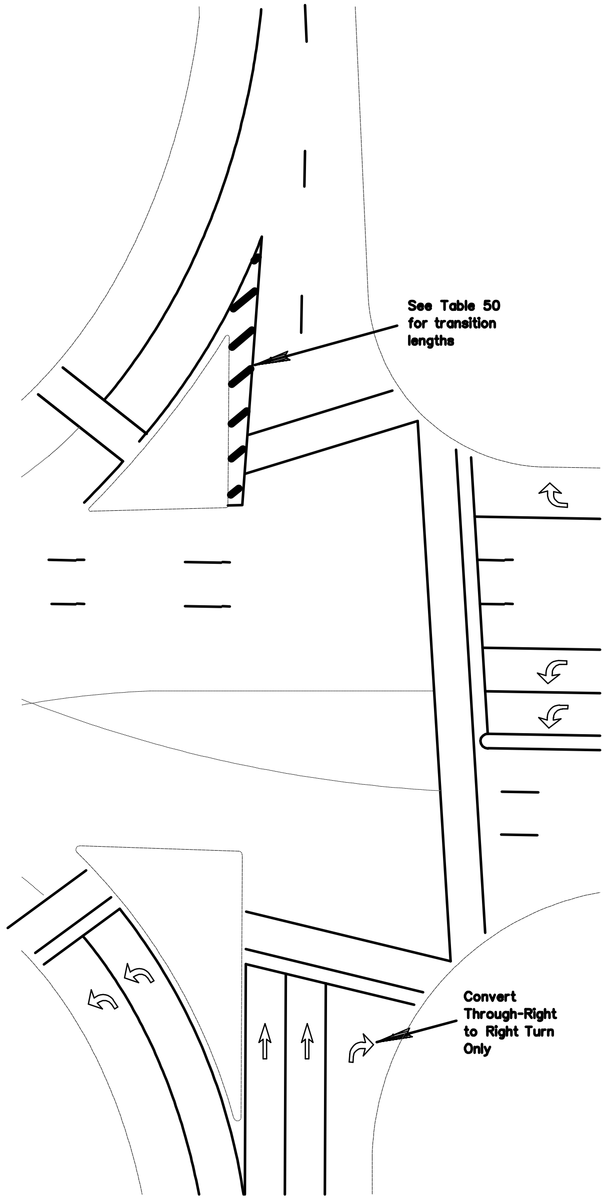


FIGURE 51
Transition to Achieve
Lane Balance

Table 50. Taper Length for Lane Transition.

Metric[1]					
	Posted Speed Limit (km/h, mph)				
	60	65	70	75	80
Taper Length, m	84	98	156	167	179
U.S. Customary[1]					
	35	40	45	50	
Taper Length, ft	245	320	540	600	

[1] length based on MUTCD (*37, p. 3B-22*)

Crossroad

Messer *et al. (5)* notes that the area underneath an overpassing bridge is the area of principal concern, primarily for the left turns. The visibility of pavement marking along the left-turning path is critical to driver safety. There should be at least one lane line marking each left-turn path throughout the central area. The presence of dual left turns requires even greater positive guidance of turning paths. Merritt *et al. (43)* emphasizes the importance of high-quality pavement markings along the left-turn path for the driver. Messer *et al. (5)* notes that there doesn't appear to be a need to mark the outer path of well designed turning paths, although markings are often set in this manner. Care should be taken to not "overdesign" the pavement marking layout at the internal portion of the SPUI. Too many markings have been shown to be confusing to motorists, Messer *et al. (5)* reports.

Frontage Road

Lane-use arrows should be in each lane to provide guidance for off-ramp traffic, and discourage potential wrong-way traffic. In addition*, MUTCD (37)* language states that lane-use arrows should be placed in each lane of an exit ramp when geometrics are such that wrong-way movements would not be difficult.

Pavement Marking Lights

Federal Aviation Administration pavement marking lights have been used at some SPUI locations in the US to delineate SPUI lane lines through the central intersection area, primarily the paths of left-turn movements. These lights illuminate when the related movement has the green signal, and turn off when the phase ends. However, these lights are not recommended for usage. They are relatively expensive and difficult to maintain. Messer *et al. (5)* reports of no evidence that these lights have improved overall safety at an interchange.

Raised Pavement Markers

Raised pavement markers (RPM) are used to supplement roadway striping, and provide guidance for motorists especially at night. RPM are recommended at the on- and off-ramps of a SPUI/F. Figure 52 presents an ADOT Type C maker which should be used at the frontage road approaches to the crossroad. This red/clear RPM provides positive guidance for traffic while rendering further warning for the wrong-way driver. Furthermore, due to the potential of driveway traffic entering the frontage road in the wrong direction, consideration should be given to extending the use of the type C marker farther along the frontage roads. The use of RPM's in area with significant snow fall may not be possible due to conflicts with snow removal requirement.

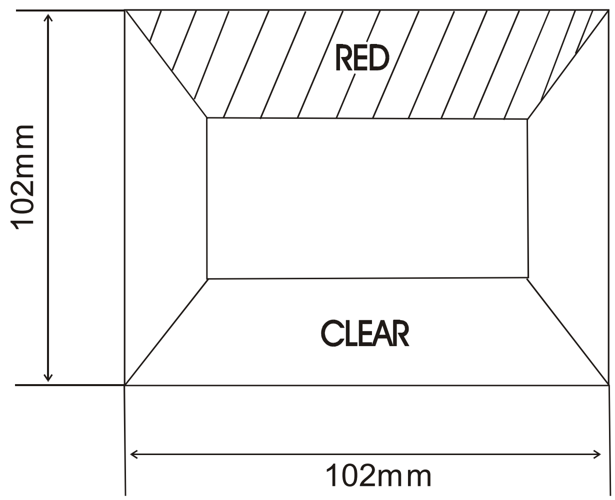


Figure 52. Red/clear RPM (ADOT Type C Marker) for use on one-way frontage roads

The Type C marker may also be used to supplement stop lines and lane use arrows where wrong-way maneuvers are a concern. This would be accomplished by outlining the stop lines and lane arrows with the Type C marker.

Traffic Signals

General

This section describes design considerations for traffic signalization at a SPUI/F. Traffic signal designs at SPUIs should be developed to comply with the *MUTCD (37).* Figure 53 presents a typical traffic signal layout used by ADOT at an underpass SPUI/F. General requirements that apply to and are of particular importance to traffic signal at SPUIs include:

1. At least two signal indications are required for each approach to the intersection.
2. Two signal heads with arrow indications are preferred for turning traffic.
3. Signal head must be place within 46 m (150 ft) of the stop line. If signal heads are beyond 150 ft, supplement signal heads at or near the stop line are required.

Figure 53 shows a signal head for each through lane on the arterial street with a supplemental head located on the channelizing island to the right of the approach. The supplemental signal head is required due to the distance between the stop line and the overhead mounted signals. Figure 53 also shows an overhead signal head for the arterial roadway left

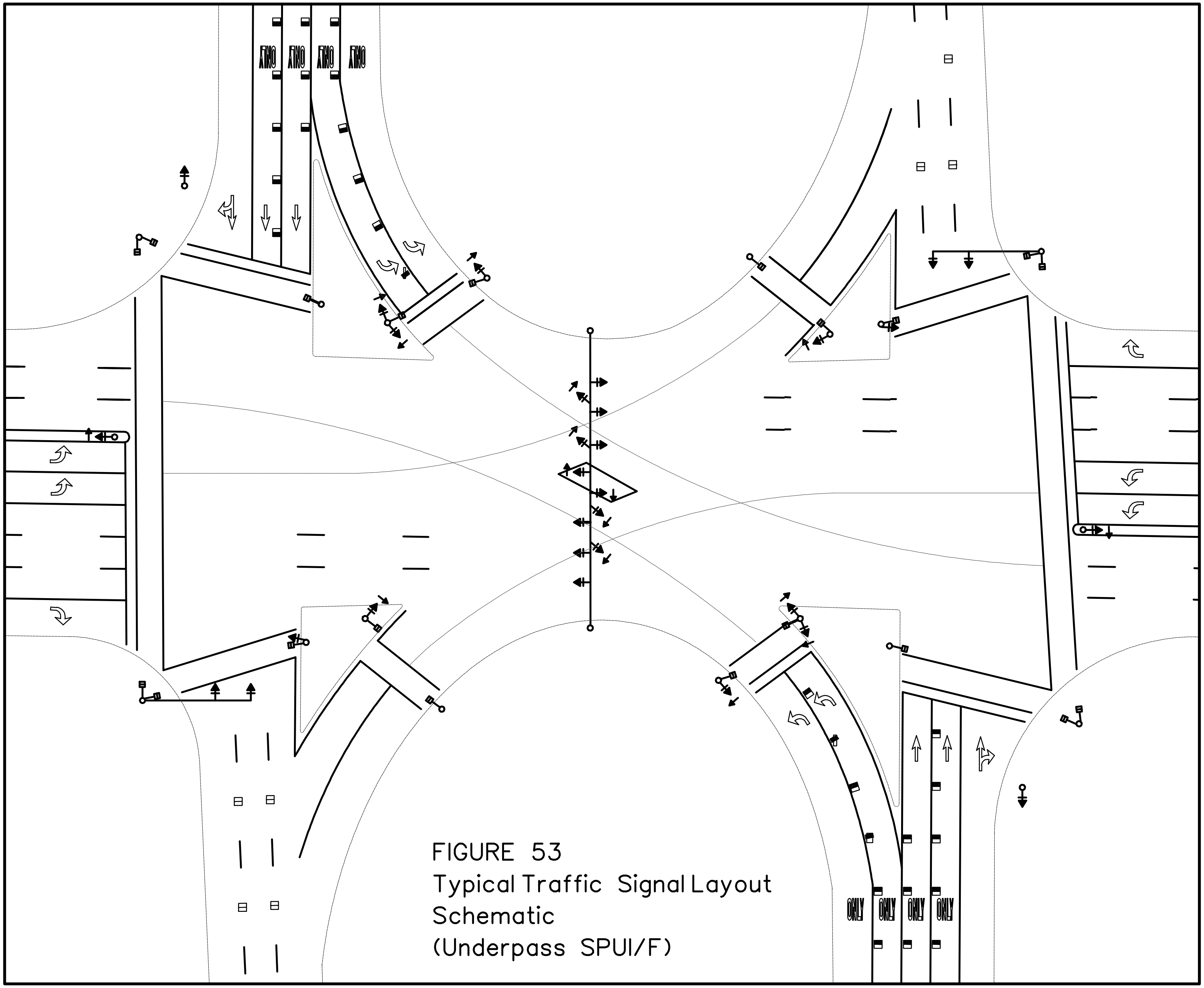


FIGURE 53
Typical Traffic Signal Layout
Schematic
(Underpass SPUI/F)

turning traffic, a near side supplement head on the left side of the road at the stop line and a pull-through head on the far channelizing island. For the frontage road traffic, two overhead signal heads are shown along with a supplemental near side head located near the stop line. Figure 53 also shows two overhead signal heads, two near side sign signal heads located at the stop line, and a pull-through head located on the far channelizing island for left turning traffic from the frontage road.

Signal Heads

The primary consideration in signal head placement should be the visibility to the approaching motorist. The visibility of signal heads for left turns from the off-ramp is a significant aspect of the overall safety of a SPUI/F. Sight control values cited in the *MUTCD (37)* are listed in Table 51, for a minimum of two signal faces. The *MUTCD (37*) advises that if the minimum sight distance requirements cannot be met, a sign "shall" be installed to warn approaching motorists of a traffic signal.

Table 51. Minimum sight distance requirements.

85^{th}-Percentile Speed, km/h	**Minimum Sight Distance, m**
30	50
40	65
50	85
60	110
70	140
80	165
90	195
100	220

Source: MUTCD, page 4D-23.

Dorothy *et al. (4)* noted the placement of traffic signal heads varies greatly from state to state, as well as by the geometric design of the SPUI (SPUI/n vs. SPUI/F). Messer *et al. (5)* reports on designs placing one signal over each left turning lane on the off-ramp, versus one signal head for the entire left-turning movement. The use of individual signal heads for each lane is recommended for this movement, so as to provide additional guidance to the driver. Merritt *et al. (43)* discusses a "pull through" signal which can be placed on the opposite triangular island of an approach when travel distances through the intersection are relatively long. Additionally, an advance signal is often required on the near-side triangular island. Signal heads can be canted to provide direction to the approaching motorists. Sight triangle distances and sight lines from the driver to the traffic signals should be checked for conformance with the *MUTCD (37)*. Head should be within 46 m (150 ft) of the stop line and within 20 degree cone of vision for the driver at the stop line. Supplement heads are required if this cannot be achieved.

The location of signal poles and heads on the crossroad is dependent on whether a SPUI is an overpass or underpass:

- Overpass SPUIs. At an overpass, a signal head cluster is normally mounted on the bridge structure, for use of off-ramp left turns, and crossroad movements (Figure 54). The edge of the bridge abutment should be checked for sight distance conflicts with the off-ramp left turn signal heads. Placement of signal heads to the underside of the bridge is generally avoided due to poor visibility.

- Underpass SPUIs. At an underpass, an overhead tubular beam is used on the bridge to provide traffic control for off-ramp left turns and crossroad movements (Figure 53)

Signal Lenses

The MUTCD states the following with regards to selection of signal lenses:

Standard:

There shall be two nominal diameter sizes for vehicular signal lenses: 200 mm (8 in) and 300 mm (12 in).

Three-hundred millimeter (12 in) signal lenses shall be used:

A. For signal indications for approaches (see definition) in Section 4A.02) where road users view both traffic control and lane-use control signal heads simultaneously.

B. If the nearest signal face is between 35 m (120 ft) and 45 m (150 ft) beyond the stop line, unless a supplemental near-side signal face is provided.

C. For signal faces located more than 45 m (150 ft) from the stop line

D. For approaches to all signalized locations for which the minimum sight distance in Table 4D-1 cannot be met.

E. For arrow signal indications.

> A 200 mm (8 in) signal lens for a CIRCULAR RED signal indication shall not be used in combination with 300 mm (12 in) signal lens for a CIRCULAR GREEN signal indication or 300 mm (12 in) signal lens for a CIRCULAR YELLOW signal indication.

In addition, *MUTCD* language states that 300 mm (12 in) should be used where:

- The 85th percentile speed on the approach exceeds 40 mph.
- A traffic control signal might be unexpected.
- The approach has no curb or gutter and signals are post-mounted only.
- There is a significant amount of elderly drivers.

It is recommended that all signal heads are 300 mm (12 in) at SPUI/Fs.

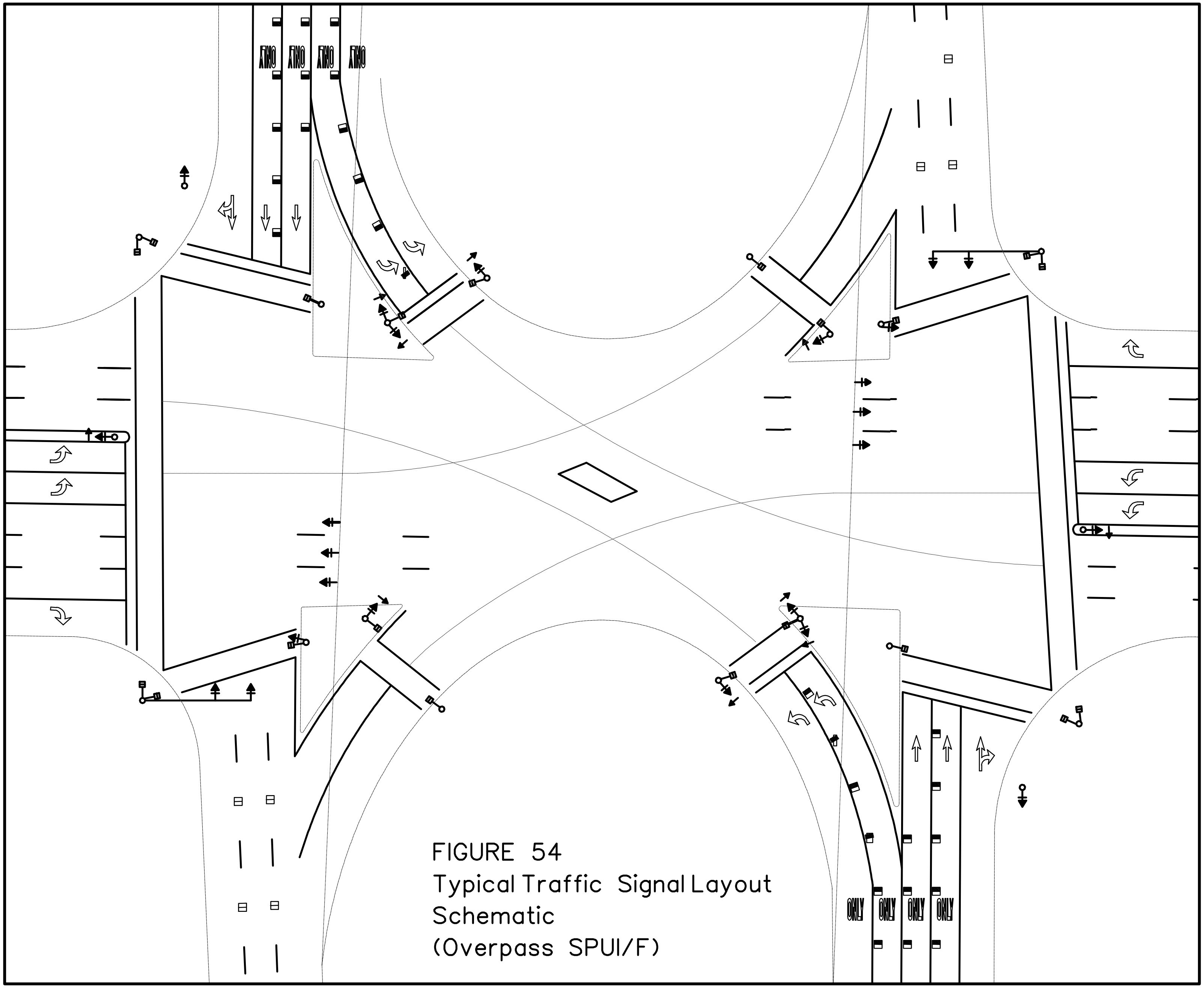


FIGURE 54
Typical Traffic Signal Layout
Schematic
(Overpass SPUI/F)

Pedestrians

Pedestrian accommodations are a critical factor in the safety of a SPUI/F due to the complexity of the interchange. The *MUTCD* (37) provides guidance for pedestrian traffic control, including pedestrian head and pushbutton usage, as well as signing and crosswalk pavement markings. The SPUI/F can usually be designed so that the pedestrian signal equipment can be placed on the same pole as traffic signal equipment. Additional pedestal poles may be required for some pedestrian crossing such as the frontage road crossing shown in Figure 53.

Crosswalks are provided at each of the five SPUI/F locations in the study. However, the presence of crosswalks across the crossroad is not necessarily a standard design for SPUIs. At SPUI/n (where frontage roads are not present), the lack of a through phase for vehicles crossing the crossroad requires a separate phase for pedestrians to make the same movement. This usually discourages the design of crosswalks across the crossroad at these locations. Messer *et* al. (5) reported that almost without exception, crosswalks were not provided for pedestrians to cross the crossroads at almost all of their 36 SPUI locations (7 of which were SPUI/Fs). Only one of the seven SPUI/F s included in the study was found the have crosswalks on the crossroad.

Although it has been shown that some SPUI/Fs operate without crosswalks on the crossroad, this design may decrease pedestrian safety by forcing pedestrians to cross the crossroad at unmarked locations. Crosswalk safety can become an issue for left turns from the off-ramp, where sight distances are often limited. However, Messer *et al. (5)* noted that pedestrians were observed to cross the on- and off-ramps parallel to the crossroad in a safe manner.

CHAPTER 7
RECOMMENDED OPERATIONAL GUIDELINES FOR THE SINGLE POINT URBAN INTERCHANGE WITH FRONTAGE ROADS

INTRODUCTION

This chapter describes guidelines for the operation of the single point urban interchange with frontage roads (SPUI/F). These guidelines are not comprehensive in terms of their treatment of the single point urban interchange. Rather, they are focused on the operation and control features of the interchange that are influenced by its frontage road approaches. As such, the guidelines are intended to be used in conjunction with existing guideline documents, including the *Highway Capacity Manual* (*13*), the Arizona Department of Transportation's *Traffic Engineering Policies, Guidelines, and Procedures* (*44*), and the *Manual on Uniform Traffic Control Devices* (*37*). Guidance regarding SPUI operation can also be found in *NCHRP Report 345 - Single Point Urban Interchange Design and Operations Analysis* (*5*).

This document addresses the traffic operation and control issues related to the SPUI/F and for which specific guidelines are not provided in the aforementioned documents. Topics addressed include: signal phase sequence, phase change interval, pedestrian phasing, traffic flow characteristics, performance measures, and delay estimation.

SIGNAL OPERATION GUIDELINES

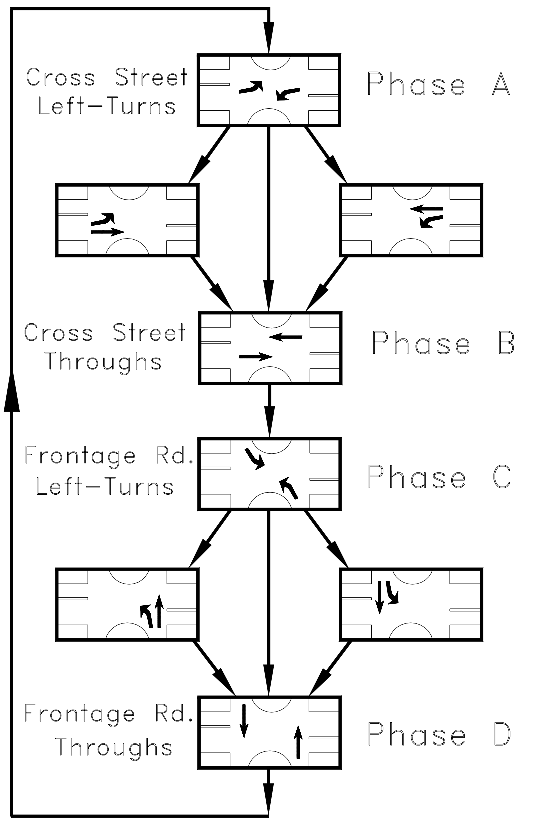
This section describes guidelines related to the signal operation of the SPUI/F. The first section to follow describes alternative signal phase sequences that can be used for the SPUI/F. The second section describes a procedure for determining the duration of the yellow and all-red clearance intervals. The last section describes a detection and control plan for serving pedestrians traversing the interchange.

Signal Phase Sequence

As noted by Messer *et al.* (*5*), three signal phase sequences are generally used at the SPUI/F. These sequences include leading left turns, lagging left turns, and direction separation (or split phasing). The advantages of each sequence, when applied to the SPUI/F, are described in the following paragraphs.

Before discussing the merits of the three phase sequences, it should be noted that all sequences employ protected left-turn phases for both the cross street and the frontage road approaches. Permitted left-turn operation is not appropriate for the SPUI/F because of its large conflict area and lengthy left-turn travel paths.

The most commonly used phase sequence has leading left turns on both the cross street and the frontage road approaches. The main advantage of this sequence is that it is consistent with driver expectancy at signalized intersections. Most drivers expect the left-turn phase to occur before the adjacent through movement and thus, tend to respond more quickly to the left-turn indication when it leads the through indication. The leading left-turn sequence is shown in Figure 55.



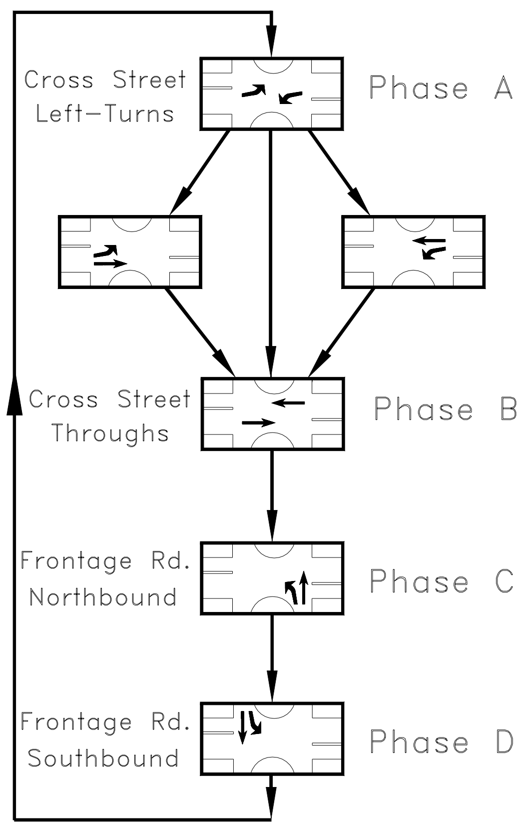


Figure 55. Leading left-turn phase sequence. *Figure 56. Direction separation phase sequence.*

The second type of phase sequence used is "direction separation" (or split phasing). This type of phase sequence is shown in Figure 56 for the frontage road. It could also be used for the cross street. In general, this phasing is associated with significantly more delay than leading (or lagging) left-turn phasing because it does not simultaneously serve movements of similar volume level (e.g., the two through movements).

Direction separation should be considered only when interchange geometry is constrained such that the opposing left-turn paths (e.g., the northbound and southbound left-turn paths) overlap within the interchange conflict area. Such constraints may occur when significant alignment skew is present or when extreme measures are taken in the design to minimize the size of the bridge structure. Direction separation may also be needed if experience indicates that left-turn drivers encroach on the opposing left-turn path. This behavior may occur when the left-turn operating speed exceeds the design speed of the left-turn path.

The third type of phase sequence used at some SPUI/F has lagging left turns on the cross street, frontage road, or both. This type of sequence is illustrated in Figure 57.

One potential advantage of the lagging left-turn phase sequence is that it can be configured to require shorter all-red clearance intervals for some SPUI/F phases (relative to those used for the leading left-turn sequence). Shorter clearance intervals are attractive because they can increase interchange capacity by minimizing phase lost time. The shortened clearance interval duration stems from consideration of the clearance path associated with each signal phase, relative to the travel path of the movement that enters on the *next* phase. Such considerations indicate that a relatively short clearance path length (and associated all-red interval) is dictated by the lagging left-turn sequence. Unfortunately, a fixed phase sequence is required to guarantee that the clearance-entry path pair always occur together each cycle. This requirement can be met with actuated control through the use of the "phase recall" setting; however, this operation defeats some of the benefits of actuated control.

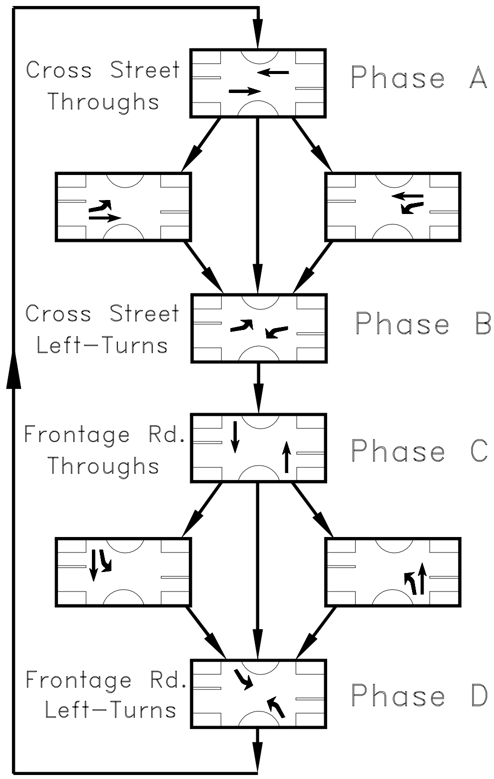


Figure 57. Lagging left-turn phase sequence.

Phase Change Interval

One procedure for calculating the duration of the phase change interval is that proposed by Technical Committee 4A-16 working under the direction of the Institute of Transportation Engineers (ITE) (*9*). The procedure recommended by this committee is based on the separate calculation of the two intervals that comprise the phase change interval (i.e., the yellow interval and the all-red clearance interval). The formula recommended by the ITE technical committee for determining the length of the yellow interval is:

(1)

$$Y(v) = T_{pr} + \frac{V_a}{2\ d_r + 2\ g\ G_r}$$

where:

$Y(v)$ = yellow interval evaluated at speed $V_a = v$, s;
d_r = deceleration rate, use 3.05 m/s^2 (10 fpss);
g = gravitational acceleration, use 9.81 m/s^2 (32.2 fpss);
G_r = approach grade, m/m;
T_{pr} = driver perception-reaction time, use 1.0 s; and
V_a = speed of vehicle approaching the intersection, m/s.

The all-red clearance interval is intended to provide time for those vehicles entering during the yellow to safely clear the intersection conflict area. If there is even minimal pedestrian activity, the conflict area is extended to include the "far-side" crosswalk. The all-red interval is calculated as:

(2)

$$AR(v) = \frac{L + d}{V_c}$$

where:
$AR(v)$ = all-red interval evaluated at speed $V_c = v$, s;
d = length of the average vehicle, use 6.10 m (20 ft);
L = the length of the clearance path, m; and
V_c = speed of clearing vehicle, m/s.

The value of L in Equation 2 is based on the amount of pedestrian activity. It is computed as:

- If there is no pedestrian activity then $L = L_w$;
- If there is some activity then L = the larger of $L_p - d$ or L_w; and
- If there is significant activity then $L = L_p$.

where:
L_w = length of the clearance path measured from the near-side stop line to the far edge of the farthest conflicting traffic lane along the actual vehicle path, m; and
L_p = length of the clearance path measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path, m.

With regard to the clearance path, a "conflicting traffic lane" is any lane controlled by a signal (including a signalized right-turn lane). Including a right-turn lane in the signalization is often attractive because it can reduce the delay to right-turning drivers. However, at a SPUI/F it can indirectly add delay to the other movements by increasing the all-red interval (via increased clearance path length) of one or more phases.

Phase Change Interval Calculation for Through Movements

Conventional Approach. Common practice for computing the yellow and all-red intervals for a through movement is through the direct use of Equations 1 and 2. For this computation, the approach speed V_a and the clearing speed V_c are set to the 85th percentile approach speed V_{85}. If the 85th percentile approach speed is not known, the ITE technical committee (*9*) recommends that it can be assumed to equal the posted speed limit.

The Conventional Approach was used to determine the yellow and all-red clearance interval requirements for the through movements at the SPUI/F. These interval durations are listed in Table 52 (they are the values *not* in parenthesis). The values in parenthesis are discussed in the next section.

Table 52. Yellow and all-red interval duration for through movements.

Metric									
85th % Speed, km/h	Yellow Interval (Y), s	Length of Clearance Path (L), m							
		20	30	40	50	75	90	105	120
		All-Red Clearance Interval [1] (AR), s							
48	3.2	1.9 (2.2)	2.7 (3.3)	3.4 (4.4)	4.2 (5.5)	6.0 (8.3)	7.2 (10.0)	8.3 (11.7)	9.4 (13.4)
56	3.6	1.7 (1.7)	2.3 (2.5)	2.9 (3.4)	3.6 (4.3)	5.2 (6.5)	6.1 (7.8)	7.1 (9.2)	8.1 (10.5)
64	3.9	1.5 (1.5)	2.0 (2.1)	2.6 (2.7)	3.1 (3.5)	4.5 (5.3)	5.4 (6.5)	6.2 (7.6)	7.1 (8.7)
72	4.3	1.3 (1.3)	1.8 (1.8)	2.3 (2.3)	2.8 (2.9)	4.0 (4.4)	4.8 (5.4)	5.5 (6.4)	6.3 (7.3)
80	4.7	1.2 (1.2)	1.6 (1.6)	2.1 (2.1)	2.5 (2.5)	3.6 (3.8)	4.3 (4.6)	5.0 (5.4)	5.6 (6.3)
88	5.0	1.1 (1.1)	1.5 (1.5)	1.9 (1.9)	2.3 (2.3)	3.3 (3.3)	3.9 (4.1)	4.5 (4.8)	5.1 (5.6)

U.S. Customary									
85th % Speed, mph	Yellow Interval (Y), s	Length of Clearance Path (L), ft							
		75	100	125	150	250	300	350	400
		All-Red Clearance Interval [1] (AR), s							
30	3.2	2.2 (2.5)	2.7 (3.4)	3.3 (4.2)	3.9 (5.1)	6.1 (8.5)	7.3 (10.2)	8.4 (11.9)	9.5 (13.6)
35	3.6	1.9 (1.9)	2.3 (2.5)	2.8 (3.2)	3.3 (3.9)	5.3 (6.6)	6.2 (8.0)	7.2 (9.3)	8.2 (10.7)
40	3.9	1.6 (1.7)	2.0 (2.1)	2.5 (2.6)	2.9 (3.2)	4.6 (5.4)	5.5 (6.6)	6.3 (7.7)	7.2 (8.8)
45	4.3	1.4 (1.4)	1.8 (1.8)	2.2 (2.2)	2.6 (2.6)	4.1 (4.5)	4.8 (5.5)	5.6 (6.5)	6.4 (7.4)
50	4.7	1.3 (1.3)	1.6 (1.6)	2.0 (2.0)	2.3 (2.3)	3.7 (3.8)	4.4 (4.7)	5.0 (5.5)	5.7 (6.4)
55	5.0	1.2 (1.2)	1.5 (1.5)	1.8 (1.8)	2.1 (2.1)	3.3 (3.4)	4.0 (4.1)	4.6 (4.9)	5.2 (5.7)

Notes:
1 - Values in parenthesis are based on consideration of both the 85th and 15th percentile speeds (conservative approach). Values not in parenthesis are based on consideration of only the 85th percentile speed (conventional approach).

Conservative Approach. The ITE technical committee (*9*) recommends that the clearance needs of both the "fast" (i.e., 85th percentile) *and* the "slow" (i.e., 15th percentile) driver should be considered. They note that, at intersections that are relatively wide or that have lower approach speeds, the slower driver can require considerably more time to clear the conflict area than the faster driver. Therefore, they recommend that the phase change interval should be calculated twice, once with the approach speed V_a set to the 15th percentile approach speed V_{15} and once with it set to the 85th percentile approach speed V_{85} (with V_c equal to V_a). The longer of these two change intervals is then used. The yellow interval is always based on V_{85}. The all-red interval is based on difference between the change interval and the yellow-warning interval. This procedure is summarized in the following calculation steps:

Step 1. $CI_{85} = Y(V_{85}) + AR(V_{85})$
Step 2. $CI_{15} = Y(V_{15}) + AR(V_{15})$
Step 3. CI = Larger of CI_{85} or CI_{15}
Step 4. $Y = Y(V_{85})$
Step 5. $AR = CI - Y(V_{85})$

where:

CI_{85} = phase change interval based on V_{85}, s;
CI_{15} = phase change interval based on V_{15}, s;
CI = phase change interval retained for use, s;
Y = yellow interval retained for use, s; and
AR = all-red clearance interval retained for use, s.

The ITE technical committee suggests that V_{15} can be assumed to be 16.1 km/h (10 mph) slower than the posted speed limit.

The purpose of the two phase change interval calculations in Steps 1 and 2 is to insure that the change interval duration is adequate for both the slow and fast driver. This sensitivity to slow speeds is particularly important when the clearance path is long, as is often found at the SPUI/F.

The all-red intervals that result from application of the Conservative Approach are listed in Table 52 (they are the values in parenthesis). These values tend to be larger than those obtained from the Conventional Approach and, if used, will result in slightly longer motorist delays. Nevertheless, they should be used whenever the right-angle crash frequency at a particular SPUI/F is abnormally high.

<u>Phase Change Interval Calculation for Left-Turn Movements</u>

The ITE technical committee′s procedure is not as precisely defined for left-turn movements as it is for through movements (*9*). In particular, the committee does not state whether both the 85th and the 15th percentile turn speeds need to be considered; however, it does state that the difference between these speeds is likely to be small for left-turn movements. Therefore, to simplify the calculation, the phase change interval is computed for one speed only.

The approach speed used to compute the yellow interval represents a compromise speed. It is recognized that approaching left-turn drivers could be in a free flow situation and approaching at a high speed or, they could be in a moving queue approaching the intersection at a slow speed. As a compromise solution, the ITE committee recommends that the average of the 85th percentile approach speed V_{85} and the average left-turn execution speed V_e be used to determine the length of the yellow interval. The following calculation steps describe the procedure for calculating the phase change interval components for left-turn movements:

Step 1. $V_m = (V_{85} + V_e) / 2$
Step 2. $Y = Y(V_m)$
Step 3. $AR = AR(V_e)$

where, V_m = compromise approach speed, m/s.

Research reported by Bonneson (*7*) indicates that V_e varies with the radius of the left-turn path. However, the data reported by Bonneson suggest that this variation is small for SPUI/F left-turn radii larger than 75 m (250 ft) and can be reasonably approximated by a constant left-turn speed of 48 km/h (30 mph).

This procedure can be used to determine the yellow and all-red clearance interval requirements for the left-turn movements at a SPUI/F. Alternatively, these interval durations are listed in Table 53 for typical speeds and clearance distances.

Table 53. Yellow and all-red interval duration for left-turn movements.

Metric								
Variable	**85th Percentile Approach Speed (V_{85}), km/h**							
	48	**56**	**64**	**72**	**80**	**88**		
Yellow Interval (*Y*), s	3.2	3.4	3.6	3.8	3.9	4.1		
Variable	**Length of Clearance Path (*L*), m**							
	20	**30**	**40**	**50**	**75**	**90**	**105**	**120**
All-Red Clearance (*AR*),s:	1.9	2.7	3.4	4.2	6.0	7.1	8.3	9.4
U.S. Customary								
Variable	**85th Percentile Approach Speed (V_{85}), mph**							
	30	**35**	**40**	**45**	**50**	**55**		
Yellow Interval (*Y*), s	3.2	3.4	3.6	3.8	3.9	4.1		
Variable	**Length of Clearance Path (*L*), ft**							
	75	**100**	**125**	**150**	**250**	**300**	**350**	**400**
All-Red Clearance (*AR*),s:	2.2	2.7	3.3	3.9	6.1	7.3	8.4	9.5

Note:
1 - Left-turn execution speed V_e assumed to equal 48 km/h (30 mph).

Pedestrian Phasing Considerations

Interchanges in urban areas must accommodate pedestrians with call buttons and pedestrian signal heads at all crossing locations. The SPUI/F presents some unique pedestrian signalization challenges because of its large size and multiple crossing points. These challenges are particularly significant for the pedestrian traveling along the cross street. This section describes the nature of this challenge and presents a feasible solution.

Recommended Pedestrian Signal Control Plan

The pedestrian traveling along the cross street through the SPUI/F is faced with four roadways to cross. In the typical pedestrian signal control plan, each crossing is controlled by a pedestrian signal and is configured to serve pedestrians concurrently with the cross-street through phase, as shown in Figure 58. At the start of this phase, a WALK indication is presented in each of the four signal heads. After a few seconds, the WALK indication is followed by a flashing DON'T WALK indication. The pedestrian crosses one of the four roadways each cycle. This type of plan requires four signal cycles to cross through the interchange and can yield a total crossing time of six to eight minutes.

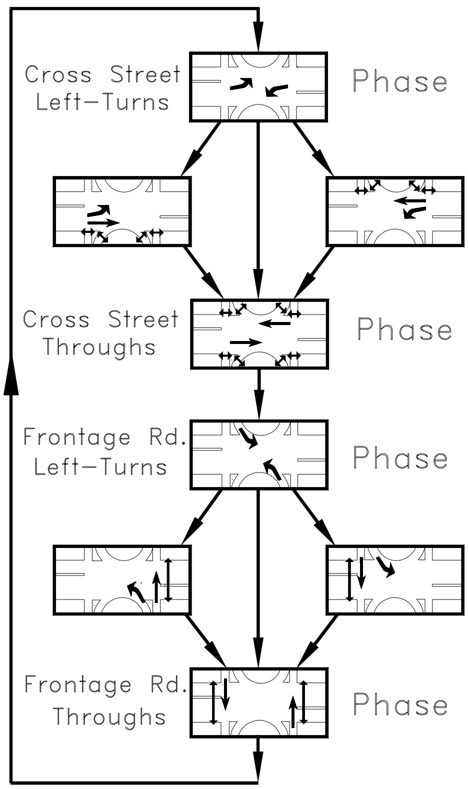


Figure 58. Pedestrian movements by phase.

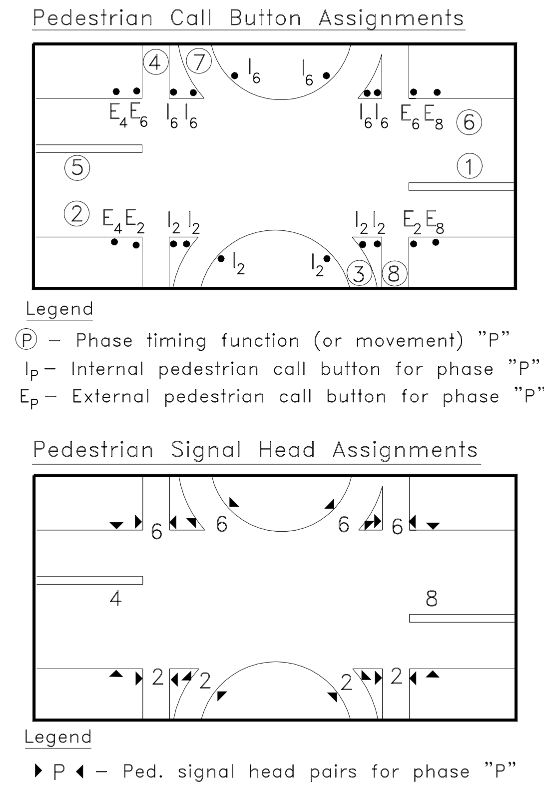


Figure 59. Pedestrian signal control plan.

A modified version of the aforementioned "typical" pedestrian signal control plan is described in this section. This modified plan can be used to reduce pedestrian crossing time along the cross street. The modified control plan recognizes that the first (and last) pair of roadways encountered can be crossed during one through signal phase. To encourage the pedestrian to complete this crossing, the WALK indication must be of sufficient length to allow the pedestrian to cross the first roadway and the intermediate island. The second roadway is then crossed during the flashing DON'T WALK interval.

The modified control plan is best illustrated by example. Consider a pedestrian crossing from west-to-east along the cross street. The pedestrian arrives at the curb and prepares to cross the southbound frontage road departure leg by pressing the pedestrian call button (denoted in Figure 59 as external button E_2). This button calls the adjacent cross-street through phase (i.e., phase 2).

Phase 2 turns green, the WALK is presented and the pedestrian crosses to the triangular island. The pedestrian crosses this island, finds that the WALK indication is still on, and continues by crossing the westbound-to-southbound left-turn path. During this second crossing, the flashing DON'T WALK indication comes on and allows time for the pedestrian to cross to the interior sidewalk. Internal button I_2 (which calls phase 2) is available if the solid DON'T WALK indication comes on before the pedestrian reaches the second roadway and gets "trapped" on the island.

After about a 35-s walk, the pedestrian reaches the northbound-to-westbound left-turn approach and presses internal button I_2. During the 35-s walk, the other phases have nearly completed their service and the cycle is about to serve the cross street through phase. The pedestrian will likely wait about 20 to 30 s at the curb for phase 2 to begin.

When phase 2 turns green (and the corresponding pedestrian signal heads show a WALK indication), the pedestrian crosses to the triangular island. The pedestrian crosses this island, finds that the WALK indication is still on, and continues by crossing the northbound frontage-road approach. During this second crossing, the flashing DON′T WALK indication comes on and allows time for the pedestrian to cross the frontage-road approach. Internal button I_2 is available if the solid DON′T WALK indication comes on before the pedestrian reaches the second roadway and gets "trapped" on the island.

Pedestrian Intervals

Specification of the minimum pedestrian phase duration for a given SPUI/F phase is based on consideration of pedestrian reaction time, walking time, and pedestrian clearance time. The need for a "walking time" component was described in the previous section. The WALK indication is displayed for the combined reaction and walking time components. The flashing DON′T WALK is displayed during the pedestrian clearance interval.

Reaction time can range from 4 to 7 s, depending on the volume of pedestrians present (*44*). The lower value is sufficient when the pedestrian volume served by the subject phase is less than 10 pedestrians per cycle. Larger values are used for moderate to heavy pedestrian volumes. Walking time is considered for the SPUI/F to allow sufficient time for the pedestrian to cross the first roadway (in a pair of roadways) and the intermediate island. The equation that combines these two time components and determines the duration of the WALK indication is:

$$T_{W,j} = T_{ppr} + \frac{D_{i,k}}{V_w} \qquad \textbf{(3)}$$

where:

$T_{W,j}$ = pedestrian WALK interval for phase j (j = 1, 2, ..., 8), s;
T_{ppr} = pedestrian perception-reaction time (typically 4.0 s), s;
$D_{i,k}$ = distance traveled during the WALK indication (measured between points i and k), see Table 54 and Figure 60, m; and
V_w = normal walking speed, use 1.2 m/s (4.0 fps), m/s.

Table 54 and Figure 60 identify the distances that need to be evaluated for each signal phase. Typical walking distances $D_{i,k}$ for the cross-street through phase at a SPUI/F range from 15 to 30 m (50 to 100 ft), which correspond to 17 to 29 s for the WALK indication.

Pedestrian clearance time is the minimum time needed for pedestrians to walk across the subject roadway. The flashing DON'T WALK indication is displayed during this interval. The equation for calculating the pedestrian clearance interval duration is:

(4)

$$T_{cl,j} = \frac{W_{i,k}}{V_w} - Y_j$$

where:

$T_{cl,j}$ = pedestrian clearance interval for phase j (j = 1, 2, ..., 8), s;
$W_{i,k}$ = width of roadway (measured between points i and k), see Table 54 and Figure 60, m; and
Y_j = yellow interval for phase j (j = 1, 2, ..., 8), see Table 54, s.

Table 54 and Figure 60 identify the widths that need to be evaluated for each signal phase. Typical widths $W_{i,k}$ for the cross-street through phase at a SPUI/F range from 8 to 12 m (25 to 40 ft), which correspond to 3 to 6 s for the flashing DON'T WALK indication.

Table 54. Variables for pedestrian interval calculation.

Phase	WALK Variable	Clearance Time Variables	
	Distance Walked[1, 3]	Width of Roadway[2, 3]	Yellow Interval
2	$D_{A,C}$, $D_{D,B}$, $D_{E,G}$, $D_{H,F}$	$W_{A,B}$, $W_{D,C}$, $W_{E,F}$, $W_{H,G}$	Y_2
4	*0.0*	$W_{I,A}$	Y_4
6	$D_{I,K}$, $D_{L,J}$, $D_{M,O}$, $D_{P,N}$	$W_{I,J}$, $W_{L,K}$, $W_{M,N}$, $W_{P,O}$	Y_6
8	*0.0*	$W_{H,P}$	Y_8

Note:
1 - For those phases with more than one distance variable, all distances should be used with Equation 3 to compute corresponding values of T_W. The largest of these values is then used in the signal controller.
2 - For those phases with more than one width variable, all widths should be used with Equation 4 to compute corresponding values of T_{cl}. The largest of these values is then used in the signal controller.
3 - Distance and width variable subscripts are defined in Figure 60.

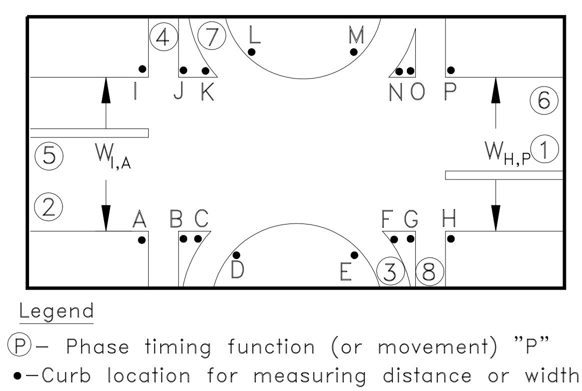


Figure 60. Roadway width variable definitions.

Table 54 lists the distances and widths that need to be evaluated with Equations 3 and 4, respectively. For both cross-street through phases (i.e., 2 and 6), the walking and clearance times need to be calculated for each of the four roadways crossed. For each phase, Equations 3 and 4 would be exercised four times, once for each roadway, with the *largest* WALK and the *largest* clearance time intervals used in the signal controller. Equations 3 and 4 can also be used (with the distances and widths in Table 54) to compute the WALK and the clearance time intervals for the frontage road phases (i.e., 4 and 8).

PERFORMANCE EVALUATION GUIDELINES

This section describes guidelines for evaluating the operation of the SPUI/F. Some of the information included in these guidelines pertains to the tight-urban diamond interchange (TUDI) because many SPUI/F performance evaluations are often conducted in the context of comparisons with the TUDI. The information in this section is offered to facilitate an equitable comparison among the two interchange forms.

The first section to follow identifies values for several traffic characteristics that have a significant influence on SPUI/F capacity. The second section defines two delay statistics that can be used to evaluation SPUI/F performance. These first two sections are intended to support the analysis of SPUI/F operation using a capacity analysis technique or simulation model. The last section provides delay characteristic curves and a procedure that can be used for "planning-level" evaluations of SPUI/F operation.

Traffic Characteristics

Saturation Flow Rate and Start-Up Lost Time

The capacity of a SPUI/F signal phase is highly dependent on the saturation flow rate and start-up lost time associated with that phase. Data published in the literature (*8, 11, 12, 45*) indicate that the SPUI/F traffic movements are very efficiently used by motorists such that the saturation flow rate is quite high, relative to a typical signalized intersection approach. A synthesis of these data suggest that the base saturation flow rates listed in Table 55 are representative of the SPUI/F traffic movements. The rates shown for the TUDI are included for comparative purposes.

Table 55. Base saturation flow rates for two interchange types.

Traffic Characteristic	Interchange Type[1]	Traffic Movement		
		Left-Turn	Through	Right-Turn
Base Saturation Flow Rate, veh/h/ln	TUDI	1,900	2,000	1,700
	SPUI/F	2,000	2,000	1,700
Start-Up Lost Time, Time, s	TUDI	2.4	2.8	1.7
	SPUI/F	2.8	2.8	1.7

Note:
1 - TUDI - tight urban diamond interchange; SPUI/F - single point urban interchange with frontage roads.

The saturation flow rates listed in Table 55 represent "base" conditions (i.e., 3.6 m lanes, no grade, no trucks, no parking, no buses, and balanced lane use). These rates should be adjusted to account for the effect of narrow lane widths, significant heavy vehicles, steep grade, frequent on-street parking activity, local bus stops, and uneven lane utilization. Adjustment factors for this purpose are defined in Chapter 16 of the *Highway Capacity Manual* (*HCM*) (*13*).

Recent research (*45*) also indicates that start-up lost time is slightly larger for those traffic movements with higher saturation flow rates. The lost times reported in Table 55 illustrate this trend and are based on the findings reported in Reference *45*.

End Lost Time

The lost time at the end of a phase is equal to the change interval duration (i.e., yellow plus all-red clearance intervals) less the initial portion of the yellow interval that is typically used by clearing drivers. Bonneson (*7*) and Poppe *et al.* (*8*) report that this "end-use" of the yellow interval varies between 2.5 and 3.0 s. Figure 61 can be used to estimate the phase end lost time for a given change interval. The trend line in this figure is based on an end-use of 2.5 s.

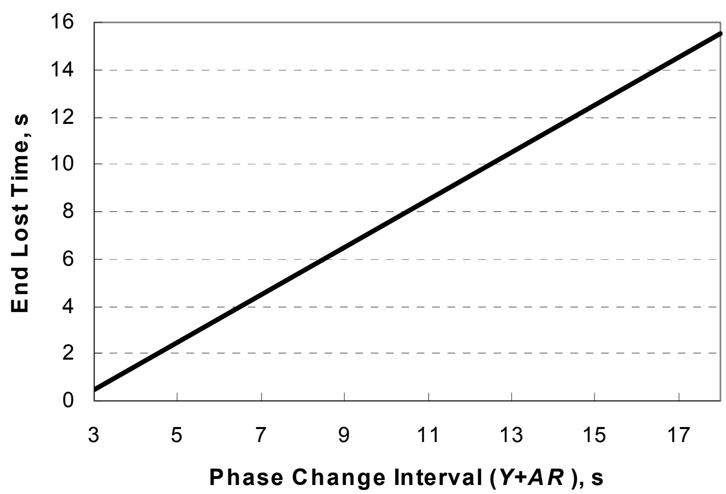


Figure 61. End lost time as a function of the phase change interval duration.

Performance Measures

The sections to follow define two delay statistics. The first delay statistic is derived to represent a performance measure that is unbiased by interchange form. As such, it can be used to compare the performance of alternative interchange types (e.g., SPUI/F and TUDI). The second delay statistic is intended for level-of-service assessment of an interchange. It is different from the first statistic in that its value is influenced by interchange configuration.

Delay Measures to Compare Interchange Alternatives

A useful delay statistic for comparing the performance of alternative interchange types is described in this section, it is termed "interchange delay." This statistic is derived to represent the average delay incurred by a motorist when traveling through the interchange (excluding motorists on the major street). It is computed as the total delay (in veh-hr) incurred by all vehicles using the interchange divided by the volume of vehicles on the *external* approaches to the interchange. Mathematically, it can be represented by the following equation:

$$d_I = \frac{\sum(d_i \; v_i) + \sum(d_j \; v_j)}{\sum v_i} \quad \textbf{(5)}$$

where:

d_I = interchange delay, s/veh;
d_i = average control delay for external movement i (i = 1, 2, ... , 14), s/veh;
d_j = average control delay for internal movement j (j = 15, 16, 17, 18), s/veh;
v_i = flow rate for external movement i (i = 1, 2, ... , 14), veh/h; and
v_j = flow rate for internal movement j (j = 15, 16, 17, 18), veh/h.

The traffic movement numbers identified in Equation 5 (by subscript) are defined in Figures 62 and 63 for the SPUI/F and TUDI, respectively.

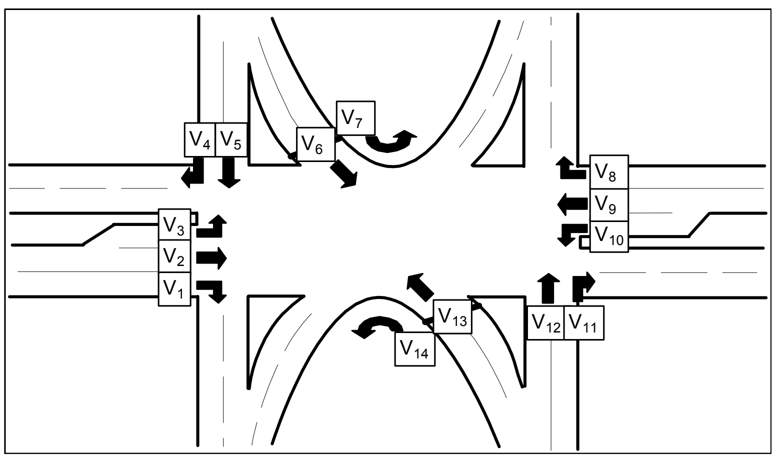


Figure 62. SPUI/F traffic movement numbers used for delay calculation.

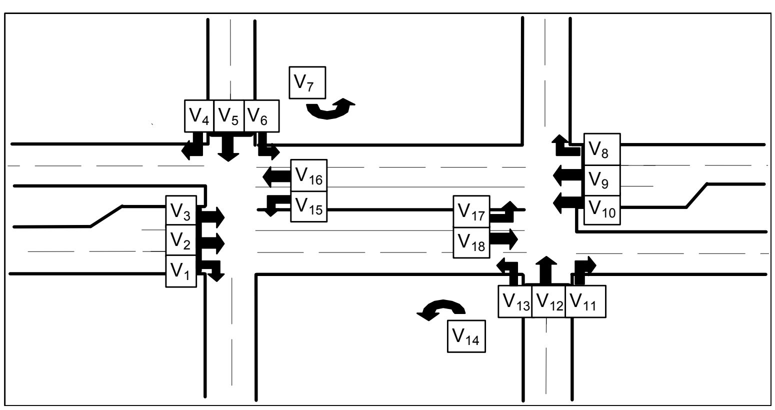


Figure 63. TUDI traffic movement numbers used for delay calculation.

An examination of the trends in Figure 63 indicates that "external" movements are those movements that allow entry to the interchange for the first time. In contrast, "internal" movements are those movements that encounter a second stop line within the interchange. SPUI/Fs have only external movements. TUDIs have internal left-turn and through movements on both of the cross-street approaches located between the ramp junctions.

The equation for computing interchange delay is defined such that its denominator is constant for all interchange forms and represents the total volume entering the interchange. As a result, this delay statistic can be used to compare alternative interchange forms without bias. On the other hand, interchange delay is not recognized in the *HCM* (*13*). A statistic suitable for assessing an interchange's level-of-service is the subject of the next section.

Delay Measures to Assess Level of Service

Chapter 26 of the *HCM* (*13*) indicates that control delay is the appropriate performance measure for assessing the level-of-service provided at intersections or interchanges. This delay can be used to estimate the level-of-service of an individual lane group or interchange approach. The relationship between level-of-service and control delay recommended in the *HCM* is reproduced in Table 56.

Table 56. Level-of-service criteria. (*13*)

Level of Service	Control Delay, s/veh
A	# 10
B	> 10 - 20
C	> 20 - 35
D	> 35 - 55
E	> 55 - 80
F	> 80

If, instead of lane-group delay or approach delay, a global delay is desired that can convey the overall level-of-service provided by an interchange, Chapter 26 of the *HCM* offers the following equation:

$$d_{aa} = \frac{\Sigma(d_i\ v_i) + \Sigma(d_j\ v_j)}{\Sigma v_i + \Sigma v_j} \quad \textbf{(6)}$$

where:

d_{aa} = average approach delay, s/veh;
d_i = average control delay for external movement i (i = 1, 2, ... , 14), s/veh;
d_j = average control delay for internal movement j (j = 15, 16, 17, 18), s/veh;
v_i = flow rate for external movement i (i = 1, 2, ... , 14), veh/h; and
v_j = flow rate for internal movement j (j = 15, 16, 17, 18), veh/h.

The traffic movement numbers identified in Equation 6 (by subscript) are defined in Figures 62 and 63 for the SPUI/F and TUDI, respectively. When applied to the SPUI/F, Equation 6 yields the same delay as Equation 5 because there are no internal movements at the SPUI/F.

The delay statistic computed by Equation 6 can be thought of as "average approach delay" because it represents an average delay for drivers on any one interchange approach (internal or external). More precisely, it is the total delay (in veh-hr) incurred by all vehicles using the interchange divided by the volume vehicles on the internal and external approaches.

Interchange Delay Estimation

This section describes a procedure that can be used to estimate the interchange delay and average cycle length for a given SPUI/F. It is based on the "critical movement analysis" (CMA) approach that forms the basis for the signalized intersection analysis procedure in Chapter 16 of the *HCM* (*13*). The CMA approach, and its applicability to the SPUI/F, is described in Chapter 4.

The procedure can be used to obtain a quick estimate of a SPUI/F's overall delay or cycle length for given volume levels, lane counts, and ramp-separation distances (i.e., the distance between the two frontage-road center lines, as measured along the cross street). The delay estimate can be useful for comparing a given SPUI/F to another interchange type or for evaluating the effect of alternative lane configurations or ramp-separation distances. The cycle length estimate can be used to assess the feasibility of coordinating the SPUI/F with adjacent signalized intersections on the cross street. A more precise delay estimate can be obtained through the direct use of the procedure in Chapter 16 of the *HCM*. A summary of *HCM* input variables that are sensitive to SPUI/F size and operation are described in a later section.

The procedure is based on three assumptions. First, it is assumed that *one* signal controller is used to control the interchange traffic movements. This assumption is consistent with current practice for SPUI operation (*5*).

Second, it is assumed that cycle time is allocated to the phases in proportion to the critical flow ratio (yielding an equal degree of saturation for all critical movements). This assumption is required for consistency with the CMA approach. It is a reasonable assumption when a full-actuated signal controller is used, provided that each phase has a reasonably short minimum green (say, 15 s or less) and large maximum green setting (say, 50 s or more). It is also a reasonable assumption when pretimed control is used and cycle time is explicitly allocated to the phases in proportion to the critical flow ratio.

Third, it is assumed that the left-turn movements are protected (no permissive operation) and are served independently of the adjacent through movement (i.e., phasing is consistent with that shown in Figures 55 and 57). This type of left-turn service yields equivalent movement delay, regardless of whether the left-turn movement leads or lags its conflicting through movement. Delay reductions resulting from short all-red intervals and fixed-sequence lagging left-turn phasing (as noted in the section titled Signal Phase Sequence) are not explicitly considered and are reasoned to be negligible. Also, direction separation phasing (i.e., Figure 56) is not addressed by this procedure.

Delay Estimation Procedure

The procedure consists of three steps that are completed in sequence. Inputs to the procedure are the movement volumes, the movement saturation flow rates, the number of traffic lanes on each approach, and the ramp-separation distance (i.e., the distance between the two frontage road center lines, as measured along the cross street). The steps are described as follows:

Step 1. Identify Movement Volumes and Lane Assignments. For this step, the design hourly volumes v are identified for the left-turn and through-plus-right-turn movements. These movements are numbered 1 through 8 using the convention identified in Figure 64.

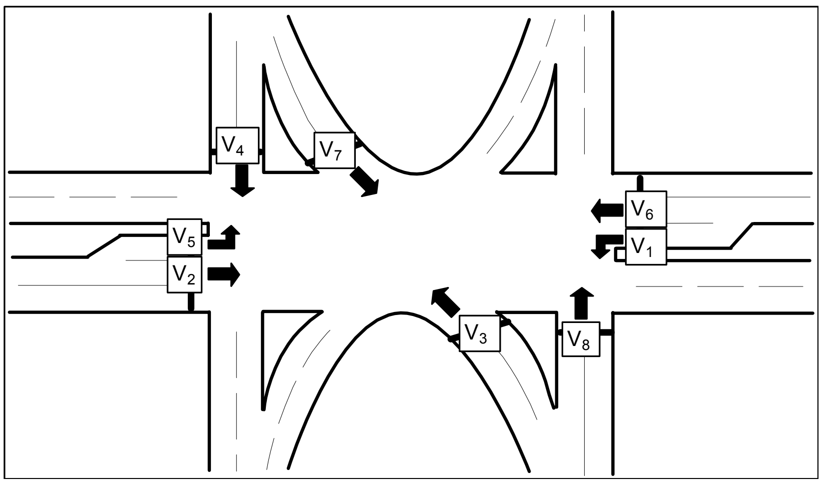


Figure 64. Movement numbers for critical flow ratio summation.

Also identified in this step is the saturation flow rate for each movement s and the number of lanes n allocated to each of the eight movements. The saturation flow rate can be estimated using the base rates listed in Table 4 and the saturation flow adjustment factors described in Chapter 16 of the *HCM* (*13*).

Step 2. Determine the Sum-of-Critical-Flow-Ratios. During this step the movement volume, saturation flow rates, and lane estimates from Step 1 should be used with Equations 7, 8 and 9 to estimate the sum-of-critical-flow ratios.

$$A = Larger\ of : \left[\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2} \ ; \ \frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6} \right] \tag{7}$$

$$B = Larger\ of : \left[\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4} \ ; \ \frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8} \right] \tag{8}$$

$$y_c = A + B \tag{9}$$

where:

y_c = sum of critical flow ratios;
v_i = volume of movement i (i = 1, 2, ... 8), veh/h;
s_i = saturation flow rate of movement i (i = 1, 2, ... 8), veh/h/ln;
n_i = number of lanes serving movement i (i = 1, 2, ... 8);
A = critical flow ratios for the cross-street movements; and
B = critical flow ratios for the frontage-road movements.

Step 3. Evaluate Delay and Cycle Length. For this step, the sum-of-critical-flow-ratios y_c from Step 2 is used with Figures 65 and 66 to determine the average interchange delay and cycle length, respectively. Table 56 can be then be checked to determine the corresponding level-of-service provided by the interchange.

The analysis that led to the development of Figures 65 and 66 also considered the effect of cross-street speed and the approach used to determine the phase change interval. The examination of speed indicated that speed did not have a significant effect on delay. Delay varied ± 1.0 percent for 85th percentile speeds ranging from 56 to 88 km/h (35 to 55 mph). For all analyses, the 85th percentile frontage road speed was set at 72 km/h (45 mph).

The examination of phase change interval focused on differences in delay due to the use of the Conventional or Conservative Approaches (as described in a previous section). The result of this examination indicated that the Conservative Approach consistently increased delay and cycle length by five percent relative to the Conventional Approach, throughout the full range of volumes and ramp-separation distances. The trends in Figures 65 and 66 are based on the use of the Conventional Approach, the values obtained from these figures should be multiplied by 1.05 if the Conservative Approach is used to set the phase change intervals for the through phases.

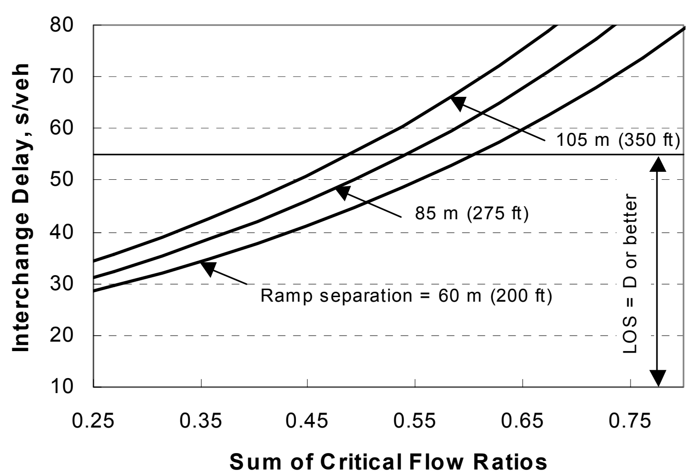


Figure 65. Interchange delay as a function of critical flow ratio.

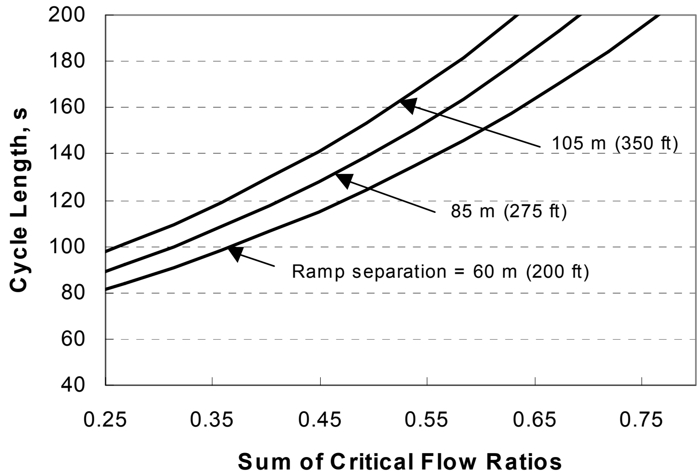


Figure 66. Average cycle length as a function of critical flow ratio.

<u>Example Application</u>

The procedure is illustrated by an example application in this section. Consider a full-actuated SPUI/F that has 70 m (230 ft) separating the two frontage-road center lines, as measured along the cross street (i.e., a 70 m ramp separation distance). The Conventional Approach was used to establish the phase change interval for the through phases. The volumes, lane counts, and saturation flow rates for this SPUI/F are listed in Table 57.

The saturation flow rates shown in Table 57 are the base rates listed in Table 55. Conditions on all approaches were such that no adjustment was needed to these rates (i.e., lane widths were 3.6 m (12 ft), negligible heavy vehicles, no grade, no on-street parking, no local buses, and balanced lane use). The saturation flow rate for the through-plus-right-turn movements is based on a weighted average flow rate and 10-percent right-turns (see Table 57 footnote).

Equation 7 requires finding the larger of two flow-ratio pairs (e.g., $v_1/(s_1\, n_1) + v_2/(s_2\, n_2)$ and $v_5/(s_5\, n_5) + v_6/(s_6\, n_6)$). The total for each pair is listed in column 7, with that for movements 5 and 6 representing the larger pair (i.e., 0.29) for the cross street approach. Equation 8 can be used to find that movements 3 and 4 represent the larger pair with a critical flow ratio of 0.28. From Equation 9, the sum-of-critical-flow-ratios is computed as 0.57.

Figures 65 and 66 can be used to estimate the delay and cycle length, respectively, that correspond to a sum-of-critical-flow-ratios of 0.57 and a ramp-separation distance of 70 m (230 ft). This estimation requires interpolation between the 60 m and 85 m trend lines. From Figure 65, a critical sum of 0.57 is consistent with an interchange delay of 54 s/veh. Because Equations 5 and 6 are equivalent when applied to the SPUI/F, the average approach delay is also 54 s/veh which, according to Table 56, corresponds to a level-of-service "D" for the interchange.

Table 57. Example critical volume computation.

Approach	Movement	Volume, veh/h	Lanes	Sat. Flow veh/h/ln	Flow Ratio	Flow Pairs	Critical Flow Ratio
Cross Street	1. Westbd. left-turn	475	2	2,000	0.12	0.29	0.29
	2. Eastbd. through + right	1,010	3	1,970	0.17		
	5. Eastbd. left-turn	475	2	2,000	0.12	<u>0.29</u>	
	6. Westbd. through + right	1,040	3	1,970	0.18		
Frontage Road	3. Northbd. left-turn	350	2	2,000	0.09	<u>0.28</u>	0.28
	4. Southbd. through + right	750	2	1,970	0.19		
	7. Southbd. left-turn	450	2	2,000	0.11	0.26	
	8. Northbd. through + right	575	2	1,970	0.15		
						Sum of Critical Flow Ratios:	0.57

Notes:
1 - Through+right saturation flow rate computed as a weighted average flow rate using the rates listed in Table 56 and a right-turn percentage of 10 percent [i.e., 1,970 = 2,000 * (1.0-0.1) + 1700 * (0.1)].

The trends in Figure 66 indicate that the interchange will operate with an average cycle length of about 145 s. As the adjacent signalized intersections on the cross street operate with a 90-s cycle length, the subject interchange is not a good candidate for coordination as a 90-s cycle length would require the interchange phases to be artificially shortened by using small maximum greens for each SPUI/F phase. Such a practice would likely result in an effective "pretimed" operation with a resulting increase in delay. Alternatively, had the adjacent intersections operated with a cycle length of 140 to 150 s, the subject interchange could easily be coordinated with these intersections without compromising the quality of interchange operation.

Interchange Delay Computation Using the Highway Capacity Manual

The procedure described in Chapter 16 of the *HCM* (*13*) can also be used to analyze SPUI/F operation. This procedure would be used when a delay estimate is desired that is more precise than that obtained from Figures 65 or 66. To obtain an accurate delay estimate with this procedure, several of its input values should be selected to reflect the SPUI/F's large size and unusual operation. The input variables include: signal phase sequence, phase change interval, minimum green duration, base saturation flow rate, start-up lost time, and extension of effective green time. Table 58 lists these variables and describes how the information in this document can be used to estimate suitable values for a SPUI/F analysis.

Table 58. Inputs to the *HCM* methodology.

Input Variable	Source	Comment
Signal Phase Sequence	Figures 55, 56, & 57	1. Use protected-only left-turn phasing. 2. Leading left-turn sequence (Fig. 48) is desirable. 3. Lagging left-turn sequence (Fig. 50) may be needed to facilitate signal coordination on the cross street. 4. Direction separation sequence (Fig. 49) should be avoided unless dictated by interchange geometry.
Phase Change Interval - Through Movements	Table 52	1. Calculate for each through phase. 2. Use Conservative Approach for through phases if right-angle crash history indicates a problem. Otherwise, use the Conventional Approach. 3. Include signalized right-turn movements when determining the length of the clearance path *L*.
Phase Change Interval - Left-Turn Movements	Table 53	1. Calculate for each left-turn phase. 2. Include signalized right-turn movements when determining the length of the clearance path *L*.
Minimum Green Duration	See section titled Pedestrian Phasing Considerations	1. Use 10 s for left-turn phases. 2. Use 10 s for through phases that have no ped. activity. 3. For through phases with pedestrian activity, use: $G_{min} = T_W + T_{cl}$ (see Equations 3 and 4).
Base Saturation Flow Rate	See Table 55	1. For through lanes (exclusive or shared) , use the value in Table 55 as the base sat. flow rate. 2. If the left-turn is from an exclusive lane, use the value in Table 55 as the base sat. flow rate and do not use *HCM* left-turn adjustment factor f_{LT}. 3. If the right-turn is from an exclusive lane, use the value in Table 55 as the base sat. flow rate and do not use *HCM* right-turn adjustment factor f_{RT}.
Start-Up Lost Time	See Table 55	1. Use the values Table 55.
Extension of Effective Green Time	See section titled End Lost Time	1. Use a value of 2.5 s.

CHAPTER 8
RECOMMENDED SELECTION GUIDELINES

INTRODUCTION

This chapter considers cost and operational components of the single-point urban interchange with frontage roads (SPUI/F) with that of the tight urban diamond interchange (TUDI) as presented in Chapters 7 and 4 respectively as they relate to the interchange selection process. Based on the crash analysis and conflict analysis in Chapter 3, there was no significant difference in the safety of the two interchange types. It should be recognized that there was not as much data as desired (i.e. 3 years of data) because most of the SPUI/F had not been completed for that period. As more crash data becomes available as the interchanges mature, the safety aspects should be reevaluated and strong consideration be given to either type that may have a demonstrated safety benefit.

This information is particularly relevant because the current ADOT process for selecting an interchange type is to generally select the least costly alternative from among those that provide an acceptable operational level. The purpose of this paper is to evaluate that process as it relates to the two previously mentioned interchange types and consider the resulting life cycle costs of each type.

RECOMMENDED PROCESS

The research project concluded that operational performance of a SPUI/F degrades rapidly as the distance between the frontage roads increases. This is documented in the examples later in this chapter. **The SPUI/F should only be considered when the spacing between frontage roads is less than approximately 60 m (200 ft).** Even then it should only be used when the cost of right of way to provide the extra width on the cross street required for the TUDI dual left turns is very expensive. In almost all normal cases the TUDI will perform at a level with sufficiently reduced delay when compared to the SPUI/F, that life cycle costs analysis would favor the TUDI.

In those situations where right of way limitations or cost require consideration of SPUI/F, the selection process can be done in two ways:

1. Comparable Performance Method. This method requires modifying the design of the SPUI/F to not only provide an “acceptable” level of service, but also one that is comparable to that of the competing TUDI design. This would be done primarily by reducing the spacing between the frontage roads, but could also involve the number of lanes. Once a comparable operation between the TUDI and the SPUI/F is achieved, the traditional cost comparison of construction and right of way costs can be used to make the selection.

2. Life Cycle Cost Method. This method estimates cost of each of the components of the interchange (e.g. right of way, construction, user costs, etc.). The cost of those components that are not believed to differ between the two types (e.g. crash costs, utilities, etc.) can be ignored in this analysis.

The following seven steps demonstrate the Life Cycle Cost Method. Each step of the process is described below with examples that present application of the process for hypothetical scenarios.

Step 1. Develop design year and design hour volumes
Step 2. Determine lane requirements for each interchange type
Step 3. Estimate right-of-way requirements and costs
Step 4. Estimate construction costs
Step 5. Predict road-user costs
Step 6. Make assumptions of service life of various components
Step 7. Compile all cost components into life cycle cost

The steps are described below and are accompanied by several examples following the process with the resulting recommended interchange type for each example. Based on these examples and the results of the safety study (Chapter 3) and operational study (Chapter 4) some general observations are made of conditions when each type would be favored.

PROCESS DESCRIPTION

Step 1. Develop design year and design hour volumes

The design year corresponds with the year in which the physical components of the interchange would be considered at the end of their service life. The difference in the opening year and the design year is used in the calculation of life-cycle costs explained in Steps 5 and 7. The design hour volume would be represented by an overall entering ADT projected for the interchange in its design year. As part of the life-cycle cost calculation, these projected entering ADT volumes are converted to hourly volumes and turning movement distributions.

The interchange ADT is converted to an assumed 2-hour AM peak and 2-hour PM peak for a total of four peak hours. Each of these peak hours is assumed to be 8% of the total daily volume. The interchange ADT is also used to estimate the following highest 8 hours of the day. This is based on the median volume of this eight-hour period being the 8^{th} highest hour of the day, using the hourly adjustment factor contained in ADOT Traffic Engineering Policies, Guidelines, and Procedures Manual *(34)*. These hourly volumes are then distributed among the various turning movements associated with an interchange based on the ten traffic distribution scenarios presented in Table 17 in Chapter 4.

Step 2. Determine lane requirements for each interchange type

For the purposes of this analysis, there are assumed to be two types of overall lane configuration associated with either the TUDI or SPUI/F. The two lane levels are called high and low as described in Chapter 4 and its associated Table 27. A high lane level will provide one additional left turn lane from the ramps and an additional through lane on the cross road when compared to the low lane level.

Typically, a high lane configuration would be chosen for interchanges expected to experience ADT volumes in the upper range of capacity.

Selection of the anticipated ramp separation distance is also part of this step. For the purposes of the detailed analysis, the options of ramp separation distances are limited to 61 m (200 ft), 92 m (300 ft), and 122 m (400 ft). Evaluations of specific ramp separation distances would be possible using the "Evaluation Model Analysis Procedure" described in Step 5. However, interpolation of a distance within the range of selected ramp separation distances would also be reasonable.

Step 3. Estimate right-of-way requirements and costs

This step in the selection process will vary depending on the actual conditions and specifications of the proposed interchange site. Therefore, for the purposes of the following examples, right of way costs will be used as a decision variable with respect to the results of the other steps of the process. Since right of way costs have apparently driven the selection of SPUI/F in the past, this is consistent with that practice.

Step 4. Estimate construction costs

Construction costs can vary significantly depending on the type of interchange and the specific location. However, for the most part, SPUI/F interchanges tend to cost more to construct. For the purposes of the following examples, it is assumed that construction of a SPUI/F interchange would cost approximately one million dollars more than the construction of a TUDI. In a Design Concept Report (DCR), the estimated cost of the two interchanges types would be used.

Step 5. Predict road-user costs

Typically, road user costs are based on the 1977 *Manual on User Benefit* Analysis *(32)* published by AASHTO. However, this data has become outdated and for this project we looked to other sources for data pertaining to road user costs with respect to delay. References pertaining to the subject of comparing cost effectiveness of SPUI interchange versus an at-grade intersection include NCHRP 345 *(5)* and an article in the Journal of Transportation Engineering *(33)*. Both sources detailed the user cost aspect of interchange evaluation between a SPUI option and an at-grade intersection. However, more up-to-date information was provided in NCHRP 345 pertaining to cost of delay, operating costs at idle, operating costs at stops, operating costs at operating speed, and accident costs.

Cost of delay data was given in NCHRP 345 as cost per vehicle-hour of delay. The figures of $12.69 per vehicle-hour for passenger cars and $23.02 per vehicle-hour for trucks are presented as 1990 dollars in the article. These values are based on research presented in NCHRP Project 7-12, which was an update to the information initially provided in the AASHTO *Manual on User Benefit Analysis*. In order to apply these delay cost figures to the calculated delay results from the project, the figures are converted to 2001 dollars. Consumer price indices (CPI) for Transportation and All Items were used in order to perform this calculation. The proportion of the Transportation CPI from 1982-2001 to the All Items CPI from 1982-2001 was calculated from CPI data provided by the Bureau of Labor Statistics. This proportion was then applied to the ratio of the All Items CPI for 2001 to the All Items CPI for 1990. The result is a factor of 1.18, which is applied to the user delay costs of $12.69 and $23.02 to obtain $14.97 and $27.16 in terms of 2001 dollars. A weighted average user delay cost of $16.19 is calculated based on truck traffic representing 10% of the traffic volume.

User cost of delay is selected as the point of comparison between SPUI/F and TUDI interchanges due to its substantial contribution to the overall road user cost. User costs of idling are assumed to be in proportion to the user costs associated with delay. However, this proportion is considered to be relatively insignificant with respect to the total user costs from delay. The weighted cost for idling used in NCHRP 345 was $0.943 per vehicle-hour. This figure, in 2001 dollars ($1.11), is only 6.8% of the weighted user cost of delay ($16.19 per vehicle-hour). Another user cost associated with interchange operations pertains to the number of vehicle stops incurred. Stop data from CORSIM model runs show 80 to 90% of entering traffic stops at the study intersections. The data also shows that there is a negligible difference in the percent stops between 4-phase TUDI operations and SPUI/F operations while comparisons of 3-phase TUDI versus SPUI/F operations show approximately 5 to 10% more stops associated with the 3-phase TUDI operation. However, this difference and the corresponding additional road user costs are still assumed to be equal due to its small overall effect when compared to the road user cost of delay. Operating speeds at SPUI/F and TUDI interchanges are assumed to be equal which negates the difference in user costs of operating speed. User costs associated with crashes are also regarded as equal based on the results presented in Chapter 3, which concluded that there is no significant difference in the crash rates associated with SPUI/Fs as compared to TUDIs. It should be noted that the calculated mean crash rate was higher for TUDI than for SPUI/F (1.79 vs. 1.57 crashes per million entering vehicles) however, statistical analysis shows this difference can result from chance. For this reason no cost advantage should be given.

The prediction of road user costs is a product of the weighted user cost of delay and the average delay calculated for each interchange type. In order to calculate the average delay for each interchange type, the process described in the "Evaluation Model Analysis Procedure" section of Chapter 4 was followed. A synopsis of the process is enumerated below:

1. Model is based on the methodology detailed in Chapter 16 of the HCM.
2. Critical movement analysis approach used to estimate the sum-of-critical-flow-rates and the delay for both the SPUI/F and the TUDI designs.
3. Evaluation model utilized in the form of a spreadsheet.
4. Evaluation tool examines five phase sequences, one for SPUI/F and four for TUDI.
5. Change interval duration, equilibrium cycle length, actuated phase green interval duration, and maximum/minimum green intervals are calculated by the evaluation model.
6. Ultimately, an average delay value in terms of delay per vehicle-hour is obtained by the evaluation model for each interchange type.

The model was evaluated for the different combinations of interchange ADT, lane configuration, and ramp separation distances. The results are compiled into tables reflecting these different scenarios, which are included in the appendix. Since the daily traffic representation at an interchange includes the effect of peak and non-peak hour traffic conditions, there are two tables for every scenario: one showing the average vehicle-hours of delay associated with the four hours of peak traffic conditions, and one showing the average vehicle-hours of delay associated with the eight hours of non-peak traffic.

The average vehicle-hours of delay presented in each table is multiplied by the weighted road user costs of delay and converted into an annual road user cost of delay (in 2001 dollars) for each interchange type in the design year. Therefore, the annual road user cost of delay for each interchange is obtained by summing the average delay costs pertaining to the scenario being examined (ADT and ramp separation) as shown in each table (4-hour results table and 8-hour results table).

Step 6. Make assumptions of service life of various components

There are numerous components that comprise the construction and operational aspects of an interchange. The Utah Department of Transportation (*31)* prepared a sample life cycle cost computation comparing a SPUI with a TUDI. The computation assumed a 7% amortization rate, and assigned a service life to each road element. It should be noted that this comparison was for a SPUI without frontage roads, however the evaluation method has relevance in this study.

Bridge	50 years
Roadway items	15 years
Earthwork	100 years
Retaining systems	50 years
Signals	10 years
Lighting	25 years
Signing	15 years
Engineering	25 years

The service life of each of these components can range from as little as 10 years to 100 years. For simplicity in this exercise and so as not to overstate the value of road user costs, the service life for the interchange of choice will be assumed to be 20 years as a whole. This assumption should provide conservative results in that some components of an interchange may last longer than 20 years and thus their overall cost would be less than what is calculated. Similarly, some components may not last as long as 20 years. However, these components such as signing, signal equipment, and roadway items tend to be a smaller percentage of the overall cost of the interchange. Such items as the bridge and earthwork will last longer and account for a larger percentage of the overall construction cost. Additionally, since we are comparing two interchange types these effects would cancel each other out since the same service life is assumed for both interchange types.

Step 7. Compile all cost components into life cycle cost

The first part of this calculation is to compare the predicted user costs for the two interchange types. The interchange with the lower user cost components is considered to have a user cost of delay benefit equal to the amount of the difference.

In order to assess the present worth of these benefits, the following equation is used *(5)*:

$$f = (e^{(r-i)*n} - 1) / (r - i) \qquad \textbf{(1)}$$

where f represents the factor that adjusts the opening year's benefits to estimate benefits for the service life of the interchange; n equals service life (20 years); $r = \ln(a) / n$, where a equals the ratio of the n-year's benefit to the opening year's benefit; and i is equal to the discount rate, which is assumed to be 0.04 (4% per year).

This value is then examined in conjunction with the other costs associated with each interchange type except for right-of-way costs, which vary from project to project. The right-of-way cost variable is then used to compare the financial conditions in which to select a certain interchange type.

The examples below illustrate the process described above. Each example attempts to vary the independent variables used in the selection process as a means of giving an overall impression of when a TUDI would be selected versus a SPUI/F.

Example 1

Step 1. Develop design year and design hour volumes

- Design year is chosen as 2020
- Total entering ADT for the interchange is estimated at 80,000 vehicles

Step 2. Determine lane requirements for each interchange type

- A high lane configuration, as described above, is chosen based on the 80,000 ADT
- Ramp separation distance is chosen as 92 m (300 ft). This will be determined by the specifics in the particular interchange.

Step 3. Estimate right-of-way requirements and costs

- Decision variable for this example

Step 4. Estimate construction costs

- SPUI/F construction costs estimated to be $1,000,000 more than TUDI. This would actually be estimated as is normally done in the Design Concept Report (DCR) process.

Step 5. Predict road-user costs

For this example, the following average annual road user costs of delay were obtained (using the tables provided in Appendix C) for the design year conditions (rounded to the nearest thousand) and are shown in Table 59.

Table 59. Average Annual Road User Costs of Delay for Design Year

Average Annual Road User Costs of Delay based on:	SPUI/F	3-ph TUDI (no overlap)	3-ph TUDI (with overlap)	4-ph TUDI (no overlap)	4-ph TUDI (with overlap)
Peak four hours	*$3,406,000*	*$3,458,000*	*$1,584,000*	*$3,601,000*	*$2,480,000*
5^{th} highest through 12^{th} highest hours*	*$3,130,000*	*$1,748,000*	*$1,801,000*	*$3,029,000*	*$1,990,000*
TOTAL	$6,536,000	$5,206,000	$3,385,000	$6,630,000	$4,470,000

* see explanation of Step 1 procedcure

The average annual road user costs for the opening year of the interchange must also be evaluated in order to determine the total benefit of selecting one interchange type over another. For this example, the opening year interchange ADT was assumed to 50,000. This opening year value would correspond with an average annual growth rate of about 2.4% in order to obtain the design year (20 years later) interchange ADT of 80,000. Therefore, the average annual road user costs of delay for the opening year of the interchange are shown in Table 60.

Table 60. Average Annual Road User Costs of Delay for Opening Year

Average Annual Road User Costs of Delay based on:	SPUI/F	3-ph TUDI (no overlap)	3-ph TUDI (with overlap)	4-ph TUDI (no overlap)	4-ph TUDI (with overlap)
Peak four hours	*$1,227,000*	*$ 715,000*	*$ 729,000*	*$1,123,000*	*$ 755,000*
5th highest through 12th highest hours*	*$1,410,000*	*$ 857,000*	*$ 843,000*	*$1,216,000*	*$ 907,000*
TOTAL	$2,637,000	$1,572,000	$1,572,000	$2,339,000	$1,662,000

* see explanation of Step 1 procedure

The road user cost benefit is in favor of the 3-phase TUDI (with overlap) for both the design year ($3,151,000) and the opening year ($1,065,000). These figures will be used in the life-cycle cost calculation in Step 7.

Step 6. Make assumptions of service life of various components

- Assumptions described above

Step 7. Compile all cost components into life cycle cost.

Applying the equation described above to the user cost benefits of a TUDI gives a present worth of road user benefits equal to $24,641,970 (f = 23.138). This value is examined in conjunction with the construction and right-of-way costs as follows:

Road user cost of delay benefits in favor of TUDI:	$24,641,970
Additional construction cost associated with SPUI/F:	+ $ 1,000,000 (estimate)
Right-of-way savings with SPUI/F:	- $ 2,000,000 (estimate)
Total present worth cost benefit of TUDI design:	$23,641,970

As shown above, the right-of-way savings from a SPUI/F design would have to exceed $25 million in order for the SPUI/F design to become the preferred option.

Example 2 (steps have been condensed into the pertinent information)

Design year interchange ADT: 70,000
Opening year interchange ADT: 40,000
Lane configuration/ramp separation: high lane level, 61 m (200 ft) ramp separation

Table 61. Total Annual User Cost of Delay Estimates (2001 dollars) for Example 2

	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Design Year	$4,325,000	$4,735,000	$2,509,000	$4,843,000	$3,420,000
Opening Year	$1,651,000	$1,081,000	$1,110,000	$1,602,000	$1,173,000

From Road User Cost Tables (Appendix C)

Present Worth of User Cost of Delay Benefits from Selecting TUDI: $13,713,060 (f = 24.058)

In Example 2, the savings in right-of-way costs for a SPUI/F design would need to meet or exceed the present worth of user cost of delay benefits figure realized by selecting a TUDI design in order to make the SPUI/F design a viable option. However, this example points out how circumstantial this case would be in that the initial interchange ADT must be relatively low with respect to the ADT levels for which an interchange would be considered at a location. Also, the construction costs of a SPUI/F would need to be approximately the same as a TUDI, which is generally not the case. Otherwise, the difference in construction cost would also need to be made up in the right-of-way cost savings. Ultimately, for this scenario to be in effect, the location would have to be in a densely developed area where the acquisition of right-of-way is difficult and expensive enough to outweigh the user cost of delay benefits associated with a TUDI design.

Example 3 (steps have been condensed into the pertinent information)

Design year interchange ADT: 90,000
Opening year interchange ADT: 70,000
Lane configuration/ramp separation: high lane level, 61 m (200 ft) ramp separation

Table 62. Total Annual User Cost of Delay Estimates (2001 dollars) for Example 3

	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Design Year	$8,575,000	$21,117,000	$11,116,000	$11,253,000	$6,572,000
Opening Year	$4,325,000	$4,735,000	$2,509,000	$4,843,000	$3,420,000

From Road User Cost Tables (Appendix C)

Present Worth of User Cost of Delay Benefits from Selecting TUDI: $26,097,736 (f = 14.371)

Example 3 shows a typical case where implementation of an interchange would be considered at an appropriate ADT level. In this scenario, the user cost of delay benefits from selecting a TUDI design are similar to the results from Example 1.

Example 4 (steps have been condensed into the pertinent information)

Design year interchange ADT: 70,000
Opening year interchange ADT: 40,000
Lane configuration/ramp separation: high lane level, 92 m (300 ft) ramp separation

Table 63. Total Annual User Cost of Delay Estimates (2001 dollars) for Example 4

	SPUI/F	3-ph TUDI no overlap	3-ph TUDI with overlap	4-ph TUDI no overlap	4-ph TUDI with overlap
Design Year	$4,930,000	$2,985,000	$2,702,000	$4,843,000	$3,192,000
Opening Year	$1,873,000	$1,135,000	$1,117,000	$1,603,000	$1,231,000

From Road User Cost Tables (Appendix C)

Present Worth of User Cost of Delay Benefits from Selecting TUDI: $17,456,796 (f = 23.091)

Example 4 considers the same parameters used in Example 2 except the ramp separation distance has been increased to 92 m (300 ft). The additional ramp separation distance has a negative effect on the operation of the SPUI/F design which is reflected in the increased present worth of user cost of delay benefits related to the TUDI design.

CHAPTER 9
CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

This chapter summarizes the major findings resulting from this research project and offers recommendations for implementation of the findings.

SAFETY

Crash Analysis

A crash analysis was prepared for the portion of the most recent three years of operation at each of the 5 TUDI and 5 SPUI/F where the interchange geometry and operations had not changed. There was no significant difference in the crash rates at the SPUI/F as compared to those at the TUDI. There was a significant difference in the location of those crashes. The greater proportion of rear-end crashes occurred on the frontage roads at SPUI/F. The greater proportion of rear-end crashes occurred on the arterial roadway at TUDI.

Conflict Analysis

At the 0.05 significance level, there was no significant difference between the SPUI/F and TUDI conflict rates, but at the 0.10 significance level, SPUI/Fs had a greater conflict rate than TUDIs.

Some correlation was found between the crash rates and conflict rates of each interchange.

Other Observations

The presence of high-volume driveways within the TUDI and SPUI/F study area tended to increase the number of conflicts that occurred at an interchange.

A common scenario for conflicts on SPUI/Fs occurred between right turns from the cross road and opposing left turns from the cross road.

The nature of the pedestrian crossings of the frontage road at the SPUI/F appears to result in decreased pedestrian compliance with the pedestrian signals.

OPERATIONS

The findings from the interchange evaluation indicate that a sound, rational approach to interchange form selection and operational evaluation is feasible using the characteristic relationship between interchange delay and the sum-of-critical-flow-ratios. The use of these curves can provide a solution to the challenging question of, "Which interchange form is most efficient?" Previous research projects directed at answering this question have produced guideline statements that can be characterized as vague, subjectively based, or difficult to apply.

The sum-of-critical-flow-ratios is a unique parameter that can combine an infinite number of interchange volume level, volume pattern, and geometry combinations into a single value. Furthermore, the analysis presented in the previous section indicates that this parameter has a unique delay relationship based on interchange type and phase sequence. These attributes can be exploited to develop a family of characteristic curves for a range of ramp separation distances that collectively can be used to identify the most efficient interchange alternative.

The characteristic curves could be used for planning-level and operations-level evaluations. At the planning level, it would be sufficient to identify and sum the critical movement lane volumes and then divide this total by a representative saturation flow rate to obtain the sum-of-critical-flow-ratios. At the operations level, the critical movement flow ratios would be computed and summed. This latter application would incorporate more detail regarding the saturation flow rate of the individual movements.

The only limitation of this approach is that it assumes that a single, actuated controller is used to control the interchange phase sequence. The use of two controllers (i.e., one for each frontage road junction) or the use of pretimed phases would violate key assumptions related to phase time allocation. Such deviations may blur the relationship between the sum-of-critical-flow-ratios and delay.

COST

A cost evaluation of interchanges can be made in various manners. If one evaluates only right-of-way and construction costs as was done in the Design Concept Reports an alternative may be selected which will provide the least initial cost, however which may result in a higher life cycle cost. This is especially true if one considers the cost to the motoring public. This evaluation has compared the planning level cost estimates of two interchange types with the actual cost when they were finally built. Although the sample sizes do not permit definitive conclusions, the cost estimates for the SPUI/F appear to have been underestimated.

When one considers the road user cost, the life cycle cost of the TUDI for all three interchanges is considerably less than that of the SPUI/F. The primary reason for this is the additional delay at the SPUI/F for interchanges with these ramp separation distances as discussed in Chapter 4.

SELECTION GUIDELINES

The current ADOT process for selecting an interchange type is to generally select the least costly alternative from among those that provide an acceptable operational level. One aspect of this research project was to evaluate that process as it relates to TUDI and SPUI/F and consider the resulting life cycle costs of each type. The operations analysis concluded that operational performance of a SPUI/F degrades rapidly as the distance between the frontage roads increases. **The SPUI/F should only be considered when the spacing between frontage roads is less than approximately 60 m (200 ft).** Even then it should only be used when the cost of right of way to provide the extra width on the cross street required for the TUDI dual left turns is very expensive. In almost all normal cases the TUDI will perform at a level with sufficiently reduced delay when compared to the SPUI/F, that life cycle costs analysis would favor the TUDI.

In those situations where right of way limitations or cost require consideration of SPUI/F, the selection process can be done in two ways:

1. Comparable Performance Method. This method requires modifying the design of the SPUI/F to not only provide an "acceptable" level of service, but also one that is comparable to that of the competing TUDI design. This would be done primarily by

reducing the spacing between the frontage roads, but could also involve the number of lanes. Once a comparable operation between the TUDI and the SPUI/F is achieved, the traditional cost comparison of construction and right of way costs can be used to make the selection.

2. Life Cycle Cost Method. This method estimates cost of each of the components of the interchange (e.g. right of way, construction, user costs, etc.). The cost of those components that are not believed to differ between the two types (e.g. crash costs, utilities, etc.) can be ignored in this analysis.

Although there are issues relating to the application of life cycle costs on transportation projects, this research documents the importance of considering future costs, including road user cost, in making decisions on interchange type.

APPENDIX A

Sample Questionnaires for Survey of Practitioners

April 11, 2002

Dear Transportation Practitioner:

The Texas Transportation Institute is conducting Research Project SPR-501 " Evaluation of Operational Efficiencies, Cost, and Accident Experience of the Four-Phase Single Point Urban Interchange" for the Arizona Department of Transportation (ADOT). One objective of this research is to develop a Guide for interchange selection, design, and operations/control.

We are contacting your state because we are aware of one or more single-point urban interchanges *with frontage roads* (SPUI/F) on your state highway system. We would like to learn about your experience with these interchanges. We are also interested in learning about the guidelines used by your engineers when selecting an interchange configuration for a specific location and when designing a SPUI/F.

We are requesting that an engineer familiar with the aforementioned SPUI/F's complete this questionnaire and return it to us by December 1, 2000. If you do not have the time or the desired experience, we would appreciate your passing this questionnaire on to someone in your agency who has the necessary experience.

We realize that you may receive many inquiries like this and that they take up a lot of your time. However, the insight and experience you provide to us through this questionnaire is very important as it will be used to help ADOT with their interchange selection and design process. If you are interested, we would be happy to provide you with a copy of the Guide developed from this research.
We sincerely appreciate your taking the time to share this information with us. Thank you in advance for your assistance.

Sincerely,

James A. Bonneson, P.E., Project Engineer

cc: Jim C. Lee, P.E., Principal Investigator, Lee Engineering
Frank R. McCullagh, PE, Senior Res. Engr., Arizona Transportation Research Center, ADOT

ARIZONA DEPARTMENT OF TRANSPORTATION
Arizona Transportation Research Center
PROJECT SPR 501: Evaluation of Four Phase Single Point Urban Interchanges

Traffic Operations Questionnaire

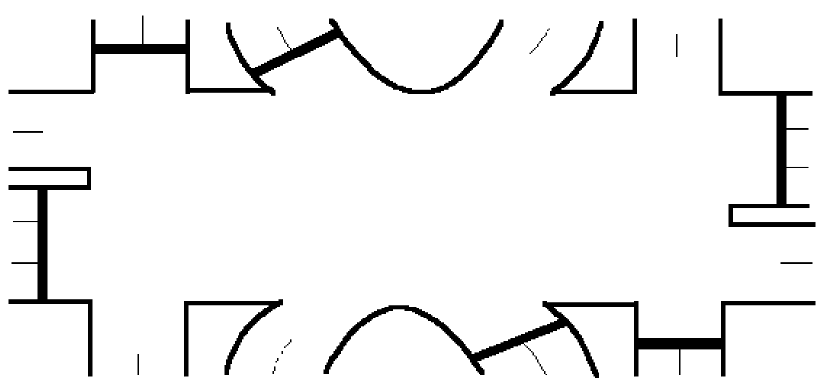
Please answer Questions 1 and 2 as they apply to the SPUI/Fs listed below:
1. ____________________________
2. ____________________________

1. Describe the traffic control features of your SPUI/F(s).			2. Rate your SPUI/F(s) on a scale of 1 to 5 (1 = poor, 5 = excellent) according to how well they meet the following expectations as compared with a similar compressed or tight diamond interchange with frontage roads.				
	Actuated or Pretimed?	**NEMA or 170/2070?**	**Traffic Capacity**	**Arterial Coordination**	**Traffic Safety**	**R.O.W. Requirements**	**Construction Cost**
SPUI/F 1							
SPUI/F 2							

Please answer Questions 3 and 4 as they relate to your agency's position on the SPUI with frontage roads (SPUI/F).
NOTE: Please restrict your answers to only those single point urban interchanges that have "continuous" frontage roads (i.e., frontage roads that include a through movement at the cross-street intersection)

3. Consider the technique used to set the all-red clearance (AR) interval for the cross-street through phase at a SPUI/F.
a. Is the AR interval... ___(i) based on interchange travel time, or ___(ii) other. If "other," please explain:
____
b. If AR is based on travel time, is the travel distance measured from the near-side stop line to the far edge of the...
___(i) farthest conflicting traffic lane along the vehicle path (based on all conflicting movements),
___(ii) farthest conflicting traffic lane along the vehicle path (based only on conflicting movements in the subsequent phase),
___(iii) farthest conflicting pedestrian crosswalk along the vehicle path, or
___(iv) other:____
c. Are any special control features used to minimize the adverse impact of the AR interval on traffic operation (e.g., variable change interval)? ___ No ___ Yes. If "Yes," please describe: ______________________________
d. Are the AR intervals for left-turn phases set using the same techniques as for the cross-street through phase?
___No ___ Yes. If "No," please explain:____
____

4. Please mark the following traffic control elements on the drawing at right (see note below):
a. Typical signal head locations (use a triangle% for each head).
b. Typical pedestrian cross walks (use a dashed line to represent walk centerline).
c. Typical pedestrian push button locations (use a dot # for each button).



NOTE: selected pages from a "typical" SPUI/F's traffic control plan can be substituted for this drawing.

(see back)

5. Does your agency have formal guidelines for **SPUI/F operation**? __ Yes __ No. If No, proceed to Question 6.
   i If Yes, can we obtain a copy of this documentation?
   ___ No (please explain)
   ____
   ___ Yes, photocopied pages are enclosed.
   ___Yes, a copy of the document can be purchased from:
   ____
   (name) (telephone no.)

6. Does your agency have any **safety study reports** for its existing SPUI/Fs? __ Yes __ No. If No, proceed to Question 7.
   i If Yes, can we obtain a copy of this documentation?
   ___ No (please explain)
   ____
   ___ Yes, photocopied pages are enclosed.
   ___Yes, a copy of the document can be purchased from:
   ____
   (name) (telephone no.)

Thank you for filling out this questionnaire. As we may have a couple of follow-up questions regarding your responses, could you please provide us with your name and telephone number in the space provided below (or just staple your business card)?

7. Name:____
   Title:____________________________________
8. Agency:____ Telephone
   Number:__________________________
9. Please add any additional comments you may have on the operational and safety characteristics of the SPUI/F.

____
____
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To be of greatest help to our project, we will need this questionnaire completed and returned by **December 1, 2000**. We have provided a self-addressed, postage-paid envelope for this purpose. Thanks again.

James Bonneson, P.E., Texas Transportation Institute, 3135 TAMU, Texas A&M University, College Station, TX 77843-3135
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ARIZONA DEPARTMENT OF TRANSPORTATION
Arizona Transportation Research Center
PROJECT SPR 501: Evaluation of Four Phase Single Point Urban Interchanges

Planning and Design Questionnaire

Please answer the questions below as they relate to your agency's position on the SPUI with frontage roads (SPUI/F) (like the one located at ___________________________________).

NOTE: Please restrict you answers to only those single point urban interchanges that have "continuous" frontage roads (i.e., frontage roads that include a through movement at the cross-street intersection).

1. How many SPUI/F's are in operation in your state?_________
2. Are you aware of any other states that have SPUI/F's?___No ___ Yes
3. If you answered Yes to Question 2, please list states and the locations (city or street) of these SPUI/F's.
   a)________________________________ b)________________________________
   c)________________________________ d)________________________________
4. How many SPUI/F's are planned for construction by your agency in the next 5 to 10 years?______

5. Indicate your reasons for selecting or considering a SPUI/F over other interchange forms. Check all that apply.

Reason for Selecting a Single-Point Urban Interchange with Frontage Roads	Yes, a primary reason for selection.
a. Restricted right-of-way cost	
b. Efficient signal phasing to obtain minimum delay	
c. SPUI/F expected to increase traffic-carrying capacity	
d. Signalization at only one major intersection simplifies coordination on the arterial	
e. SPUI/F design lessens construction cost	
f. To accommodate extremely high left-turn volumes	
g. Existence of excessive large-truck operations involving left-turn movements	
h. SPUI/F expected to relieve congestion difficulties	
i. SPUI/F is the safer alternative design	
j. Easier access to surrounding land uses	
k. Other:	

6. Does your agency have guidelines for (a) interchange selection or (b) SPUI/F design?_ Yes _ No. If No, proceed to Question 7.
   $ If Yes, can we obtain a copy of this documentation?
   ___ No (please explain) ________________________________
   ___ Yes, photocopied pages are enclosed.
   ___Yes, a copy of the document can be purchased from: ________________________________
   (name) (telephone no.)

Thank you for filling out this questionnaire. As we may have a couple of follow-up questions regarding your responses, could you please provide us with your name and telephone number in the space provided below (or just staple your business card)?

7. Name:________________________________ Title:________________________________
8. Agency:________________________________ Telephone Number:________________________
9. Please add any additional comments you may have on the operational and safety characteristics of the SPUI/F on the back side.

To be of greatest help to our project, we will need this questionnaire completed and returned by **December 1, 2000**. We have provided a self-addressed, postage-paid envelope for this purpose. Thanks again.

James Bonneson, P.E., Texas Transportation Institute, 3135 TAMU, Texas A&M University, College Station, TX 77843-3135
(979) 845-9906 Fax: (979) 845-6254 e-mail: j-bonneson@tamu.edu

APPENDIX B

Horizontal Layout of Selected Single-Point Urban Interchanges with Frontage Roads

Table B-1. Summary of physical attributes of selected single point urban interchanges with frontage roads.

Attribute		Interchange Location				
		US 231 & US 72	**US 19 & SR 686**	**Peachtree Ind. & Winters Chapel**	**Peachtree Ind. & J. Carter Blvd.**	**US 54 & West St.**
Location	City	Huntsville	Largo	Atlanta	Atlanta	Wichita
	State	Alabama	Florida	Georgia	Georgia	Kansas
U-turn Lanes?		Yes	Yes	Yes	Yes (one side)	No
Cross Section of Cross Street[1]	Left-turn lanes	1	2	1	1/2	2
	Through lanes	2	3	2	2	2
	Right-turn lanes	0	1	0/1	0	0/1
	Total width, m (ft)	15 (50)	30 (100)	24 (80)	24 (80)	27 (90)
Cross Section of Ramps[1]	Left-turn lanes	1	2	2	2	1/2
	Through lanes	2	2	2	1	0/2
	Right-turn lanes	1	0	0	0	1
	Total width, m (ft)[2]	58 (190)	82 (270)	73 (240)	76 (250)	67 (220)
Dist. between cross st. stop lines		64 (210)	82 (270)	81 (265)	85 (280)	67 (220)
Ave. Left-Turn Radii	Cross road, m (ft)	30 (98)	70 (230)	44 (145)	18 (60)	55 (180)
	Ramp, m (ft)	25 (82)	99 (325)	30 (100)	18 (60)	56 (183)
Center, raised-curb island?		No	Yes	No	No	Yes

Notes:

1 - Lane numbers shown reflect the total count of lanes in one direction of travel. Unless shown by a fraction (e.g., 0/1), the lane count is the same for the opposing direction of travel.

2 - Total width for the "Ramp" category is measured across both ramps (outside edge-of-curb to outside edge-of-curb) and includes the width of the major road cross section. It reflects the total width of the SPUI/F as measured along a line perpendicular to the major road centerline.

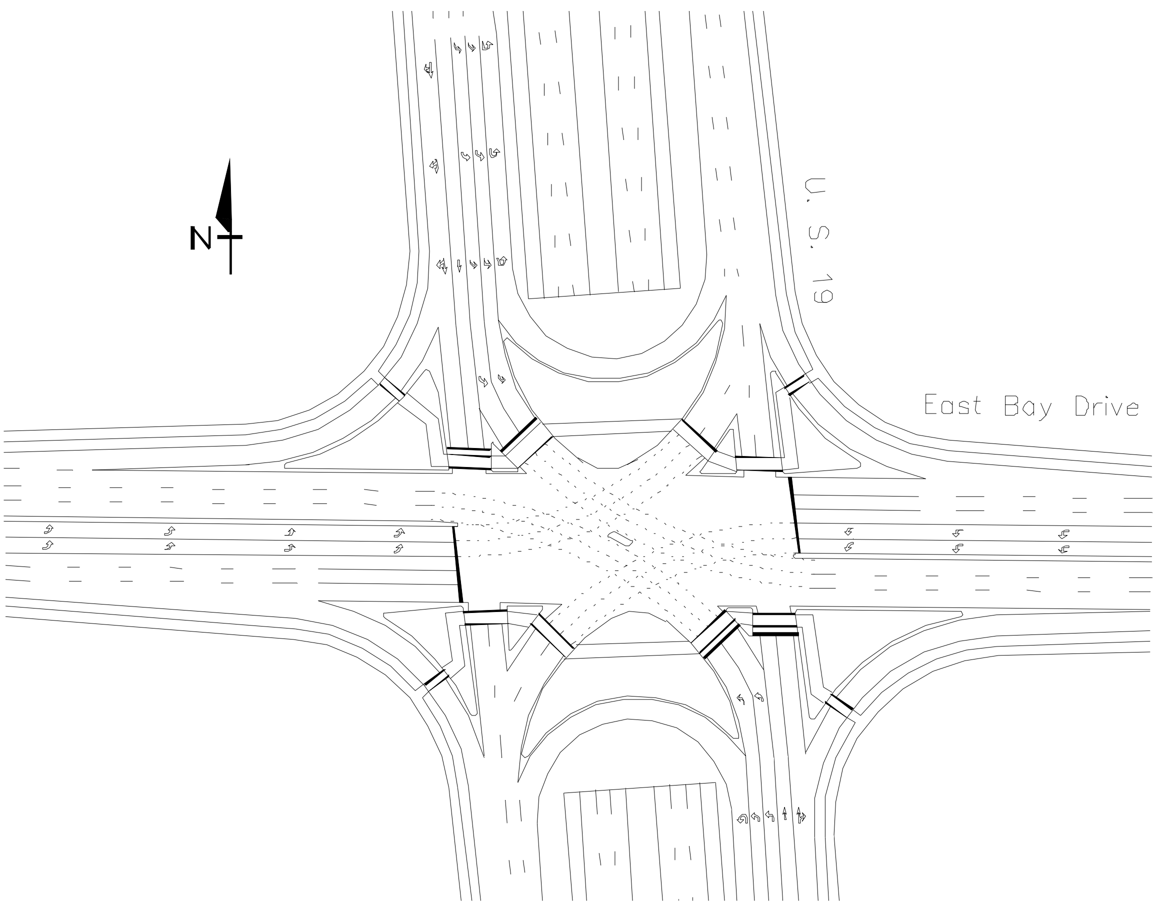

Figure B-1. Interchange at U.S. 19 and SR 686 in Largo, Florida.

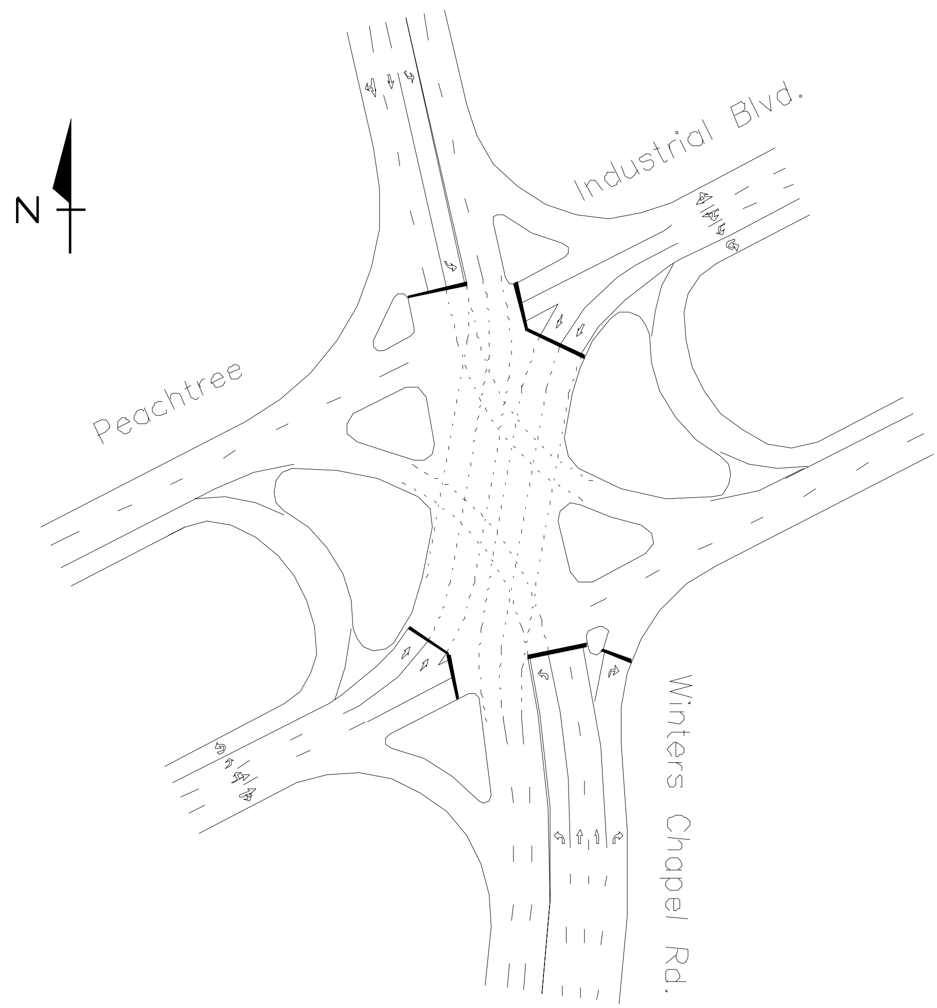

Figure B-2. Interchange at Peachtree Industrial Boulevard and Winters Chapel Road in Atlanta, Georgia.

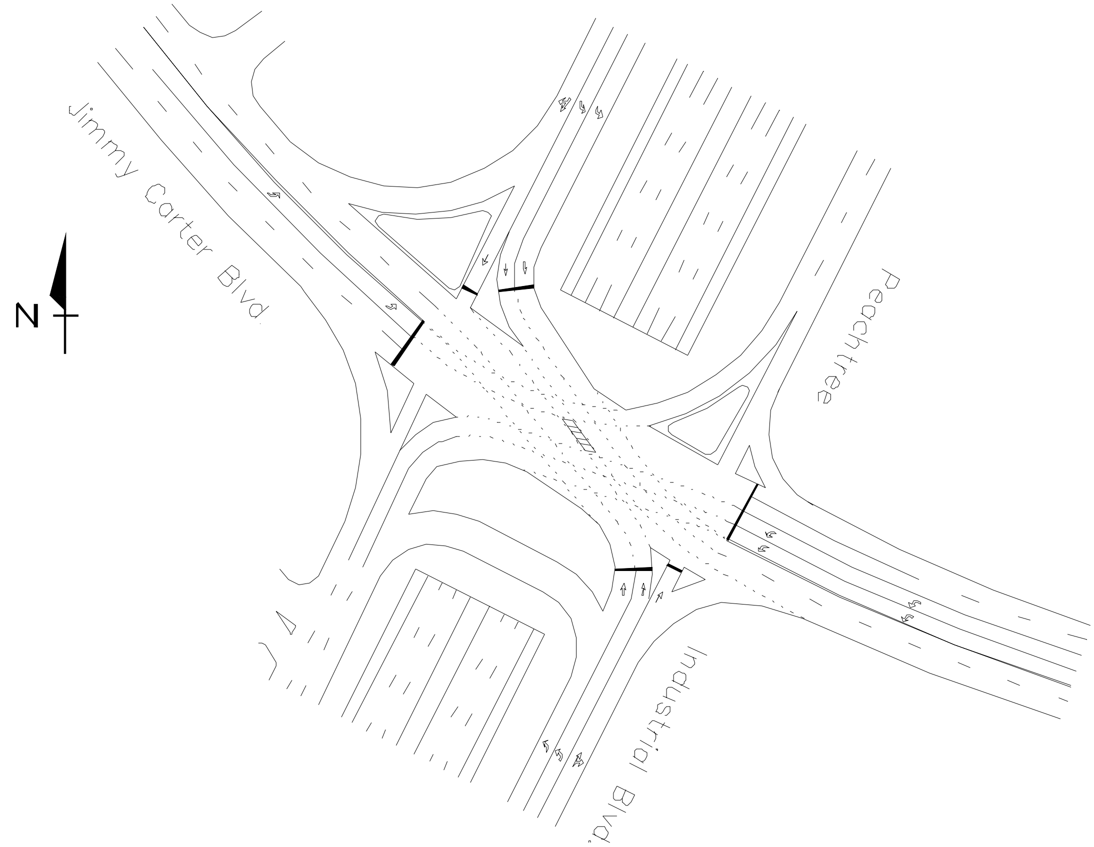

Figure B-3. Interchange at Peachtree Industrial Boulevard and Jimmy Carter Boulevard in Atlanta, Georgia.

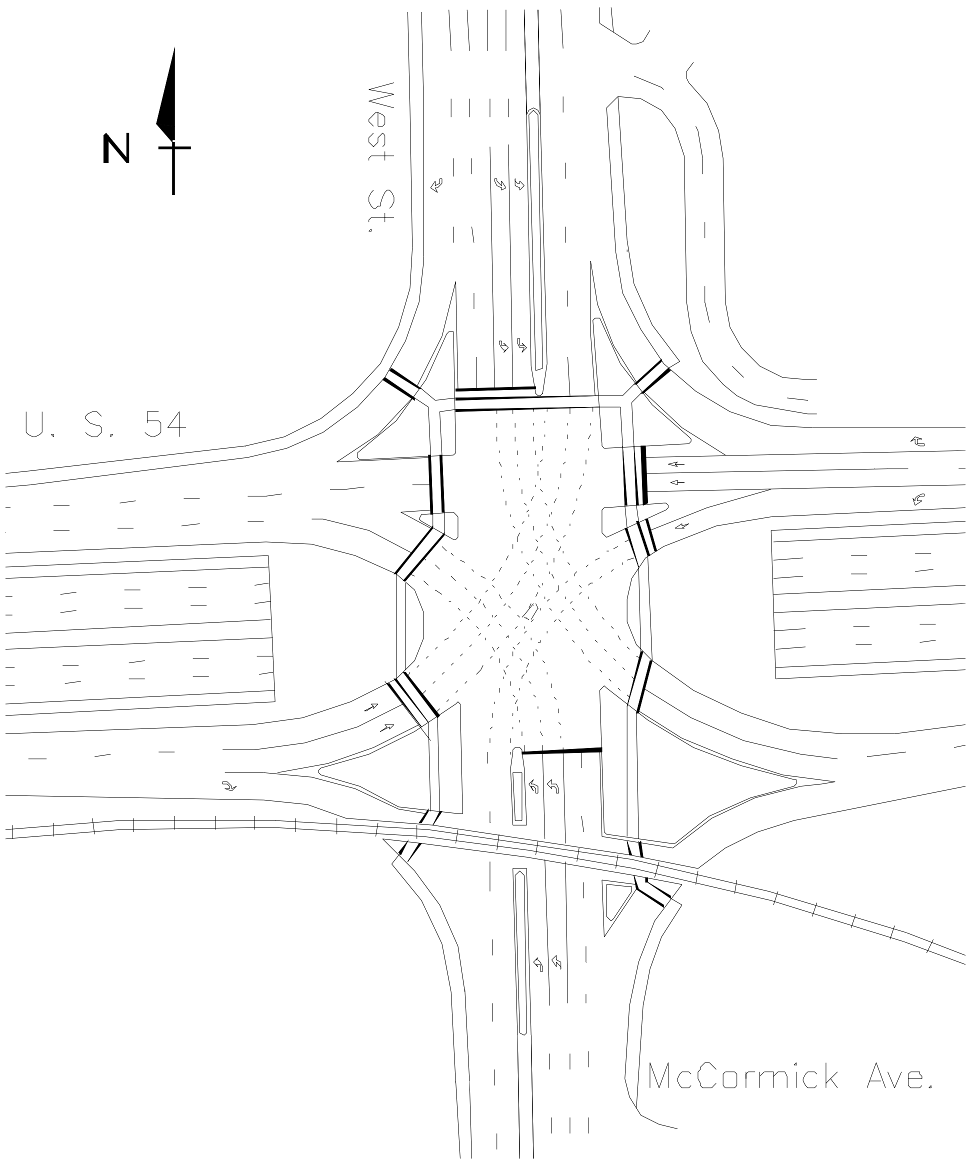

Figure B-4. Interchange at U.S. 54 (Kellog) and West Street in Wichita, Kansas.

APPENDIX C

User Cost of Delay Tables

4-Hour Peak Traffic Results

Low Lane Level

				4-phase SPUI				*3-ph TUDI no overlap*				*3-ph TUDI w/overlap*				*4-ph TUDI no overlap*				*4-ph TUDI w/overlap*			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	200	0.42	104.34	39.14	822.31	0.35	63.78	25.18	529.07	0.35	63.78	25.18	529.04	0.46	106.48	38.19	802.30	0.37	83.65	27.46	576.90
	2	35	200	0.47	114.50	42.55	893.89	0.34	62.13	24.78	520.55	0.34	62.13	24.78	520.55	0.48	109.02	39.28	825.33	0.39	85.33	28.23	593.21
	3	35	200	0.48	116.04	42.22	886.97	0.41	62.62	24.65	517.99	0.41	62.62	24.77	520.51	0.50	114.68	40.78	856.74	0.41	89.06	29.21	613.72
	4	35	200	0.53	129.50	46.50	977.05	0.40	61.12	24.32	510.91	0.40	61.12	24.42	512.96	0.52	117.98	42.12	884.85	0.42	91.21	30.13	633.05
	5	35	200	0.43	106.43	38.93	817.98	0.41	53.65	22.28	468.15	0.36	60.61	22.62	475.27	0.48	109.29	38.65	811.96	0.39	85.52	27.66	581.21
	6	35	200	0.49	117.41	42.50	892.99	0.40	52.34	21.95	461.25	0.35	59.20	22.26	467.70	0.49	112.10	40.42	849.21	0.40	87.37	28.49	598.52
	7	35	200	0.44	108.66	40.33	847.36	0.37	55.19	21.89	459.92	0.37	65.87	26.05	547.39	0.46	106.48	38.00	798.45	0.37	83.65	27.30	573.59
	8	35	200	0.50	119.79	44.01	924.55	0.36	53.82	21.56	452.91	0.36	64.23	25.65	538.90	0.48	109.02	39.09	821.34	0.39	85.33	28.07	589.81
	9	35	200	0.46	111.06	40.45	849.74	0.38	61.28	24.92	523.54	0.38	61.31	24.01	504.40	0.48	109.20	39.13	822.17	0.39	85.44	28.11	590.55
	10	35	200	0.51	123.16	44.32	931.16	0.37	59.79	24.58	516.36	0.37	59.80	23.63	496.42	0.49	112.10	40.36	848.01	0.40	87.37	28.98	608.77
	avg	35	200	0.47	115.09	42.09	**884.40**	0.38	58.57	23.61	**496.07**	0.37	62.07	24.34	**511.31**	0.48	110.64	39.60	**832.04**	0.39	86.39	28.36	**595.93**
50K	1	35	200	0.53	127.39	47.62	1250.71	0.43	66.48	27.25	715.64	0.43	66.48	27.25	715.64	0.58	135.85	48.86	1283.27	0.49	102.53	34.44	904.51
	2	35	200	0.59	147.49	54.48	1430.74	0.42	64.60	26.75	702.54	0.42	64.60	26.75	702.54	0.60	141.29	51.07	1341.23	0.50	105.87	35.87	941.92
	3	35	200	0.60	150.70	54.41	1428.86	0.51	65.70	27.15	713.07	0.51	65.70	26.57	697.87	0.63	153.56	54.65	1435.11	0.54	113.24	37.97	997.21
	4	35	200	0.67	181.31	64.39	1691.11	0.50	63.93	26.70	701.27	0.50	63.93	26.13	686.13	0.65	161.11	57.56	1511.63	0.55	117.65	39.75	1043.86
	5	35	200	0.54	131.40	47.81	1255.69	0.51	56.29	26.26	689.76	0.45	64.82	24.71	649.02	0.60	141.78	50.26	1319.87	0.51	106.20	35.17	923.73
	6	35	200	0.61	153.62	55.16	1448.66	0.50	54.74	25.61	672.57	0.44	63.09	24.26	637.01	0.61	147.87	52.67	1383.21	0.52	109.85	36.69	963.65
	7	35	200	0.55	135.69	50.11	1315.87	0.46	57.43	23.70	622.34	0.46	68.55	28.47	747.70	0.58	135.96	48.66	1277.94	0.49	102.63	34.28	900.23
	8	35	200	0.62	158.79	57.87	1519.66	0.45	55.84	23.27	611.13	0.45	66.65	27.96	734.22	0.60	141.29	50.81	1334.45	0.50	105.87	35.66	936.50
	9	35	200	0.57	140.43	50.78	1333.68	0.48	64.47	27.74	728.55	0.48	64.51	25.81	677.81	0.60	141.66	50.91	1337.01	0.51	106.10	35.73	938.29
	10	35	200	0.64	166.32	59.24	1555.83	0.47	62.69	27.24	715.47	0.47	62.69	25.33	665.31	0.61	147.87	53.40	1402.47	0.52	109.85	37.31	979.80
	avg	35	200	0.59	149.31	54.19	**1423.08**	0.47	61.22	26.17	**687.23**	0.46	65.10	26.32	**691.33**	0.61	144.82	51.89	**1362.62**	0.51	107.98	36.29	**952.97**
60K	1	35	200	0.63	163.76	61.04	1923.62	0.52	69.46	29.90	942.31	0.52	69.46	29.90	942.31	0.70	187.84	67.81	2137.11	0.60	132.62	45.58	1436.58
	2	35	200	0.71	187.66	71.81	2263.20	0.51	67.26	29.22	920.86	0.51	67.26	29.22	920.86	0.72	200.68	72.82	2294.86	0.62	139.45	48.36	1523.99
	3	35	200	0.72	202.89	75.37	2375.19	0.61	69.10	30.09	948.28	0.61	69.10	28.71	904.66	0.75	197.82	71.59	2256.08	0.66	155.43	53.31	1680.07
	4	35	200	0.80	212.11	92.04	2900.52	0.60	67.00	29.46	928.30	0.60	67.00	28.13	886.60	0.78	202.77	73.99	2331.65	0.68	165.66	57.27	1804.77
	5	35	200	0.65	171.67	62.16	1959.09	0.61	59.20	93.56	2948.51	0.54	70.02	27.50	866.52	0.72	185.60	67.59	2129.93	0.62	140.05	47.50	1497.00
	6	35	200	0.73	190.29	72.09	2271.74	0.60	57.37	83.11	2619.02	0.52	67.94	26.91	848.04	0.74	189.96	70.06	2207.91	0.65	147.92	50.62	1595.19
	7	35	200	0.67	180.63	66.38	2091.78	0.56	59.87	26.25	827.13	0.56	71.46	31.36	988.15	0.70	183.02	66.43	2093.51	0.60	132.75	45.37	1429.69
	8	35	200	0.74	199.44	77.06	2428.39	0.54	58.03	25.62	807.37	0.54	69.25	30.64	965.60	0.72	187.12	68.59	2161.69	0.62	139.45	48.08	1515.10
	9	35	200	0.68	190.93	68.58	2161.28	0.57	68.00	35.55	1120.47	0.57	68.05	27.96	880.99	0.72	201.57	72.70	2291.25	0.62	139.92	48.24	1520.39
	10	35	200	0.76	202.71	79.59	2508.17	0.56	65.88	33.64	1060.22	0.56	65.88	27.35	861.77	0.74	216.24	80.27	2529.73	0.65	147.92	51.46	1621.74
	avg	35	200	0.71	190.21	72.61	**2288.30**	0.57	64.12	41.64	**1312.25**	0.55	68.54	28.77	**906.55**	0.73	195.26	71.19	**2243.37**	0.63	144.12	49.58	**1562.45**

8-Hour Non-Peak Traffic Results

Low Lane Level

				4-phase SPUI				*3-ph TUDI no overlap*				*3-ph TUDI w/overlap*				*4-ph TUDI no overlap*				*4-ph TUDI w/overlap*			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	200	0.30	86.46	32.57	978.47	0.25	57.91	21.93	658.84	0.25	57.91	21.93	658.81	0.33	85.38	30.53	917.36	0.25	72.61	23.23	698.02
	2	35	200	0.34	91.23	34.15	1025.93	0.24	57.40	21.92	658.48	0.24	57.40	21.92	658.48	0.34	86.50	31.08	933.68	0.25	69.93	22.72	682.60
	3	35	200	0.34	91.93	33.84	1016.66	0.29	59.44	22.61	679.42	0.29	59.44	23.00	690.95	0.36	88.99	31.64	950.53	0.29	72.10	23.24	698.19
	4	35	200	0.38	97.68	35.63	1070.40	0.29	58.21	22.36	671.79	0.29	58.21	22.72	682.50	0.37	90.40	32.26	969.21	0.28	71.12	22.94	689.28
	5	35	200	0.31	87.48	32.41	973.79	0.29	50.92	20.01	601.30	0.26	56.89	20.81	625.16	0.34	86.66	30.58	918.77	0.26	73.81	23.24	698.09
	6	35	200	0.35	92.54	34.06	1023.35	0.29	49.85	19.78	594.32	0.25	55.68	20.50	615.82	0.35	87.87	31.15	935.84	0.26	70.90	22.46	674.71
	7	35	200	0.32	88.54	33.07	993.67	0.27	52.83	20.19	606.62	0.27	59.20	22.70	682.02	0.33	85.38	30.40	913.19	0.24	77.00	24.35	731.53
	8	35	200	0.35	93.59	34.71	1042.83	0.26	51.68	19.93	598.88	0.26	58.76	22.72	682.71	0.34	86.50	30.94	929.41	0.26	67.87	21.76	653.81
	9	35	200	0.33	89.68	33.08	993.82	0.27	58.01	22.54	677.17	0.27	58.04	22.24	668.15	0.34	86.58	30.95	929.87	0.27	69.72	22.53	676.89
	10	35	200	0.36	95.04	34.78	1044.81	0.27	56.80	22.30	669.87	0.27	56.81	21.94	659.24	0.35	87.87	31.55	948.01	0.27	68.71	22.23	667.75
	avg	35	400	0.34	91.42	33.83	**1016.37**	0.27	55.31	21.36	**641.67**	0.26	57.83	22.05	**662.38**	0.35	87.21	31.11	**934.59**	0.26	71.38	22.87	**687.09**
50K	1	35	200	0.38	96.82	36.37	1365.89	0.31	62.65	24.38	915.67	0.31	62.65	24.38	915.67	0.42	97.41	34.90	1310.49	0.32	78.55	25.51	958.09
	2	35	200	0.42	104.56	38.96	1462.99	0.30	61.14	24.03	902.47	0.30	61.14	24.03	902.47	0.43	99.38	35.77	1343.19	0.33	78.83	25.82	969.50
	3	35	200	0.43	105.70	38.62	1450.33	0.36	61.39	23.86	895.91	0.36	61.39	24.08	904.35	0.45	103.54	36.81	1382.45	0.36	82.18	26.75	1004.59
	4	35	200	0.48	115.48	41.69	1565.82	0.36	60.00	23.55	884.55	0.36	60.00	23.75	892.07	0.46	105.93	37.81	1419.82	0.37	83.26	27.61	1037.08
	5	35	200	0.39	98.48	36.19	1359.27	0.36	52.60	21.34	801.53	0.32	58.99	21.84	820.14	0.43	99.59	35.19	1321.44	0.34	79.02	25.30	950.09
	6	35	200	0.43	106.72	38.86	1459.28	0.36	51.38	21.06	790.79	0.31	57.68	21.51	807.73	0.44	101.65	36.08	1354.99	0.35	80.38	25.94	973.99
	7	35	200	0.40	100.17	37.27	1399.62	0.33	54.29	21.22	797.04	0.33	64.79	25.35	952.09	0.42	97.50	34.76	1305.44	0.32	78.61	25.39	953.43
	8	35	200	0.44	108.47	39.99	1501.75	0.32	53.00	20.92	785.61	0.32	63.26	24.98	938.27	0.43	99.37	35.60	1336.78	0.33	78.83	25.67	963.92
	9	35	200	0.41	101.99	37.32	1401.50	0.34	60.01	23.96	899.95	0.34	60.05	23.32	875.70	0.43	99.51	35.62	1337.81	0.34	79.01	25.79	968.52
	10	35	200	0.46	110.93	40.15	1507.79	0.33	58.64	23.67	888.76	0.33	58.65	22.97	862.73	0.44	101.65	36.56	1372.99	0.35	80.37	26.38	990.83
	avg	35	400	0.42	104.93	38.54	**1447.42**	0.34	57.51	22.80	**856.23**	0.33	60.86	23.62	**887.12**	0.43	100.55	35.91	**1348.54**	0.34	79.90	26.02	**977.00**
60K	1	35	200	0.45	110.14	41.27	1859.90	0.37	64.52	25.73	1159.61	0.37	64.52	25.73	1159.61	0.50	113.53	40.75	1836.30	0.41	88.31	29.18	1315.13
	2	35	200	0.51	122.44	45.42	2046.79	0.36	62.82	25.31	1140.71	0.36	62.82	25.31	1140.71	0.51	116.75	42.10	1897.49	0.42	90.41	30.13	1357.61
	3	35	200	0.51	124.33	45.13	2033.89	0.44	63.48	25.23	1136.83	0.44	63.48	25.27	1138.79	0.54	123.77	44.02	1983.62	0.45	94.94	31.34	1412.37
	4	35	200	0.57	141.21	50.54	2277.76	0.43	61.91	24.86	1120.55	0.43	61.91	24.89	1121.60	0.55	127.91	45.67	2058.10	0.46	97.57	32.44	1462.08
	5	35	200	0.46	112.63	41.14	1853.84	0.44	54.39	23.04	1038.43	0.38	61.78	23.19	1045.21	0.51	117.07	41.42	1866.84	0.42	90.64	29.52	1330.42
	6	35	200	0.52	126.02	45.51	2051.15	0.43	53.01	22.67	1021.81	0.37	60.27	22.80	1027.63	0.53	120.56	42.86	1931.46	0.44	92.88	30.50	1374.54
	7	35	200	0.48	115.32	43.41	1956.15	0.40	55.82	22.38	1008.37	0.40	66.63	26.89	1211.81	0.50	113.63	40.58	1828.92	0.41	88.39	29.05	1308.96
	8	35	200	0.53	128.98	47.27	2130.24	0.39	54.39	22.02	992.32	0.39	64.91	26.46	1192.54	0.51	116.76	41.90	1888.22	0.42	90.42	29.95	1349.86
	9	35	200	0.49	118.23	42.97	1936.35	0.41	62.17	25.63	1155.12	0.41	62.20	24.50	1104.23	0.51	116.97	41.95	1890.51	0.42	90.55	29.99	1351.65
	10	35	200	0.55	133.18	47.78	2153.42	0.40	60.61	25.26	1138.28	0.40	60.61	24.10	1086.03	0.53	120.56	43.44	1957.74	0.44	92.88	31.47	1418.15
	avg	35	400	0.51	123.25	45.04	**2029.95**	0.41	59.31	24.21	**1091.20**	0.40	62.91	24.92	**1122.82**	0.52	118.75	42.47	**1913.92**	0.43	91.70	30.36	**1368.08**

4-Hour Peak Traffic Results

Low Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	300	0.42	119.10	44.78	940.86	0.35	66.69	25.02	525.75	0.35	66.69	25.03	525.77	0.46	106.48	38.19	802.27	0.35	88.68	28.58	600.39
	2	35	300	0.47	130.70	48.67	1022.60	0.34	65.75	24.89	522.88	0.34	65.75	24.89	522.88	0.48	109.02	39.28	825.33	0.36	81.95	26.31	552.81
	3	35	300	0.48	132.46	48.30	1014.81	0.41	73.38	27.56	579.01	0.41	73.38	28.18	592.14	0.50	114.68	40.78	856.74	0.41	87.99	28.21	592.76
	4	35	300	0.53	147.82	53.19	1117.59	0.40	72.41	27.44	576.52	0.40	72.41	28.08	589.93	0.52	117.98	42.12	884.85	0.40	87.36	28.07	589.80
	5	35	300	0.43	121.48	44.55	935.97	0.41	65.65	25.29	531.28	0.36	67.76	24.48	514.39	0.48	109.29	38.65	811.96	0.36	90.05	28.23	593.11
	6	35	300	0.49	134.02	48.63	1021.73	0.40	64.05	24.90	523.16	0.35	66.78	24.32	510.86	0.49	112.10	39.83	836.73	0.37	84.80	26.82	563.54
	7	35	300	0.44	124.02	46.14	969.46	0.37	69.18	25.76	541.31	0.37	69.18	26.34	553.42	0.46	106.48	38.00	798.45	0.35	89.56	28.33	595.19
	8	35	300	0.50	136.75	50.34	1057.66	0.36	67.71	25.60	537.88	0.36	68.29	26.24	551.24	0.48	109.02	39.09	821.34	0.36	82.65	26.37	553.97
	9	35	300	0.46	126.76	46.28	972.27	0.38	70.41	26.76	562.23	0.38	70.41	26.76	562.23	0.48	109.20	39.13	822.17	0.38	84.17	27.05	568.35
	10	35	300	0.51	140.58	50.70	1065.22	0.37	69.48	26.65	559.97	0.37	69.48	26.65	559.97	0.49	112.10	40.36	848.01	0.37	83.83	27.00	567.35
	avg	35	300	0.47	131.37	48.16	**1011.82**	0.38	68.47	25.99	**546.00**	0.37	69.01	26.10	**548.28**	0.48	110.64	39.54	**830.79**	0.37	86.10	27.50	**577.73**
50K	1	35	300	0.53	145.41	54.47	1430.60	0.43	76.83	29.53	775.65	0.43	76.83	29.53	775.65	0.58	135.85	48.86	1283.27	0.46	97.59	31.97	839.60
	2	35	300	0.59	168.36	62.30	1636.04	0.42	75.38	29.25	768.30	0.42	75.38	29.25	768.30	0.60	141.29	51.07	1341.23	0.48	100.61	33.28	873.99
	3	35	300	0.60	172.03	62.22	1633.99	0.51	84.58	32.74	859.78	0.51	84.58	33.38	876.67	0.63	153.56	54.65	1435.11	0.51	107.24	35.16	923.48
	4	35	300	0.67	202.86	73.98	1942.75	0.50	82.30	32.18	845.06	0.50	82.30	32.81	861.61	0.65	161.11	57.56	1511.63	0.53	111.19	36.77	965.79
	5	35	300	0.54	149.98	54.69	1436.27	0.51	68.88	27.78	729.51	0.45	77.91	28.95	760.22	0.60	141.78	50.26	1319.87	0.48	100.91	32.60	856.03
	6	35	300	0.61	175.35	63.08	1656.64	0.50	66.99	27.27	716.07	0.44	75.69	28.34	744.37	0.61	147.87	52.67	1383.21	0.50	104.20	33.98	892.46
	7	35	300	0.55	154.87	57.30	1504.82	0.46	72.26	27.96	734.27	0.46	81.18	31.69	832.34	0.58	135.96	48.66	1277.94	0.46	97.68	31.81	835.45
	8	35	300	0.62	181.26	66.16	1737.57	0.45	70.25	27.44	720.55	0.45	79.62	31.39	824.35	0.60	141.29	50.81	1334.45	0.48	100.62	33.08	868.76
	9	35	300	0.57	160.29	58.08	1525.27	0.48	81.11	32.24	846.77	0.48	81.11	31.70	832.43	0.60	141.66	50.91	1337.01	0.48	100.82	33.14	870.38
	10	35	300	0.64	189.85	67.74	1778.92	0.47	78.87	31.68	831.86	0.47	78.87	31.12	817.29	0.61	147.87	53.40	1402.47	0.50	104.20	34.59	908.32
	avg	35	300	0.59	170.03	62.00	**1628.29**	0.47	75.74	29.81	**782.78**	0.46	79.35	30.82	**809.32**	0.61	144.82	51.89	**1362.62**	0.49	102.51	33.64	**883.43**
60K	1	35	300	0.63	186.91	69.78	2199.14	0.52	89.90	36.05	1135.96	0.52	89.90	36.05	1135.96	0.70	187.84	67.81	2137.11	0.58	124.47	41.97	1322.51
	2	35	300	0.71	198.15	77.74	2450.08	0.51	87.04	35.24	1110.73	0.51	87.04	35.24	1110.73	0.72	200.68	72.82	2294.86	0.60	130.47	44.43	1400.18
	3	35	300	0.72	211.20	82.18	2589.72	0.61	88.97	35.60	1121.87	0.61	88.97	35.87	1130.30	0.75	197.82	71.59	2256.08	0.64	144.35	48.71	1535.15
	4	35	300	0.80	219.63	103.02	3246.57	0.60	86.25	34.88	1099.20	0.60	86.25	35.15	1107.71	0.78	202.77	73.99	2331.65	0.66	153.13	52.15	1643.33
	5	35	300	0.65	194.85	73.10	2303.82	0.61	72.45	31.74	1000.18	0.54	82.51	31.25	984.68	0.72	185.60	67.59	2129.93	0.60	131.00	43.60	1373.90
	6	35	300	0.73	200.96	79.30	2499.11	0.60	70.21	30.86	972.46	0.52	79.83	30.49	960.97	0.74	189.96	70.06	2207.91	0.62	137.85	46.34	1460.44
	7	35	300	0.67	198.56	74.96	2362.41	0.56	75.32	30.23	952.58	0.56	94.64	38.18	1203.07	0.70	183.02	66.43	2093.51	0.58	124.59	41.76	1315.91
	8	35	300	0.74	207.91	85.68	2700.07	0.54	72.99	29.57	931.85	0.54	91.71	37.37	1177.79	0.72	187.12	68.59	2161.69	0.60	130.47	44.16	1391.76
	9	35	300	0.68	207.22	76.70	2417.10	0.57	85.56	35.75	1126.61	0.57	85.61	34.16	1076.46	0.72	201.57	72.70	2291.25	0.60	130.88	44.31	1396.44
	10	35	300	0.76	214.11	90.61	2855.56	0.56	82.89	34.96	1101.62	0.56	82.88	33.42	1053.11	0.74	216.24	80.27	2529.73	0.62	137.85	47.15	1485.86
	avg	35	300	0.71	203.95	81.31	**2562.36**	0.57	81.16	33.49	**1055.31**	0.55	86.93	34.72	**1094.08**	0.73	195.26	71.19	**2243.37**	0.61	134.51	45.46	**1432.55**

8-Hour Non-Peak Traffic Results

Low Lane Level

				4-phase SPUI				*3-ph TUDI no overlap*				*3-ph TUDI w/overlap*				*4-ph TUDI no overlap*				*4-ph TUDI w/overlap*			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	300	0.30	98.64	37.26	1119.49	0.25	57.84	21.10	634.08	0.25	57.84	21.10	634.08	0.33	85.31	30.51	916.61	0.26	73.24	22.84	686.33
	2	35	300	0.34	104.14	39.09	1174.31	0.24	57.40	21.11	634.19	0.24	57.40	21.11	634.19	0.34	86.50	31.08	933.72	0.26	73.24	22.87	687.01
	3	35	300	0.34	104.94	38.70	1162.57	0.29	61.35	22.43	673.90	0.29	61.35	22.91	688.42	0.36	88.99	31.64	950.53	0.29	76.02	23.70	711.90
	4	35	300	0.38	111.51	40.73	1223.79	0.29	60.89	22.44	674.12	0.29	60.89	22.93	689.05	0.37	90.40	32.26	969.21	0.28	78.93	24.60	739.08
	5	35	300	0.31	99.85	36.99	1111.39	0.29	61.35	22.50	676.06	0.26	58.48	20.58	618.21	0.34	86.66	30.58	918.77	0.26	79.16	24.25	728.69
	6	35	300	0.35	105.64	38.87	1167.87	0.29	60.89	22.51	676.32	0.25	57.96	20.53	616.74	0.35	87.87	31.15	935.84	0.25	80.68	24.61	739.27
	7	35	300	0.32	101.07	37.86	1137.45	0.27	59.27	21.48	645.31	0.27	59.20	21.90	658.11	0.33	85.38	30.40	913.19	0.26	78.01	24.09	723.73
	8	35	300	0.35	106.84	40.23	1208.77	0.26	58.76	21.45	644.57	0.26	58.76	21.92	658.42	0.34	86.50	30.94	929.41	0.26	78.27	24.16	725.89
	9	35	300	0.33	102.36	37.79	1135.40	0.27	59.84	22.11	664.37	0.27	59.84	22.11	664.37	0.34	86.58	30.95	929.87	0.27	73.87	23.11	694.21
	10	35	300	0.36	108.49	39.73	1193.52	0.27	59.39	22.13	664.79	0.27	59.39	22.13	664.79	0.35	87.87	31.55	948.01	0.27	75.91	23.71	712.47
	avg	35	300	0.34	104.35	38.73	**1163.45**	0.27	59.70	21.93	**658.77**	0.26	59.11	21.72	**652.64**	0.35	87.21	31.11	**934.52**	0.26	76.73	23.79	**714.86**
50K	1	35	300	0.38	110.52	41.63	1563.27	0.31	63.05	23.41	879.24	0.31	63.05	23.41	879.24	0.42	97.41	34.90	1310.49	0.31	81.69	25.81	969.19
	2	35	300	0.42	119.36	45.18	1696.83	0.30	62.37	23.36	877.10	0.30	62.37	23.36	877.10	0.43	99.38	35.77	1343.19	0.31	79.45	25.15	944.39
	3	35	300	0.43	120.66	44.16	1658.38	0.36	68.38	25.43	954.86	0.36	68.38	25.99	976.11	0.45	103.54	36.81	1382.45	0.36	81.45	25.90	972.77
	4	35	300	0.48	131.82	47.67	1790.35	0.36	67.64	25.37	952.65	0.36	67.64	25.95	974.42	0.46	105.93	37.81	1419.82	0.35	82.12	26.09	979.94
	5	35	300	0.39	112.40	41.32	1551.91	0.36	64.36	24.39	915.82	0.32	63.98	22.89	859.61	0.43	99.59	35.19	1321.44	0.32	84.37	26.22	984.51
	6	35	300	0.43	121.82	44.37	1666.35	0.36	62.88	24.04	902.72	0.31	63.19	22.77	855.30	0.44	101.65	36.08	1354.99	0.32	81.11	25.31	950.34
	7	35	300	0.40	114.34	42.65	1601.65	0.33	65.09	24.00	901.17	0.33	65.09	24.52	920.91	0.42	97.50	34.76	1305.44	0.30	86.49	27.05	1015.71
	8	35	300	0.44	123.82	45.75	1718.24	0.32	64.40	23.94	898.97	0.32	64.40	24.47	918.98	0.43	99.37	35.60	1336.78	0.32	79.43	25.02	939.65
	9	35	300	0.41	116.41	42.64	1601.25	0.34	66.06	24.85	933.07	0.34	66.06	24.85	933.07	0.43	99.51	35.62	1337.81	0.34	78.28	24.95	937.12
	10	35	300	0.46	126.62	45.88	1722.97	0.33	65.34	24.79	931.15	0.33	65.34	24.79	931.15	0.44	101.65	36.56	1372.99	0.33	78.59	25.03	940.09
	avg	35	300	0.42	119.78	44.13	**1657.12**	0.34	64.96	24.36	**914.68**	0.33	64.95	24.30	**912.59**	0.43	100.55	35.91	**1348.54**	0.33	81.30	25.65	**963.37**
60K	1	35	300	0.45	125.71	47.22	2127.89	0.37	69.28	26.18	1179.79	0.37	69.28	26.18	1179.79	0.50	113.53	40.75	1836.30	0.38	90.87	29.16	1314.28
	2	35	300	0.51	139.76	51.95	2341.19	0.36	68.28	26.03	1173.24	0.36	68.28	26.03	1173.24	0.51	116.75	42.10	1897.49	0.39	86.63	28.06	1264.61
	3	35	300	0.51	141.92	51.63	2326.71	0.44	77.23	29.20	1316.03	0.44	77.23	29.87	1346.25	0.54	123.77	44.02	1983.62	0.44	92.74	29.92	1348.33
	4	35	300	0.57	161.18	57.80	2604.96	0.43	76.07	29.03	1308.32	0.43	76.07	29.72	1339.15	0.55	127.91	45.67	2058.10	0.44	93.09	30.16	1359.26
	5	35	300	0.46	128.56	47.07	2121.17	0.44	66.55	25.95	1169.36	0.38	70.61	25.68	1157.51	0.51	117.07	41.42	1866.84	0.39	91.60	28.97	1305.49
	6	35	300	0.52	143.85	52.07	2346.40	0.43	64.87	25.53	1150.46	0.37	69.47	25.47	1148.04	0.53	120.56	42.86	1931.46	0.41	88.89	28.37	1278.62
	7	35	300	0.48	131.62	48.89	2203.37	0.40	70.23	26.56	1196.83	0.40	72.28	27.72	1249.34	0.50	113.63	40.58	1828.92	0.38	90.87	28.99	1306.47
	8	35	300	0.53	147.23	54.06	2436.48	0.39	68.42	26.11	1176.62	0.39	71.23	27.57	1242.60	0.51	116.76	41.90	1888.22	0.39	86.63	27.89	1257.02
	9	35	300	0.49	134.95	49.16	2215.27	0.41	73.73	28.22	1271.72	0.41	73.72	28.22	1271.67	0.51	116.97	41.95	1890.51	0.41	88.43	28.60	1288.69
	10	35	300	0.55	152.02	54.66	2463.07	0.40	72.62	28.06	1264.76	0.40	72.62	28.06	1264.76	0.53	120.56	43.44	1957.74	0.41	88.81	28.86	1300.44
	avg	35	300	0.51	140.68	51.45	**2318.65**	0.41	70.73	27.09	**1220.71**	0.40	72.08	27.45	**1237.23**	0.52	118.75	42.47	**1913.92**	0.40	89.85	28.90	**1302.32**

4-Hour Peak Traffic Results

Low Lane Level

				4-phase SPUI				*3-ph TUDI no overlap*				*3-ph TUDI w/overlap*				*4-ph TUDI no overlap*				*4-ph TUDI w/overlap*			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	400	0.42	133.85	50.43	1059.44	0.35	66.69	24.23	509.07	0.35	66.69	24.23	509.07	0.46	106.48	38.19	802.28	0.35	87.30	27.23	572.03
	2	35	400	0.47	146.90	54.80	1151.35	0.34	65.76	24.08	505.90	0.34	65.76	24.08	505.90	0.48	109.02	39.28	825.33	0.34	90.05	28.05	589.28
	3	35	400	0.48	148.87	54.39	1142.63	0.41	73.38	26.80	563.07	0.41	73.38	27.43	576.20	0.50	114.68	40.78	856.74	0.41	88.91	27.82	584.55
	4	35	400	0.53	166.15	59.88	1258.16	0.40	72.41	26.67	560.32	0.40	72.41	27.31	573.74	0.52	117.98	42.12	884.85	0.40	92.59	28.98	608.88
	5	35	400	0.43	136.53	50.17	1054.04	0.41	73.38	26.93	565.72	0.36	67.76	23.72	498.45	0.48	109.29	38.65	811.96	0.36	94.31	28.96	608.50
	6	35	400	0.49	150.63	54.76	1150.42	0.40	72.41	26.80	562.98	0.35	66.78	23.54	494.66	0.49	112.10	39.83	836.73	0.35	94.23	28.89	606.87
	7	35	400	0.44	139.38	51.96	1091.60	0.37	69.26	25.00	525.28	0.37	69.18	25.55	536.70	0.46	106.48	38.00	798.45	0.34	94.66	29.23	614.13
	8	35	400	0.50	153.69	56.68	1190.73	0.36	68.29	24.85	521.99	0.36	68.29	25.43	534.25	0.48	109.02	39.09	821.34	0.36	91.35	28.30	594.66
	9	35	400	0.46	142.46	52.11	1094.77	0.38	70.41	26.00	546.30	0.38	70.41	26.00	546.30	0.48	109.20	39.13	822.17	0.38	85.57	26.84	563.87
	10	35	400	0.51	158.01	57.08	1199.33	0.37	69.48	25.88	543.78	0.37	69.48	25.88	543.78	0.49	112.10	40.36	848.01	0.37	88.33	27.69	581.82
	avg	35	400	0.47	147.65	54.22	**1139.25**	0.38	70.15	25.72	**540.44**	0.37	69.01	25.32	**531.90**	0.48	110.64	39.54	**830.79**	0.37	90.73	28.20	**592.46**
50K	1	35	400	0.53	163.43	61.32	1610.44	0.43	76.83	28.74	754.75	0.43	76.83	28.74	754.75	0.58	135.85	48.86	1283.27	0.44	104.81	33.49	879.62
	2	35	400	0.59	189.22	70.11	1841.29	0.42	75.38	28.45	747.06	0.42	75.38	28.45	747.06	0.60	141.29	51.07	1341.23	0.45	95.94	30.90	811.41
	3	35	400	0.60	193.35	70.03	1839.13	0.51	88.62	33.31	874.78	0.51	88.62	34.12	895.97	0.63	153.56	54.65	1435.11	0.51	104.95	33.76	886.58
	4	35	400	0.67	213.57	79.04	2075.88	0.50	86.81	32.94	865.03	0.50	86.81	33.76	886.52	0.65	161.11	57.56	1511.63	0.50	105.40	34.03	893.74
	5	35	400	0.54	168.56	61.57	1616.87	0.51	81.47	31.33	822.70	0.45	78.72	28.35	744.57	0.60	141.78	50.26	1319.87	0.45	105.86	33.33	875.38
	6	35	400	0.61	192.78	70.88	1861.52	0.50	79.23	30.74	807.27	0.44	77.08	27.98	734.78	0.61	147.87	52.67	1383.21	0.47	99.96	31.74	833.67
	7	35	400	0.55	174.06	64.50	1693.83	0.46	81.18	30.17	792.31	0.46	81.18	30.90	811.43	0.58	135.96	48.66	1277.94	0.44	104.82	33.29	874.20
	8	35	400	0.62	200.31	74.66	1960.74	0.45	79.62	29.85	783.90	0.45	79.62	30.58	803.12	0.60	141.29	50.81	1334.45	0.45	97.89	31.79	834.79
	9	35	400	0.57	180.14	65.37	1716.79	0.48	83.32	31.69	832.32	0.48	83.32	31.69	832.32	0.60	141.66	50.91	1337.01	0.48	99.17	31.96	839.22
	10	35	400	0.64	202.80	74.51	1956.89	0.47	81.65	31.36	823.54	0.47	81.65	31.36	823.54	0.61	147.87	53.40	1402.47	0.47	99.17	32.08	842.55
	avg	35	400	0.59	187.82	69.20	**1817.34**	0.47	81.41	30.86	**810.37**	0.46	80.92	30.59	**803.40**	0.61	144.82	51.89	**1362.62**	0.47	101.80	32.64	**857.12**
60K	1	35	400	0.63	202.66	77.34	2437.48	0.52	90.76	34.95	1101.30	0.52	90.76	34.95	1101.30	0.70	187.84	67.81	2137.11	0.55	117.26	38.67	1218.75
	2	35	400	0.71	208.71	85.19	2684.76	0.51	88.31	34.32	1081.74	0.51	88.31	34.32	1081.74	0.72	200.68	72.82	2294.86	0.57	122.57	40.88	1288.39
	3	35	400	0.72	218.41	87.63	2761.53	0.61	108.83	42.29	1332.66	0.61	108.83	43.27	1363.61	0.75	197.82	71.59	2256.08	0.61	133.16	44.10	1389.67
	4	35	400	0.80	227.19	116.54	3672.57	0.60	105.50	41.41	1305.07	0.60	105.50	42.40	1336.17	0.78	202.77	73.99	2331.65	0.63	142.36	47.63	1501.02
	5	35	400	0.65	205.13	76.78	2419.73	0.61	85.69	34.62	1090.90	0.54	93.90	34.78	1096.04	0.72	185.60	67.59	2129.93	0.57	123.05	40.07	1262.69
	6	35	400	0.73	211.71	89.19	2810.72	0.60	83.04	33.83	1066.01	0.52	91.13	34.05	1072.97	0.74	189.96	70.06	2207.91	0.59	129.07	42.51	1339.59
	7	35	400	0.67	207.03	80.91	2549.75	0.56	90.78	35.07	1105.18	0.56	98.22	38.51	1213.72	0.70	183.02	66.43	2093.51	0.55	117.37	38.47	1212.48
	8	35	400	0.74	215.04	93.21	2937.59	0.54	87.96	34.29	1080.63	0.54	95.45	37.80	1191.20	0.72	187.12	68.59	2161.69	0.57	122.57	40.63	1280.38
	9	35	400	0.68	218.13	82.14	2588.60	0.57	102.02	39.96	1259.30	0.57	102.02	39.96	1259.25	0.72	201.57	72.70	2291.25	0.57	121.62	40.33	1271.05
	10	35	400	0.76	225.59	104.34	3288.29	0.56	99.00	39.19	1234.93	0.56	99.00	39.19	1234.93	0.74	216.24	80.27	2529.73	0.59	129.06	43.28	1364.08
	avg	35	200	0.71	213.96	89.33	**2815.10**	0.57	94.19	36.99	**1165.77**	0.55	97.31	37.92	**1195.09**	0.73	195.26	71.19	**2243.37**	0.58	125.81	41.66	**1312.81**

8-Hour Non-Peak Traffic Results

Low Lane Level

				4-phase SPUI				*3-ph TUDI no overlap*				*3-ph TUDI w/overlap*				*4-ph TUDI no overlap*				*4-ph TUDI w/overlap*			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	400	0.30	110.87	41.98	1261.12	0.25	57.84	20.31	610.16	0.25	57.91	20.34	610.99	0.33	85.38	30.53	917.38	0.28	84.89	25.61	769.34
	2	35	400	0.34	117.05	44.02	1322.63	0.24	57.40	20.30	609.90	0.24	57.40	20.30	609.89	0.34	86.50	31.08	933.69	0.28	84.89	25.64	770.22
	3	35	400	0.34	117.94	43.55	1308.46	0.29	61.35	21.67	651.12	0.29	61.35	22.16	665.63	0.36	88.99	31.64	950.53	0.29	86.36	26.05	782.71
	4	35	400	0.38	125.32	45.84	1377.17	0.29	60.89	21.67	650.95	0.29	60.89	22.16	665.89	0.37	90.40	32.66	981.32	0.28	87.58	26.45	794.55
	5	35	400	0.31	112.22	41.58	1249.11	0.29	61.35	21.74	653.27	0.26	58.48	19.82	595.42	0.34	86.66	30.58	918.77	0.27	88.97	26.61	799.57
	6	35	400	0.35	118.73	43.68	1312.41	0.29	60.89	21.74	653.12	0.25	57.96	19.76	593.58	0.35	87.87	31.15	935.84	0.27	88.88	26.61	799.55
	7	35	400	0.32	113.58	42.64	1281.17	0.27	59.27	20.68	621.40	0.27	59.20	21.11	634.20	0.33	85.38	30.40	913.19	0.28	86.73	26.09	783.95
	8	35	400	0.35	120.08	44.75	1344.38	0.26	58.76	20.65	620.28	0.26	58.76	21.11	634.12	0.34	86.50	30.94	929.41	0.28	86.45	26.05	782.53
	9	35	400	0.33	115.03	42.50	1276.98	0.27	59.84	21.35	641.58	0.27	59.84	21.35	641.58	0.34	86.58	30.95	929.87	0.27	84.90	25.65	770.67
	10	35	400	0.36	121.94	44.68	1342.23	0.27	59.39	21.36	641.63	0.27	59.39	21.36	641.63	0.35	87.87	31.55	948.01	0.27	84.90	25.68	771.58
	avg	35	400	0.34	117.28	43.52	**1307.57**	0.27	59.70	21.15	**635.34**	0.26	59.12	20.95	**629.29**	0.35	87.21	31.15	**935.80**	0.28	86.45	26.04	**782.47**
50K	1	35	400	0.38	124.21	46.88	1760.58	0.31	63.05	22.62	849.35	0.31	63.05	22.62	849.35	0.42	97.41	34.90	1310.49	0.32	84.90	26.18	983.20
	2	35	400	0.42	134.15	50.19	1885.01	0.30	62.37	22.55	846.74	0.30	62.37	22.55	846.74	0.43	99.38	35.77	1343.19	0.32	84.90	26.22	984.60
	3	35	400	0.43	135.61	49.73	1867.55	0.36	68.38	24.67	926.38	0.36	68.38	25.23	947.62	0.45	103.54	36.81	1382.45	0.36	87.58	27.01	1014.43
	4	35	400	0.48	148.16	53.68	2015.92	0.36	67.64	24.60	923.69	0.36	67.64	25.18	945.46	0.46	105.93	37.81	1419.82	0.35	90.37	27.94	1049.27
	5	35	400	0.39	126.32	46.54	1747.85	0.36	68.38	24.77	930.30	0.32	63.98	22.13	831.13	0.43	99.59	35.19	1321.44	0.32	91.59	27.85	1045.74
	6	35	400	0.43	136.92	49.97	1876.55	0.36	67.64	24.70	927.58	0.31	63.19	22.00	826.34	0.44	101.65	36.08	1354.99	0.31	93.40	28.28	1062.07
	7	35	400	0.40	128.50	48.03	1803.62	0.33	65.17	23.23	872.44	0.33	65.09	23.73	891.02	0.42	97.50	34.76	1305.44	0.32	90.58	27.70	1040.36
	8	35	400	0.44	139.17	51.52	1934.81	0.32	64.40	23.13	868.59	0.32	64.40	23.66	888.61	0.43	99.37	35.60	1336.78	0.32	90.56	27.71	1040.81
	9	35	400	0.41	130.83	48.02	1803.33	0.34	66.06	24.09	904.58	0.34	66.06	24.09	904.58	0.43	99.51	35.62	1337.81	0.34	84.99	26.27	986.38
	10	35	400	0.46	142.32	51.66	1940.22	0.33	65.34	24.02	902.20	0.33	65.34	24.02	902.20	0.44	101.65	36.56	1372.99	0.33	86.48	26.78	1005.82
	avg	35	400	0.42	134.62	49.62	**1863.54**	0.34	65.84	23.84	**895.19**	0.33	64.95	23.52	**883.31**	0.43	100.55	35.91	**1348.54**	0.33	88.54	27.19	**1021.27**
60K	1	35	400	0.45	141.28	53.16	2395.79	0.37	69.28	25.38	1143.92	0.37	69.28	25.38	1143.92	0.50	113.53	40.75	1836.30	0.37	92.05	28.90	1302.27
	2	35	400	0.51	157.09	58.48	2635.67	0.36	68.28	25.23	1136.81	0.36	68.28	25.23	1136.81	0.51	116.75	42.10	1897.49	0.37	91.37	28.70	1293.50
	3	35	400	0.51	159.51	58.97	2657.52	0.44	77.23	28.44	1281.85	0.44	77.23	29.11	1312.07	0.54	123.77	44.02	1983.62	0.44	92.12	29.10	1311.59
	4	35	400	0.57	181.16	65.07	2932.24	0.43	76.07	28.26	1273.58	0.43	76.07	28.94	1304.40	0.55	127.91	45.67	2058.10	0.43	94.18	29.73	1339.82
	5	35	400	0.46	144.49	53.00	2388.51	0.44	77.23	28.59	1288.45	0.38	70.61	24.93	1123.33	0.51	117.07	41.42	1866.84	0.38	97.15	30.02	1353.04
	6	35	400	0.52	161.69	58.62	2641.73	0.43	76.07	28.41	1280.11	0.37	69.47	24.70	1113.29	0.53	120.56	42.86	1931.46	0.38	93.79	29.06	1309.49
	7	35	400	0.48	147.93	55.05	2480.72	0.40	72.36	26.35	1187.31	0.40	72.28	26.93	1213.48	0.50	113.63	40.58	1828.92	0.36	98.33	30.59	1378.38
	8	35	400	0.53	165.48	60.86	2742.82	0.39	71.23	26.14	1178.12	0.39	71.23	26.76	1206.16	0.51	116.76	41.90	1888.22	0.39	91.89	28.72	1294.19
	9	35	400	0.49	151.67	55.35	2494.20	0.41	73.72	27.46	1237.49	0.41	73.72	27.46	1237.49	0.51	116.97	41.95	1890.51	0.41	87.88	27.81	1253.44
	10	35	400	0.55	170.87	61.53	2772.81	0.40	72.62	27.29	1230.01	0.40	72.62	27.29	1230.01	0.53	120.56	43.44	1957.74	0.40	89.64	28.35	1277.44
	avg	35	400	0.51	158.12	58.01	**2614.20**	0.41	73.41	27.16	**1223.76**	0.40	72.08	26.67	**1202.10**	0.52	118.75	42.47	**1913.92**	0.39	92.84	29.10	**1311.32**

4-Hour Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	**run**	**speed**	**r-to-r dist**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**	**v/s**	**cycle**	**delay**	**'01 cost of delay $1K**
40K	1	35	200	0.32	89.35	33.40	701.65	0.32	60.93	24.04	505.13	0.32	60.93	24.04	505.13	0.41	95.98	34.55	725.85	0.32	75.34	24.62	517.22
	2	35	200	0.36	93.76	34.96	734.50	0.31	60.32	23.91	502.42	0.31	60.32	23.91	502.42	0.40	94.97	34.30	720.68	0.31	75.13	24.64	517.69
	3	35	200	0.37	96.10	34.95	734.38	0.37	55.68	22.41	470.75	0.35	59.35	22.84	479.78	0.44	102.27	36.33	763.31	0.35	80.79	26.19	550.24
	4	35	200	0.41	101.58	36.74	771.94	0.37	55.17	22.29	468.35	0.35	58.81	22.69	476.73	0.44	101.26	36.09	758.15	0.34	80.13	26.06	547.59
	5	35	200	0.34	91.56	33.14	696.26	0.37	49.45	20.82	437.35	0.32	58.81	21.86	459.33	0.41	96.59	34.18	718.01	0.32	81.51	25.99	545.98
	6	35	200	0.37	96.39	34.76	730.35	0.37	49.00	20.74	435.67	0.31	58.32	21.74	456.67	0.40	95.54	33.92	712.67	0.31	80.09	25.63	538.57
	7	35	200	0.34	91.61	33.97	713.62	0.32	60.90	24.05	505.34	0.32	60.90	24.05	505.32	0.41	95.90	34.55	725.79	0.32	75.34	24.64	517.61
	8	35	200	0.37	96.08	35.52	746.35	0.31	60.32	23.94	502.96	0.31	60.32	23.94	502.96	0.40	94.97	34.33	721.20	0.31	75.13	24.66	518.08
	9	35	200	0.36	93.80	34.01	714.47	0.37	52.57	21.38	449.13	0.33	58.95	22.35	469.54	0.43	99.30	35.27	741.08	0.33	78.78	25.42	534.06
	10	35	200	0.39	98.92	35.69	749.87	0.37	52.08	21.27	446.96	0.33	58.41	22.21	466.57	0.42	98.32	35.04	736.22	0.33	78.11	25.30	531.53
	avg	35	200	0.36	94.91	34.71	**729.34**	0.35	55.64	22.49	**472.41**	0.32	59.51	22.96	**482.45**	0.41	97.51	34.86	**732.30**	0.32	78.03	25.31	**531.86**
50K	1	35	200	0.40	101.40	37.80	992.64	0.39	63.54	25.89	679.94	0.39	63.54	25.89	679.90	0.51	115.66	41.73	1096.01	0.42	89.70	29.89	784.84
	2	35	200	0.44	108.75	40.29	1058.14	0.39	62.87	25.74	676.03	0.39	62.87	25.74	676.03	0.50	113.92	41.24	1083.11	0.41	88.58	29.60	777.42
	3	35	200	0.46	112.73	40.75	1070.06	0.47	58.43	24.41	641.16	0.44	62.49	24.57	645.24	0.55	127.63	45.35	1190.87	0.46	97.40	32.19	845.36
	4	35	200	0.51	122.42	43.80	1150.21	0.47	57.84	24.27	637.37	0.44	61.85	24.38	640.37	0.55	125.63	44.76	1175.53	0.45	96.15	31.86	836.79
	5	35	200	0.42	105.07	37.83	993.40	0.47	51.90	24.70	648.76	0.40	62.29	23.64	620.92	0.51	116.86	41.43	1088.14	0.42	90.50	29.53	775.63
	6	35	200	0.47	113.23	40.45	1062.22	0.47	51.37	24.72	649.09	0.39	61.71	23.48	616.69	0.50	114.94	40.89	1073.85	0.41	89.25	29.22	767.25
	7	35	200	0.42	104.83	38.73	1017.09	0.39	63.54	25.94	681.26	0.39	63.54	25.94	681.23	0.51	115.66	41.77	1097.04	0.42	89.70	29.91	785.59
	8	35	200	0.46	112.70	41.34	1085.80	0.39	62.87	25.79	677.39	0.39	62.87	25.79	677.39	0.50	113.92	41.28	1084.19	0.41	88.58	29.63	778.18
	9	35	200	0.44	108.79	39.20	1029.47	0.47	55.17	23.75	623.63	0.42	62.23	24.09	632.72	0.53	121.95	43.36	1138.85	0.44	93.78	30.87	810.79
	10	35	200	0.49	117.65	42.00	1103.04	0.47	54.61	23.63	620.48	0.41	61.60	23.91	627.91	0.52	120.05	42.82	1124.64	0.43	92.58	30.57	802.82
	avg	35	200	0.45	110.76	40.22	**1056.21**	0.44	58.21	24.88	**653.51**	0.41	62.50	24.74	**649.84**	0.52	118.62	42.47	**1115.22**	0.43	91.62	30.33	**796.47**
60K	1	35	200	0.48	117.37	43.64	1375.39	0.47	66.43	28.16	887.34	0.47	66.42	28.16	887.31	0.61	145.66	52.71	1661.22	0.52	108.52	36.88	1162.40
	2	35	200	0.53	129.46	47.72	1503.94	0.47	65.65	27.97	881.34	0.47	65.65	27.97	881.34	0.60	142.32	51.66	1628.16	0.51	106.52	36.29	1143.80
	3	35	200	0.56	136.32	48.97	1543.38	0.56	61.48	27.05	852.42	0.52	65.94	26.69	840.97	0.66	169.73	60.33	1901.37	0.57	122.59	41.30	1301.52
	4	35	200	0.61	154.02	54.62	1721.42	0.56	60.79	26.87	846.87	0.52	65.23	26.47	834.32	0.66	165.45	58.96	1858.11	0.56	120.18	40.56	1278.35
	5	35	200	0.51	123.24	44.14	1391.09	0.56	54.60	101.86	3210.12	0.47	67.79	26.65	839.88	0.61	147.89	52.57	1656.83	0.52	109.90	36.61	1153.66
	6	35	200	0.56	137.21	48.63	1532.58	0.56	53.99	105.48	3324.09	0.47	67.39	26.59	838.09	0.60	144.25	51.44	1621.04	0.51	107.70	35.96	1133.22
	7	35	200	0.51	122.86	45.22	1425.09	0.47	66.42	28.29	891.50	0.47	66.47	28.31	892.22	0.61	145.66	52.78	1663.28	0.52	108.52	36.93	1163.82
	8	35	200	0.56	136.27	49.68	1565.53	0.47	65.65	28.10	885.62	0.47	65.65	28.10	885.63	0.60	142.32	51.73	1630.19	0.51	106.52	36.34	1145.22
	9	35	200	0.53	129.50	46.38	1461.67	0.56	58.04	28.31	892.08	0.50	65.87	26.25	827.21	0.64	157.99	56.26	1773.12	0.55	115.84	38.89	1225.74
	10	35	200	0.58	145.13	51.37	1618.78	0.56	57.39	28.27	891.02	0.50	65.17	26.04	820.64	0.63	154.12	55.05	1734.74	0.54	113.59	38.23	1204.72
	avg	35	200	0.54	133.14	48.04	**1513.89**	0.53	61.04	43.04	**1356.24**	0.49	66.16	27.12	**854.76**	0.62	151.54	54.35	**1712.80**	0.53	111.99	37.80	**1191.25**
70K	1	35	200	0.57	139.20	51.64	1898.82	0.55	69.58	31.35	1152.65	0.55	69.58	31.35	1152.68	0.71	190.99	70.70	2599.51	0.62	137.35	47.63	1751.10
	2	35	200	0.62	159.90	58.70	2158.14	0.55	68.69	31.12	1144.07	0.55	68.69	31.12	1144.07	0.70	189.58	69.04	2538.43	0.61	133.59	46.40	1706.09
	3	35	200	0.65	172.41	61.58	2264.07	0.65	64.85	32.43	1192.47	0.61	69.84	30.15	1108.42	0.78	202.88	76.02	2795.13	0.68	165.36	56.81	2088.90
	4	35	200	0.71	207.18	74.29	2731.41	0.65	64.05	32.31	1187.95	0.61	69.00	29.92	1100.06	0.76	201.76	74.94	2755.20	0.67	160.22	55.10	2025.90
	5	35	200	0.59	149.03	53.11	1952.72	0.65	57.60	267.12	9821.08	0.55	76.75	40.28	1481.15	0.72	187.17	69.10	2540.72	0.62	139.86	47.56	1748.70
	6	35	200	0.65	174.06	61.25	2251.81	0.65	56.88	273.05	10039.39	0.55	76.16	41.59	1529.21	0.71	183.65	66.68	2451.72	0.61	135.75	46.24	1699.95
	7	35	200	0.59	148.37	54.42	2000.83	0.55	69.58	32.11	1180.52	0.55	69.62	32.14	1181.84	0.71	190.99	70.81	2603.30	0.62	137.35	47.70	1753.71
	8	35	200	0.65	172.30	62.47	2296.70	0.55	68.69	31.90	1172.70	0.55	68.69	31.90	1172.73	0.70	189.58	69.15	2542.32	0.61	133.59	46.47	1708.67
	9	35	200	0.62	159.92	56.95	2093.87	0.65	61.22	98.98	3639.19	0.58	70.01	30.01	1103.50	0.75	194.87	72.49	2665.41	0.65	151.47	51.89	1907.70
	10	35	200	0.68	189.35	66.47	2444.08	0.65	60.47	101.49	3731.37	0.58	69.17	29.80	1095.72	0.73	193.08	70.27	2583.58	0.64	146.97	50.42	1853.66
	avg	35	200	0.63	167.17	60.09	**2209.25**	0.61	64.16	93.19	**3426.14**	0.57	70.75	32.83	**1206.94**	0.73	192.45	70.92	**2607.53**	0.63	144.15	49.62	**1824.44**
80K	1	35	200	0.65	170.85	64.14	2695.13	0.63	73.10	41.58	1746.98	0.63	73.10	41.58	1746.98	0.81	206.06	88.37	3713.07	0.72	186.88	66.14	2779.27
	2	35	200	0.71	195.06	73.42	3085.16	0.63	72.02	41.21	1731.66	0.63	72.02	41.21	1731.66	0.80	207.36	82.96	3486.12	0.71	179.08	63.44	2665.91
	3	35	200	0.74	195.54	75.19	3159.62	0.75	68.62	101.12	4249.20	0.70	74.23	66.68	2801.93	0.89	209.38	105.24	4422.20	0.79	189.96	67.60	2840.67
	4	35	200	0.81	214.91	91.85	3859.58	0.75	67.68	102.74	4316.91	0.70	73.24	67.56	2839.02	0.87	208.06	88.87	3734.42	0.78	184.04	65.48	2751.35
	5	35	200	0.68	177.07	64.60	2714.44	0.75	60.94	455.01	19119.17	0.63	88.43	146.28	6146.76	0.82	191.71	77.71	3265.38	0.73	170.08	60.16	2527.83
	6	35	200	0.75	196.64	74.82	3143.95	0.75	60.11	463.08	19458.57	0.63	87.51	150.22	6312.03	0.81	190.18	74.77	3141.91	0.71	164.68	58.23	2446.92
	7	35	200	0.68	179.80	67.23	2825.06	0.63	73.05	71.57	3007.17	0.63	73.10	71.93	3022.62	0.81	206.00	88.42	3715.19	0.72	187.03	66.31	2786.47
	8	35	200	0.74	206.13	77.86	3271.76	0.63	72.02	71.98	3024.59	0.63	72.02	71.99	3024.92	0.80	207.36	83.10	3491.97	0.71	179.08	63.56	2670.68
	9	35	200	0.71	185.72	69.88	2936.19	0.75	64.78	238.94	10040.29	0.67	75.87	77.20	3244.11	0.85	200.35	87.58	3679.86	0.76	179.63	63.88	2684.12
	10	35	200	0.78	205.53	81.09	3407.51	0.75	63.89	243.00	10210.85	0.66	75.05	79.43	3337.64	0.84	198.91	79.97	3360.38	0.75	174.00	61.87	2599.82
	avg	35	200	0.72	192.73	74.01	**3109.84**	0.70	67.62	183.02	**7690.54**	0.65	76.46	81.41	**3420.77**	0.83	202.54	85.70	**3601.05**	0.74	179.45	63.67	**2675.30**
90K	1	35	200	0.73	195.56	75.18	3553.67	0.71	76.89	170.43	8056.47	0.71	76.89	170.44	8057.04	0.91	219.49	176.92	8363.26	0.82	207.22	86.93	4109.55
	2	35	200	0.80	210.74	97.20	4595.05	0.71	75.70	170.82	8075.01	0.71	75.70	170.82	8075.03	0.90	216.90	137.57	6503.20	0.81	204.66	80.22	3792.20
	3	35	200	0.84	211.76	108.74	5140.21	0.84	72.85	292.17	13811.38	0.79	79.21	166.35	7863.71	1.00	216.34	194.83	9209.78	0.90	204.91	92.47	4371.04
	4	35	200	0.91	222.30	174.31	8240.13	0.84	71.75	295.64	13975.43	0.78	78.03	168.00	7941.74	0.98	214.78	145.50	6878.18	0.89	201.02	74.50	3521.75
	5	35	200	0.76	186.87	74.96	3543.67	0.84	64.70	694.01	32807.35	0.71	91.71	225.35	10652.88	0.92	196.49	132.69	6272.63	0.83	183.50	68.97	3260.15
	6	35	200	0.84	202.15	107.61	5087.12	0.84	63.72	704.12	33284.92	0.71	90.18	226.87	10724.55	0.91	194.75	106.57	5037.79	0.81	176.93	66.10	3124.80
	7	35	200	0.76	213.22	83.34	3939.53	0.71	76.89	191.52	9053.65	0.71	76.94	192.12	9081.86	0.91	219.48	179.78	8498.31	0.82	207.22	87.92	4155.96
	8	35	200	0.83	221.69	116.93	5527.46	0.71	75.70	192.41	9095.75	0.71	75.69	192.36	9093.17	0.90	216.90	140.65	6648.64	0.81	204.66	81.24	3840.41
	9	35	200	0.80	198.73	87.04	4114.47	0.84	68.78	445.31	21050.64	0.75	87.26	213.42	10088.98	0.96	206.12	157.12	7427.41	0.87	195.12	78.06	3689.85
	10	35	200	0.88	212.25	137.22	6486.79	0.84	67.74	451.26	21331.85	0.75	86.09	216.71	10244.07	0.94	204.48	123.68	5846.41	0.85	188.42	70.14	3315.88
	avg	35	200	0.81	207.53	106.25	**5022.81**	0.79	71.47	360.77	**17054.25**	0.73	81.77	194.24	**9182.30**	0.93	210.57	149.53	**7068.56**	0.84	197.37	78.65	**3718.16**

8-Hour Non-Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	run	speed	r-to-r dist	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K
40K	1	35	200	0.23	81.71	30.45	914.86	0.23	56.18	21.36	641.80	0.23	56.18	21.36	641.80	0.29	80.34	28.85	866.82	0.23	62.76	20.24	608.06
	2	35	200	0.25	81.07	30.46	915.20	0.22	56.10	21.42	643.45	0.22	56.10	21.42	643.45	0.29	79.83	28.77	864.39	0.23	62.36	20.08	603.27
	3	35	200	0.27	86.02	31.34	941.70	0.27	52.84	20.42	613.48	0.25	56.10	21.13	634.97	0.32	83.38	29.63	890.29	0.25	67.06	21.46	644.87
	4	35	200	0.29	85.12	31.26	939.28	0.27	52.41	20.33	610.93	0.25	55.68	21.04	632.25	0.31	82.93	29.57	888.44	0.23	72.77	23.03	691.79
	5	35	200	0.24	82.90	30.12	904.87	0.27	46.93	18.71	562.09	0.23	55.30	20.16	605.78	0.29	80.64	28.48	855.66	0.23	71.00	22.32	670.57
	6	35	200	0.27	82.47	30.24	908.66	0.27	46.54	18.64	560.13	0.22	54.88	20.06	602.80	0.29	80.12	28.40	853.20	0.22	69.81	21.89	657.69
	7	35	200	0.24	83.19	30.79	924.96	0.23	56.11	21.34	641.27	0.23	56.12	21.34	641.28	0.29	80.27	28.84	866.54	0.23	62.76	20.25	608.40
	8	35	200	0.27	82.30	30.71	922.80	0.22	56.10	21.43	643.75	0.22	56.10	21.43	643.70	0.29	79.83	28.79	864.84	0.23	62.36	20.09	603.58
	9	35	200	0.25	84.44	30.70	922.48	0.27	49.89	19.52	586.35	0.24	55.59	20.66	620.68	0.30	81.95	29.09	874.02	0.24	68.86	21.87	657.01
	10	35	200	0.28	83.77	30.73	923.15	0.27	49.47	19.44	584.04	0.24	55.15	20.56	617.61	0.30	81.50	29.03	872.27	0.22	69.78	22.01	661.32
	avg	35	200	0.26	83.30	30.68	**921.80**	0.25	52.26	20.26	**608.73**	0.23	55.72	20.92	**628.43**	0.30	81.08	28.95	**869.65**	0.23	66.95	21.32	**640.66**
50K	1	35	200	0.29	86.27	32.21	1209.67	0.28	59.84	23.32	875.64	0.28	59.84	23.32	875.65	0.36	89.41	32.15	1207.55	0.28	70.79	23.01	864.07
	2	35	200	0.32	88.57	33.12	1243.73	0.28	59.30	23.22	871.94	0.28	59.30	23.22	871.94	0.36	88.69	32.01	1202.07	0.28	70.65	23.00	863.77
	3	35	200	0.33	92.06	33.51	1258.29	0.33	54.58	21.53	808.45	0.31	58.11	22.18	832.82	0.40	94.30	33.51	1258.28	0.31	74.90	24.16	907.15
	4	35	200	0.36	94.73	34.45	1293.92	0.33	54.10	21.42	804.53	0.31	57.60	22.05	828.03	0.39	93.55	33.34	1252.23	0.30	77.22	24.86	933.67
	5	35	200	0.30	87.92	31.87	1196.85	0.33	48.48	19.93	748.64	0.28	57.47	21.20	796.34	0.37	89.96	31.80	1194.42	0.28	76.50	24.25	910.78
	6	35	200	0.33	90.66	32.89	1235.22	0.33	48.05	19.86	745.78	0.28	56.99	21.08	791.71	0.36	89.14	31.63	1187.78	0.27	76.90	24.36	914.85
	7	35	200	0.30	88.29	32.71	1228.54	0.28	59.84	23.34	876.41	0.28	59.84	23.34	876.40	0.36	89.41	32.17	1208.31	0.28	70.84	23.04	865.33
	8	35	200	0.33	90.41	33.54	1259.77	0.28	59.30	23.24	872.71	0.28	59.30	23.24	872.71	0.36	88.69	32.03	1202.86	0.28	70.65	23.02	864.35
	9	35	200	0.32	89.99	32.66	1226.63	0.33	51.54	20.64	775.08	0.30	57.65	21.69	814.44	0.38	92.04	32.68	1227.42	0.30	73.86	23.69	889.76
	10	35	200	0.35	92.65	33.63	1262.96	0.33	51.08	20.55	771.60	0.30	57.15	21.56	809.71	0.38	91.30	32.53	1221.71	0.28	75.19	24.10	905.10
	avg	35	200	0.32	90.15	33.06	**1241.56**	0.31	54.61	21.70	**815.08**	0.29	58.33	22.29	**836.97**	0.37	90.65	32.39	**1216.26**	0.29	73.75	23.75	**891.88**
60K	1	35	200	0.35	92.55	34.56	1557.68	0.34	61.64	24.54	1105.70	0.34	61.64	24.54	1105.71	0.44	100.90	36.34	1637.83	0.34	79.86	26.22	1181.72
	2	35	200	0.38	97.67	36.35	1638.02	0.34	61.04	24.41	1100.25	0.34	61.04	24.41	1100.25	0.43	99.78	36.45	1642.58	0.34	78.53	25.89	1166.63
	3	35	200	0.40	100.40	36.45	1642.65	0.40	56.45	22.95	1034.20	0.37	60.23	23.31	1050.44	0.47	108.52	38.55	1737.40	0.38	85.00	27.71	1248.76
	4	35	200	0.43	106.86	38.53	1736.24	0.40	55.92	22.83	1028.68	0.37	59.66	23.15	1043.47	0.47	107.30	38.23	1723.00	0.38	84.20	27.54	1241.00
	5	35	200	0.36	95.10	34.37	1548.88	0.40	50.14	21.55	970.96	0.34	59.77	22.34	1006.91	0.44	101.71	36.01	1622.64	0.35	83.97	26.96	1214.94
	6	35	200	0.40	100.73	36.23	1632.61	0.40	49.66	21.47	967.45	0.34	59.26	22.21	1000.94	0.43	100.46	35.69	1608.25	0.34	82.41	26.55	1196.71
	7	35	200	0.36	94.94	35.17	1585.02	0.34	61.64	24.57	1107.13	0.34	61.64	24.57	1107.15	0.44	100.90	36.37	1639.14	0.34	79.94	26.27	1183.93
	8	35	200	0.40	100.37	37.03	1668.64	0.34	61.04	24.45	1101.68	0.34	61.04	24.45	1101.68	0.43	99.78	36.09	1626.56	0.34	78.53	25.91	1167.61
	9	35	200	0.38	97.71	35.36	1593.44	0.40	53.30	21.93	988.07	0.36	59.86	22.82	1028.57	0.46	104.96	37.29	1680.60	0.36	82.61	26.81	1208.41
	10	35	200	0.42	103.70	37.30	1681.06	0.40	52.79	21.82	983.12	0.36	59.30	22.67	1021.66	0.45	103.76	36.99	1667.00	0.36	81.82	26.65	1201.08
	avg	35	200	0.39	99.00	36.13	**1628.42**	0.38	56.36	23.05	**1038.72**	0.35	60.34	23.45	**1056.68**	0.45	102.81	36.80	**1658.50**	0.35	81.69	26.65	**1201.08**
70K	1	35	200	0.40	101.53	37.85	1989.86	0.39	63.55	25.90	1361.69	0.39	63.56	25.90	1361.79	0.51	115.78	41.78	2196.47	0.42	89.78	29.91	1572.70
	2	35	200	0.44	108.84	40.32	2120.03	0.39	62.89	25.75	1353.93	0.39	62.89	25.75	1353.93	0.50	114.03	41.28	2170.59	0.41	88.65	29.63	1557.86
	3	35	200	0.46	112.83	40.78	2144.04	0.47	58.45	24.42	1284.19	0.44	62.51	24.58	1292.26	0.55	127.79	45.40	2387.07	0.46	97.50	32.23	1694.30
	4	35	200	0.51	122.54	43.84	2304.96	0.47	57.86	24.28	1276.59	0.44	61.87	24.39	1282.51	0.55	125.78	44.82	2356.26	0.45	96.24	31.90	1676.99
	5	35	200	0.42	105.14	37.85	1990.12	0.47	51.91	24.74	1300.96	0.40	62.31	23.65	1243.59	0.51	116.98	41.48	2180.78	0.42	90.58	29.56	1554.35
	6	35	200	0.47	113.33	40.48	2128.31	0.47	51.38	24.76	1301.73	0.39	61.73	23.49	1235.13	0.50	115.06	40.93	2152.07	0.41	89.33	29.24	1537.51
	7	35	200	0.42	104.91	38.76	2037.67	0.39	63.55	25.95	1364.43	0.39	63.55	25.95	1364.37	0.51	115.78	41.82	2198.57	0.42	89.78	29.94	1574.28
	8	35	200	0.46	112.80	41.38	2175.56	0.39	62.89	25.80	1356.68	0.39	62.89	25.80	1356.67	0.50	114.03	41.33	2172.74	0.41	88.65	29.66	1559.39
	9	35	200	0.44	108.88	39.23	2062.59	0.47	55.18	23.76	1249.22	0.42	62.25	24.10	1267.20	0.53	122.09	43.41	2282.59	0.44	93.87	30.91	1624.90
	10	35	200	0.49	117.76	42.04	2210.26	0.47	54.62	23.64	1242.91	0.41	61.62	23.92	1257.57	0.53	120.18	42.87	2254.03	0.43	92.66	30.60	1608.90
	avg	35	200	0.45	110.86	40.25	**2116.34**	0.44	58.23	24.90	**1309.23**	0.41	62.52	24.75	**1301.50**	0.52	118.75	42.51	**2235.12**	0.43	91.70	30.36	**1596.12**
80K	1	35	200	0.46	112.43	41.84	2513.80	0.45	65.63	27.49	1651.55	0.45	65.63	27.49	1651.55	0.58	135.80	49.10	2950.40	0.49	102.50	34.64	2081.67
	2	35	200	0.51	122.91	45.36	2725.71	0.45	64.85	27.29	1639.77	0.45	64.85	27.29	1639.77	0.57	133.03	48.25	2899.48	0.48	100.81	34.16	2052.79
	3	35	200	0.53	128.78	46.34	2784.51	0.54	60.59	26.21	1574.78	0.50	64.92	26.03	1563.82	0.63	155.38	55.22	3318.17	0.54	114.31	38.30	2301.48
	4	35	200	0.58	143.64	51.06	3068.22	0.53	59.93	26.04	1564.62	0.50	64.25	25.83	1551.90	0.62	151.96	54.15	3253.66	0.53	112.32	37.72	2266.23
	5	35	200	0.49	117.55	42.16	2533.37	0.54	53.81	61.38	3688.21	0.45	65.64	25.46	1529.64	0.59	137.66	48.90	2938.08	0.49	103.68	34.34	2063.24
	6	35	200	0.53	129.53	46.00	2764.25	0.53	53.23	64.14	3853.88	0.45	65.28	25.40	1526.16	0.58	134.64	47.98	2882.80	0.48	101.80	33.80	2031.20
	7	35	200	0.48	117.21	43.71	2626.58	0.45	65.59	27.56	1656.00	0.45	65.59	27.56	1656.07	0.58	135.80	49.16	2953.79	0.49	102.51	34.68	2084.14
	8	35	200	0.53	128.73	47.00	2824.25	0.45	64.85	27.39	1645.68	0.45	64.85	27.39	1645.68	0.57	133.03	48.31	2902.89	0.48	100.81	34.20	2055.18
	9	35	200	0.51	122.95	44.11	2650.34	0.54	57.20	26.34	1582.93	0.48	64.84	25.59	1537.81	0.61	145.92	51.94	3120.97	0.52	108.68	36.29	2180.56
	10	35	200	0.56	136.22	48.33	2903.77	0.53	56.58	26.24	1576.86	0.47	64.13	25.38	1524.81	0.60	142.78	50.97	3062.91	0.51	106.80	35.75	2148.29
	avg	35	200	0.52	125.99	45.59	**2739.48**	0.50	60.23	34.01	**2043.43**	0.46	65.00	26.34	**1582.72**	0.59	140.60	50.40	**3028.31**	0.50	105.42	35.39	**2126.48**
90K	1	35	200	0.52	125.96	46.79	3163.01	0.51	67.76	29.36	1984.84	0.51	67.76	29.36	1984.76	0.65	164.19	59.50	4022.38	0.56	119.43	40.94	2767.82
	2	35	200	0.57	141.15	51.94	3510.77	0.50	66.94	29.15	1970.82	0.50	66.94	29.15	1970.82	0.64	159.62	58.02	3922.40	0.55	116.82	40.14	2713.28
	3	35	200	0.60	149.98	53.74	3632.84	0.60	62.90	28.66	1937.63	0.56	67.58	27.87	1884.12	0.71	198.13	70.45	4762.46	0.62	138.13	46.93	3172.48
	4	35	200	0.65	173.49	61.31	4144.29	0.60	62.16	28.48	1925.39	0.56	66.82	27.64	1868.76	0.70	191.90	68.40	4623.84	0.61	134.84	45.88	3101.47
	5	35	200	0.55	133.28	47.63	3219.68	0.60	55.86	170.85	11548.97	0.51	71.41	29.13	1968.83	0.66	167.21	59.52	4023.40	0.57	121.19	40.73	2753.46
	6	35	200	0.60	151.12	53.39	3609.29	0.60	55.21	175.59	11869.99	0.51	70.94	29.11	1968.04	0.65	162.24	57.92	3915.37	0.56	118.33	39.85	2693.93
	7	35	200	0.54	132.79	48.80	3298.87	0.51	67.76	29.60	2001.18	0.51	67.80	29.63	2002.85	0.65	164.19	59.58	4027.78	0.56	119.43	41.00	2771.44
	8	35	200	0.60	149.90	54.51	3685.10	0.50	66.94	29.40	1987.65	0.50	66.94	29.40	1987.69	0.64	159.62	58.10	3927.77	0.55	116.82	40.19	2716.96
	9	35	200	0.57	141.18	50.44	3409.45	0.60	59.38	39.54	2672.79	0.54	67.61	27.49	1858.12	0.69	181.30	64.62	4367.93	0.59	129.04	43.70	2954.36
	10	35	200	0.63	161.54	56.97	3851.20	0.60	58.69	40.39	2730.23	0.53	66.85	27.27	1843.12	0.68	175.83	62.84	4247.98	0.58	126.05	42.77	2891.32
	avg	35	200	0.58	146.04	52.55	**3552.45**	0.56	62.36	60.10	**4062.95**	0.52	68.07	28.61	**1933.71**	0.67	172.42	61.90	**4184.13**	0.58	124.01	42.21	**2853.65**

4-Hour Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	run	speed	r-to-r dist	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K
40K	1	35	300	0.32	101.98	38.23	803.18	0.32	63.55	23.85	501.00	0.32	63.55	23.85	501.00	0.41	95.99	34.55	725.87	0.32	75.83	24.10	506.33
	2	35	300	0.36	107.02	39.97	839.72	0.31	63.40	23.87	501.47	0.31	63.40	23.87	501.47	0.40	94.97	34.30	720.68	0.31	74.92	23.78	499.59
	3	35	300	0.37	109.69	40.01	840.65	0.37	68.10	25.73	540.57	0.35	66.81	24.77	520.45	0.44	102.27	36.33	763.31	0.35	80.55	25.47	535.07
	4	35	300	0.41	115.96	42.00	882.48	0.37	67.51	25.58	537.42	0.35	66.78	24.82	521.48	0.44	101.26	36.09	758.15	0.32	87.96	27.60	579.96
	5	35	300	0.34	104.50	38.27	804.05	0.37	57.75	22.42	471.06	0.32	63.64	22.77	478.38	0.41	96.59	34.18	718.01	0.32	85.71	26.68	560.53
	6	35	300	0.37	110.02	39.75	835.07	0.37	57.23	22.29	468.34	0.31	63.49	22.76	478.28	0.40	95.54	33.92	712.67	0.30	86.47	26.78	562.55
	7	35	300	0.34	104.34	38.81	815.42	0.32	63.47	23.83	500.65	0.32	63.49	23.84	500.85	0.41	95.90	34.55	725.79	0.32	75.84	24.12	506.72
	8	35	300	0.37	104.34	38.66	812.17	0.31	63.47	23.91	502.39	0.31	63.49	23.92	502.58	0.40	95.90	34.67	728.32	0.31	75.84	24.22	508.75
	9	35	300	0.36	107.01	38.91	817.48	0.37	62.95	24.02	504.71	0.33	65.15	23.80	499.98	0.43	99.30	35.27	741.08	0.33	82.90	26.03	546.97
	10	35	300	0.39	112.91	40.80	857.29	0.37	62.37	23.87	501.58	0.33	65.09	23.84	500.78	0.42	98.32	35.04	736.22	0.30	85.81	26.77	562.40
	avg	35	300	0.36	107.78	39.54	**830.75**	0.35	62.98	23.94	**502.92**	0.32	64.49	23.82	**500.52**	0.41	97.60	34.89	**733.01**	0.32	81.18	25.55	**536.89**
50K	1	35	300	0.40	115.74	43.26	1136.04	0.39	71.72	27.50	722.10	0.39	71.72	27.50	722.09	0.51	115.66	41.73	1096.01	0.39	85.86	27.85	731.37
	2	35	300	0.44	124.14	46.07	1210.01	0.39	71.57	27.52	722.86	0.39	71.57	27.52	722.86	0.50	113.92	41.24	1083.11	0.39	85.62	27.84	731.11
	3	35	300	0.46	128.68	46.62	1224.46	0.47	71.50	27.83	730.98	0.44	76.43	29.09	764.01	0.55	127.63	45.35	1190.87	0.44	92.66	29.85	783.83
	4	35	300	0.51	139.74	50.10	1315.68	0.47	70.78	27.62	725.47	0.44	75.69	28.87	758.08	0.55	125.63	44.76	1175.53	0.43	92.45	29.84	783.63
	5	35	300	0.42	139.74	50.45	1324.85	0.47	70.78	30.37	797.64	0.40	75.69	27.95	734.13	0.51	125.63	44.54	1169.82	0.40	92.45	29.11	764.49
	6	35	300	0.47	129.32	46.29	1215.76	0.47	60.00	24.44	641.75	0.39	71.72	26.23	688.96	0.50	114.95	40.89	1073.89	0.39	93.93	29.83	783.38
	7	35	300	0.42	119.65	44.32	1163.87	0.39	71.72	27.52	722.69	0.39	71.72	27.52	722.68	0.51	115.66	41.77	1097.05	0.39	85.92	27.90	732.60
	8	35	300	0.46	128.64	47.28	1241.59	0.39	71.57	27.55	723.46	0.39	71.57	27.55	723.46	0.50	113.92	41.28	1084.19	0.39	85.62	27.87	731.83
	9	35	300	0.44	124.18	44.86	1178.08	0.47	66.06	26.07	684.67	0.42	74.42	27.75	728.68	0.53	121.95	43.36	1138.85	0.42	90.10	28.86	757.91
	10	35	300	0.49	134.29	48.04	1261.71	0.47	65.39	25.88	679.71	0.41	73.77	27.73	728.21	0.52	120.05	42.82	1124.64	0.41	89.05	28.60	751.03
	avg	35	300	0.45	128.41	46.73	**1227.21**	0.44	69.11	27.23	**715.13**	0.41	73.43	27.77	**729.32**	0.52	119.50	42.78	**1123.40**	0.40	89.37	28.75	**755.12**
60K	1	35	300	0.48	133.97	49.93	1573.47	0.47	82.42	32.28	1017.43	0.47	82.42	32.29	1017.47	0.61	145.66	52.71	1661.22	0.49	103.00	34.20	1077.86
	2	35	300	0.53	147.77	54.58	1720.17	0.47	82.16	32.27	1017.10	0.47	82.16	32.27	1017.11	0.60	142.32	51.66	1628.16	0.48	101.21	33.68	1061.56
	3	35	300	0.56	155.62	56.02	1765.38	0.56	75.23	30.25	953.29	0.52	80.74	31.37	988.57	0.66	169.73	60.33	1901.37	0.55	115.59	38.14	1202.09
	4	35	300	0.61	175.81	62.46	1968.55	0.56	74.39	29.98	944.90	0.52	79.83	31.06	978.94	0.66	165.45	58.96	1858.11	0.54	113.45	37.50	1181.94
	5	35	300	0.51	140.67	50.50	1591.53	0.56	63.77	28.15	887.06	0.47	77.40	29.04	915.06	0.61	147.89	52.57	1656.83	0.50	106.50	34.63	1091.20
	6	35	300	0.56	156.62	55.63	1753.04	0.56	63.06	28.00	882.49	0.47	76.53	28.75	905.97	0.60	144.25	51.44	1621.04	0.49	103.95	33.87	1067.42
	7	35	300	0.51	140.23	51.73	1630.13	0.47	82.42	32.32	1018.51	0.47	82.42	32.32	1018.48	0.61	145.66	52.78	1663.28	0.49	103.11	34.28	1080.35
	8	35	300	0.56	155.54	56.82	1790.50	0.47	82.16	32.31	1018.16	0.47	82.16	32.31	1018.15	0.60	142.32	51.73	1630.19	0.48	101.20	33.73	1062.85
	9	35	300	0.53	147.80	53.05	1671.97	0.56	69.50	28.56	900.19	0.50	78.93	30.20	951.77	0.64	157.99	56.26	1773.12	0.52	109.57	35.98	1133.87
	10	35	300	0.58	165.66	58.75	1851.43	0.56	68.72	28.33	892.83	0.50	78.04	29.90	942.41	0.63	154.12	55.05	1734.74	0.51	107.57	35.40	1115.56
	avg	35	300	0.54	151.97	54.95	**1731.62**	0.53	74.38	30.25	**953.20**	0.49	80.06	30.95	**975.39**	0.62	151.54	54.35	**1712.80**	0.50	106.51	35.14	**1107.47**
70K	1	35	300	0.57	158.89	59.06	2171.54	0.55	87.54	35.99	1323.27	0.55	87.54	35.99	1323.23	0.71	190.99	70.70	2599.51	0.59	128.63	43.79	1609.97
	2	35	300	0.62	182.52	67.11	2467.54	0.55	86.41	35.65	1310.60	0.55	86.41	35.65	1310.59	0.70	189.58	69.04	2538.43	0.58	125.33	42.73	1571.10
	3	35	300	0.65	193.12	70.37	2587.39	0.65	79.36	33.56	1233.96	0.61	85.52	34.05	1251.94	0.78	202.88	76.02	2795.13	0.66	152.88	51.73	1901.82
	4	35	300	0.71	214.63	79.96	2940.02	0.65	78.38	33.24	1222.07	0.61	84.44	33.67	1237.85	0.76	201.76	74.94	2755.20	0.65	148.48	50.27	1848.46
	5	35	300	0.59	170.10	60.74	2233.27	0.65	67.27	76.85	2825.51	0.55	82.61	31.77	1168.00	0.72	187.17	69.10	2540.72	0.60	130.83	43.65	1604.98
	6	35	300	0.65	198.05	70.90	2606.89	0.65	66.43	78.89	2900.62	0.55	81.57	31.40	1154.51	0.71	183.65	66.68	2451.72	0.59	127.23	42.50	1562.78
	7	35	300	0.59	169.35	62.23	2287.95	0.55	87.54	36.11	1327.73	0.55	87.54	36.11	1327.73	0.71	190.99	70.81	2603.30	0.59	128.63	43.85	1612.35
	8	35	300	0.65	196.68	71.42	2625.81	0.55	86.41	35.77	1315.11	0.55	86.41	35.77	1315.11	0.70	189.58	69.15	2542.32	0.58	125.33	42.80	1573.47
	9	35	300	0.62	182.53	65.12	2394.17	0.65	73.32	32.37	1190.27	0.58	83.89	32.88	1208.86	0.75	194.87	72.49	2665.41	0.63	140.93	47.46	1744.92
	10	35	300	0.68	206.21	75.61	2780.13	0.65	72.41	32.11	1180.71	0.58	82.82	32.50	1195.05	0.73	193.08	70.27	2583.58	0.62	137.03	46.20	1698.72
	avg	35	300	0.63	187.21	68.25	**2509.47**	0.61	78.51	43.05	**1582.99**	0.57	84.88	33.98	**1249.29**	0.73	192.45	70.92	**2607.53**	0.61	134.53	45.50	**1672.86**
80K	1	35	300	0.65	180.36	68.78	2890.16	0.63	91.91	39.53	1661.13	0.63	91.91	39.53	1661.15	0.81	206.01	88.28	3709.53	0.69	171.38	59.84	2514.46
	2	35	300	0.71	205.93	79.16	3326.31	0.63	90.60	39.10	1642.97	0.63	90.60	39.10	1642.97	0.80	207.36	82.96	3486.08	0.68	164.54	57.48	2415.40
	3	35	300	0.74	206.52	81.43	3421.82	0.75	83.97	38.17	1603.88	0.70	90.85	37.63	1581.19	0.89	209.38	105.24	4422.20	0.77	181.84	63.75	2678.80
	4	35	300	0.81	222.54	106.28	4465.72	0.75	82.82	37.78	1587.38	0.70	89.62	37.18	1562.35	0.87	208.06	88.87	3734.42	0.76	176.27	61.77	2595.44
	5	35	300	0.68	185.46	70.15	2947.71	0.75	71.18	219.28	9214.07	0.63	88.59	35.77	1503.19	0.82	191.71	77.71	3265.38	0.70	163.06	56.59	2377.98
	6	35	300	0.75	203.60	80.44	3380.04	0.75	70.20	222.99	9369.73	0.63	87.56	35.45	1489.43	0.81	190.18	74.77	3141.91	0.69	157.94	54.79	2302.42
	7	35	300	0.68	193.79	74.41	3126.76	0.63	91.91	39.97	1679.61	0.63	91.91	39.97	1679.71	0.81	206.00	88.42	3715.19	0.69	171.21	59.88	2516.30
	8	35	300	0.74	217.64	85.39	3587.91	0.63	90.60	39.54	1661.60	0.63	90.60	39.54	1661.60	0.80	207.36	83.10	3491.97	0.68	164.54	57.59	2419.72
	9	35	300	0.71	195.59	75.04	3153.26	0.75	77.57	72.80	3058.98	0.67	89.51	36.57	1536.68	0.85	200.35	87.58	3679.86	0.73	172.10	60.18	2528.86
	10	35	300	0.78	212.82	89.49	3760.33	0.75	76.51	73.68	3095.99	0.66	88.25	36.10	1516.97	0.84	198.91	79.97	3360.38	0.72	166.79	58.31	2450.19
	avg	35	300	0.72	202.43	81.06	**3406.00**	0.70	82.73	82.28	**3457.54**	0.65	89.94	37.69	**1583.52**	0.83	202.53	85.69	**3600.69**	0.71	168.97	59.02	**2479.96**
90K	1	35	300	0.73	217.63	87.77	4148.85	0.71	96.74	45.76	2163.40	0.71	96.74	45.77	2163.42	0.91	219.49	176.92	8363.26	0.79	204.44	79.28	3747.77
	2	35	300	0.80	222.53	120.28	5685.75	0.71	95.23	45.17	2135.34	0.71	95.23	45.17	2135.34	0.90	216.90	137.57	6503.20	0.78	198.99	73.19	3460.01
	3	35	300	0.84	220.05	127.48	6026.21	0.84	89.15	73.33	3466.34	0.79	96.94	54.91	2595.48	1.00	216.34	194.83	9209.78	0.88	199.29	83.40	3942.24
	4	35	300	0.91	228.13	196.99	9312.17	0.84	87.80	73.12	3456.44	0.78	95.48	54.42	2572.63	0.98	214.78	145.50	6878.18	0.86	192.29	69.98	3308.14
	5	35	300	0.76	197.45	85.44	4038.93	0.84	75.57	383.88	18146.81	0.71	104.31	99.43	4700.06	0.92	196.49	132.69	6272.63	0.80	175.83	64.36	3042.64
	6	35	300	0.84	209.39	129.52	6122.71	0.84	74.42	388.85	18381.87	0.71	102.97	100.63	4757.20	0.91	194.75	106.57	5037.79	0.79	169.65	61.88	2925.06
	7	35	300	0.76	223.04	92.32	4363.92	0.71	96.74	56.88	2688.97	0.71	96.79	57.09	2698.85	0.91	219.48	179.78	8498.31	0.79	204.45	80.10	3786.58
	8	35	300	0.83	229.03	136.76	6464.68	0.71	95.23	56.06	2649.95	0.71	95.22	56.04	2649.12	0.90	216.90	140.65	6648.64	0.78	198.99	73.90	3493.20
	9	35	300	0.80	210.01	104.82	4954.84	0.84	82.36	193.53	9148.34	0.75	95.88	56.88	2688.61	0.96	206.12	157.12	7427.41	0.84	187.05	68.47	3236.58
	10	35	300	0.88	219.86	162.13	7664.01	0.84	81.11	195.34	9234.24	0.75	94.42	56.38	2665.37	0.94	204.48	123.68	5846.41	0.83	180.47	65.89	3114.70
	avg	35	300	0.81	217.71	124.35	**5878.21**	0.79	87.43	151.19	**7147.17**	0.73	97.40	62.67	**2962.61**	0.93	210.57	149.53	**7068.56**	0.82	191.15	72.04	**3405.69**

8-Hour Non-Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	run	speed	r-to-r dist	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K
40K	1	35	300	0.23	90.97	34.15	1025.97	0.23	56.11	20.54	617.07	0.23	56.18	20.57	617.92	0.29	80.34	28.85	866.80	0.26	73.24	22.58	678.53
	2	35	300	0.25	92.49	34.81	1045.83	0.22	56.10	20.61	619.16	0.22	56.10	20.61	619.16	0.29	79.83	28.77	864.39	0.26	73.24	22.53	677.03
	3	35	300	0.27	95.76	35.11	1054.98	0.27	59.36	21.77	653.95	0.25	57.95	20.98	630.40	0.32	83.38	29.63	890.29	0.25	75.75	23.36	701.72
	4	35	300	0.29	97.11	35.68	1071.87	0.27	59.36	21.82	655.65	0.25	57.95	21.04	632.16	0.31	82.93	29.57	888.44	0.25	76.51	23.48	705.32
	5	35	300	0.24	92.30	33.70	1012.61	0.27	54.81	20.44	614.04	0.23	57.16	20.01	601.28	0.29	80.64	28.48	855.66	0.24	76.31	23.47	705.20
	6	35	300	0.27	94.07	34.45	1035.07	0.27	54.36	20.34	611.06	0.22	56.72	19.90	597.74	0.29	80.12	28.40	853.20	0.25	76.51	23.42	703.69
	7	35	300	0.24	92.65	34.54	1037.69	0.23	56.11	20.55	617.36	0.23	56.18	20.57	618.14	0.29	80.27	28.84	866.54	0.26	73.24	22.60	678.88
	8	35	300	0.27	93.89	35.10	1054.48	0.22	56.10	20.62	619.46	0.22	56.10	20.62	619.43	0.29	79.83	28.79	864.84	0.26	73.24	22.55	677.39
	9	35	300	0.25	94.00	34.38	1032.95	0.27	59.36	21.79	654.52	0.24	57.04	20.35	611.52	0.30	81.95	29.09	874.02	0.24	76.29	23.47	705.14
	10	35	300	0.28	95.57	35.04	1052.63	0.27	59.25	21.91	658.22	0.24	57.04	20.41	613.16	0.30	81.50	29.03	872.27	0.25	76.52	23.44	704.17
	avg	35	300	0.26	93.88	34.70	**1042.41**	0.25	57.09	21.04	**632.05**	0.23	56.84	20.51	**616.09**	0.30	81.08	28.95	**869.65**	0.25	75.09	23.09	**693.71**
50K	1	35	300	0.29	97.00	36.41	1367.41	0.28	60.51	22.50	844.84	0.28	60.51	22.50	844.84	0.36	89.41	32.15	1207.55	0.29	73.35	23.05	865.55
	2	35	300	0.32	101.10	37.86	1422.01	0.28	60.47	22.56	847.12	0.28	60.47	22.56	847.12	0.36	88.69	32.01	1202.07	0.29	73.24	22.95	861.82
	3	35	300	0.33	103.22	37.76	1418.17	0.33	65.33	24.32	913.27	0.31	63.21	23.23	872.41	0.40	94.30	33.51	1258.28	0.31	77.59	24.32	913.50
	4	35	300	0.36	108.13	39.36	1478.07	0.33	65.33	24.38	915.48	0.31	63.19	23.28	874.45	0.39	93.55	33.34	1252.23	0.28	81.51	25.33	951.32
	5	35	300	0.30	99.09	36.07	1354.47	0.33	56.62	21.64	812.59	0.28	60.66	21.52	808.07	0.37	89.96	31.80	1194.42	0.28	82.14	25.40	954.01
	6	35	300	0.33	103.49	37.54	1409.93	0.33	56.12	21.52	808.13	0.28	60.54	21.52	808.30	0.36	89.14	31.63	1187.78	0.28	80.50	24.80	931.24
	7	35	300	0.30	98.96	36.88	1384.94	0.28	60.51	22.51	845.34	0.28	60.54	22.52	845.82	0.36	89.41	32.17	1208.31	0.29	73.35	23.06	866.06
	8	35	300	0.33	103.20	38.35	1440.27	0.28	60.47	22.57	847.62	0.28	60.47	22.57	847.64	0.36	88.69	32.03	1202.86	0.29	73.24	22.96	862.39
	9	35	300	0.32	101.14	36.88	1384.99	0.33	61.68	23.22	872.18	0.30	61.87	22.41	841.43	0.38	92.04	32.68	1227.42	0.30	79.68	24.84	932.82
	10	35	300	0.35	105.75	38.40	1442.00	0.33	61.16	23.10	867.70	0.30	61.84	22.45	843.11	0.38	91.30	32.53	1221.71	0.28	80.47	24.89	934.76
	avg	35	300	0.32	102.11	37.55	**1410.23**	0.31	60.82	22.83	**857.43**	0.29	61.33	22.46	**843.32**	0.37	90.65	32.39	**1216.26**	0.29	77.51	24.16	**907.34**
60K	1	35	300	0.35	105.64	39.56	1782.99	0.34	65.66	24.79	1117.26	0.34	65.66	24.79	1117.19	0.44	100.90	36.34	1637.83	0.34	77.70	24.90	1122.13
	2	35	300	0.38	111.48	41.55	1872.61	0.34	65.57	24.84	1119.43	0.34	65.57	24.84	1119.41	0.43	99.78	36.45	1642.58	0.34	77.47	24.80	1117.50
	3	35	300	0.40	114.60	41.72	1880.09	0.40	69.04	26.31	1185.51	0.37	69.52	25.93	1168.51	0.47	108.52	38.55	1737.40	0.37	82.98	26.39	1189.21
	4	35	300	0.43	121.97	44.04	1984.69	0.40	68.43	26.15	1178.27	0.37	69.47	25.97	1170.58	0.47	107.30	38.23	1723.00	0.35	88.89	28.13	1267.69
	5	35	300	0.36	108.55	39.35	1773.15	0.40	58.56	23.00	1036.34	0.34	65.84	23.70	1067.97	0.44	101.71	36.01	1622.64	0.34	88.04	27.54	1241.05
	6	35	300	0.40	114.99	41.42	1866.72	0.40	58.00	22.86	1030.13	0.34	65.68	23.69	1067.40	0.43	100.46	35.69	1608.25	0.32	88.97	27.73	1249.77
	7	35	300	0.36	108.36	40.26	1814.20	0.34	65.66	24.81	1118.02	0.34	65.68	24.82	1118.32	0.44	100.90	36.37	1639.14	0.34	77.70	24.92	1123.01
	8	35	300	0.40	114.57	42.33	1907.57	0.34	65.57	24.86	1120.21	0.34	65.57	24.86	1120.20	0.43	99.78	36.09	1626.56	0.34	77.47	24.82	1118.43
	9	35	300	0.38	111.52	40.47	1823.94	0.40	63.82	24.58	1107.84	0.36	67.59	24.84	1119.25	0.46	104.96	37.29	1680.60	0.36	85.03	26.85	1209.93
	10	35	300	0.42	118.38	42.65	1921.89	0.40	63.21	24.42	1100.62	0.36	67.52	24.87	1120.77	0.45	103.76	36.99	1667.00	0.33	86.59	27.25	1227.84
	avg	35	300	0.39	113.01	41.33	**1862.79**	0.38	64.35	24.66	**1111.36**	0.35	66.81	24.83	**1118.96**	0.45	102.81	36.80	**1658.50**	0.34	83.08	26.33	**1186.65**
70K	1	35	300	0.40	115.88	43.31	2277.03	0.39	71.76	27.52	1446.71	0.39	71.76	27.52	1446.70	0.51	115.78	41.78	2196.47	0.39	85.98	27.89	1466.38
	2	35	300	0.44	124.24	46.11	2424.32	0.39	71.62	27.55	1448.24	0.39	71.62	27.55	1448.24	0.50	114.03	41.28	2170.59	0.39	85.68	27.86	1464.86
	3	35	300	0.46	128.79	46.66	2453.40	0.47	71.52	27.85	1464.03	0.44	76.45	29.10	1530.12	0.55	127.79	45.40	2387.07	0.44	92.73	29.87	1570.60
	4	35	300	0.51	139.89	50.15	2636.58	0.47	70.80	27.64	1452.96	0.44	75.71	28.88	1518.22	0.55	125.78	44.82	2356.26	0.43	92.48	29.85	1569.54
	5	35	300	0.42	120.01	43.32	2277.71	0.47	60.63	24.61	1293.79	0.40	72.00	26.29	1382.31	0.51	116.98	41.48	2180.78	0.40	95.30	30.20	1588.06
	6	35	300	0.47	129.37	46.31	2434.70	0.47	60.01	24.45	1285.49	0.39	71.76	26.25	1380.33	0.50	115.06	40.93	2152.07	0.39	94.02	29.87	1570.22
	7	35	300	0.42	119.74	44.35	2331.73	0.39	71.76	27.54	1447.92	0.39	71.76	27.54	1447.91	0.51	115.78	41.82	2198.57	0.39	85.98	27.92	1467.84
	8	35	300	0.46	128.75	47.32	2487.74	0.39	71.62	27.57	1449.46	0.39	71.62	27.57	1449.45	0.50	114.03	41.33	2172.74	0.39	85.68	27.89	1466.31
	9	35	300	0.44	124.28	44.89	2360.32	0.47	66.07	26.08	1371.30	0.42	74.48	27.77	1460.00	0.53	122.09	43.41	2282.59	0.42	90.13	28.87	1518.09
	10	35	300	0.49	134.42	48.09	2528.31	0.47	65.40	25.89	1361.36	0.41	73.79	27.74	1458.43	0.53	120.18	42.87	2254.03	0.41	89.07	28.61	1504.20
	avg	35	300	0.45	126.54	46.05	**2421.18**	0.44	68.12	26.67	**1402.13**	0.41	73.09	27.62	**1452.17**	0.52	118.75	42.51	**2235.12**	0.40	89.71	28.88	**1518.61**
80K	1	35	300	0.46	128.33	47.86	2876.02	0.45	79.12	30.80	1850.97	0.45	79.11	30.80	1850.95	0.58	135.92	49.14	2952.96	0.46	97.66	32.20	1935.09
	2	35	300	0.51	140.29	51.89	3117.91	0.45	78.89	30.81	1851.20	0.45	78.89	30.81	1851.19	0.57	133.03	48.25	2899.42	0.45	96.03	31.75	1907.54
	3	35	300	0.53	147.00	53.01	3185.37	0.54	74.15	29.53	1774.54	0.50	79.45	30.68	1843.59	0.63	155.38	55.22	3318.16	0.52	108.20	35.46	2130.58
	4	35	300	0.58	163.96	58.40	3509.14	0.53	73.34	29.28	1759.60	0.50	78.62	30.41	1827.47	0.62	151.96	54.15	3253.66	0.51	106.42	34.95	2099.94
	5	35	300	0.49	134.17	48.24	2898.71	0.54	62.85	26.81	1611.02	0.45	76.05	28.37	1704.54	0.59	137.66	48.90	2938.08	0.47	103.40	33.37	2005.37
	6	35	300	0.53	147.85	52.63	3162.25	0.53	62.17	26.65	1601.15	0.45	75.23	28.10	1688.41	0.58	134.64	47.98	2882.80	0.46	101.03	32.69	1963.98
	7	35	300	0.48	133.78	49.41	2968.66	0.45	79.11	30.83	1852.70	0.45	79.12	30.83	1852.80	0.58	135.80	49.16	2953.79	0.46	97.66	32.24	1937.24
	8	35	300	0.53	146.94	53.76	3230.57	0.45	78.89	30.84	1853.00	0.45	78.89	30.84	1853.01	0.57	133.03	48.31	2902.89	0.45	96.03	31.78	1909.75
	9	35	300	0.51	140.33	50.46	3031.98	0.54	68.50	27.80	1670.53	0.48	77.65	29.54	1774.94	0.61	145.92	51.94	3120.97	0.49	103.14	33.63	2020.90
	10	35	300	0.56	155.49	55.28	3321.39	0.53	67.75	27.58	1657.38	0.47	76.80	29.26	1758.27	0.60	142.78	50.97	3062.91	0.48	101.46	33.16	1992.66
	avg	35	300	0.52	143.81	52.09	**3130.20**	0.50	72.48	29.09	**1748.21**	0.46	77.98	29.96	**1800.52**	0.59	140.61	50.40	**3028.57**	0.48	101.10	33.12	**1990.30**
90K	1	35	300	0.52	143.78	53.52	3617.98	0.51	85.25	34.34	2321.48	0.51	85.26	34.34	2321.50	0.65	164.19	59.50	4022.38	0.53	112.78	37.86	2559.02
	2	35	300	0.57	161.11	59.39	4014.91	0.50	84.21	34.04	2301.01	0.50	84.21	34.04	2301.01	0.64	159.62	58.02	3922.40	0.52	110.45	37.15	2511.33
	3	35	300	0.60	171.20	61.46	4154.64	0.60	76.97	31.73	2145.08	0.56	82.75	32.47	2194.89	0.71	198.13	70.45	4762.46	0.59	129.31	43.13	2915.82
	4	35	300	0.65	198.04	70.10	4738.34	0.60	76.07	31.44	2125.52	0.56	81.77	32.13	2172.12	0.70	191.90	68.40	4623.84	0.58	126.43	42.23	2854.63
	5	35	300	0.55	152.12	54.48	3682.93	0.60	65.24	32.88	2222.69	0.51	79.58	30.15	2037.98	0.66	167.21	59.52	4023.40	0.54	114.35	37.60	2541.74
	6	35	300	0.60	172.51	61.06	4127.88	0.60	64.48	32.98	2229.40	0.51	78.65	29.83	2016.41	0.65	162.24	57.92	3915.37	0.53	111.81	36.83	2489.76
	7	35	300	0.54	151.56	55.81	3772.95	0.51	85.25	34.42	2326.60	0.51	85.25	34.42	2326.70	0.65	164.19	59.58	4027.78	0.53	112.78	37.91	2562.42
	8	35	300	0.60	171.11	62.34	4214.10	0.50	84.21	34.12	2306.32	0.50	84.21	34.12	2306.32	0.64	159.62	58.10	3927.77	0.52	110.45	37.20	2514.73
	9	35	300	0.57	161.14	57.69	3899.47	0.60	71.11	29.92	2022.28	0.54	81.01	31.30	2115.76	0.69	181.30	64.62	4367.93	0.57	121.31	40.27	2722.40
	10	35	300	0.63	184.40	65.15	4403.87	0.60	70.28	29.67	2005.43	0.53	80.05	30.97	2093.42	0.68	175.83	62.84	4247.98	0.56	118.67	39.46	2667.70
	avg	35	300	0.58	166.70	60.10	**4062.71**	0.56	76.31	32.55	**2200.58**	0.52	82.28	32.38	**2188.61**	0.67	172.42	61.90	**4184.13**	0.55	116.83	38.96	**2633.95**

4-Hour Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	run	speed	r-to-r dist	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K
40K	1	35	400	0.32	114.61	43.06	904.68	0.32	63.47	23.02	483.63	0.32	63.55	23.05	484.28	0.41	95.98	34.55	725.86	0.34	84.90	26.10	548.30
	2	35	400	0.36	120.29	44.98	945.00	0.31	63.40	23.06	484.48	0.31	63.40	23.06	484.48	0.40	94.97	34.30	720.68	0.34	84.90	26.01	546.54
	3	35	400	0.37	123.29	45.07	946.94	0.37	69.48	25.34	532.35	0.35	66.81	24.01	504.51	0.44	102.27	36.33	763.31	0.35	87.78	27.01	567.37
	4	35	400	0.41	130.33	47.26	993.00	0.37	69.48	25.39	533.35	0.35	66.78	24.05	505.28	0.44	101.26	36.09	758.15	0.32	90.60	27.67	581.26
	5	35	400	0.34	117.45	42.75	898.06	0.37	66.06	24.45	513.74	0.32	65.56	22.76	478.14	0.41	96.59	34.18	718.01	0.32	90.60	27.68	581.54
	6	35	400	0.37	123.66	45.15	948.55	0.37	65.45	24.28	510.18	0.31	64.99	22.58	474.36	0.40	95.54	33.92	712.67	0.32	90.60	27.52	578.21
	7	35	400	0.34	117.27	43.72	918.50	0.32	63.55	23.07	484.60	0.32	63.55	23.07	484.60	0.41	95.90	34.55	725.79	0.34	84.90	26.12	548.70
	8	35	400	0.37	123.26	45.70	960.20	0.31	63.40	23.08	484.80	0.31	63.40	23.08	484.80	0.40	94.97	34.33	721.20	0.34	84.90	26.03	546.95
	9	35	400	0.36	120.33	43.85	921.30	0.37	69.48	25.37	533.03	0.33	65.15	23.04	484.03	0.43	99.30	35.27	741.08	0.33	89.12	27.33	574.28
	10	35	400	0.39	126.91	45.92	964.78	0.37	69.48	25.42	534.05	0.33	65.09	23.06	484.58	0.42	98.32	35.04	736.22	0.32	90.59	27.57	579.26
	avg	35	400	0.36	121.74	44.75	**940.10**	0.35	66.32	24.25	**509.42**	0.32	64.83	23.18	**486.91**	0.41	97.51	34.86	**732.30**	0.33	87.89	26.90	**565.24**
50K	1	35	400	0.40	130.09	48.72	1279.40	0.39	71.72	26.70	701.20	0.39	71.72	26.70	701.19	0.51	115.66	41.73	1096.01	0.39	87.75	27.72	727.92
	2	35	400	0.44	139.53	51.88	1362.48	0.39	71.57	26.72	701.62	0.39	71.57	26.72	701.62	0.50	113.92	41.24	1083.11	0.39	86.94	27.43	720.35
	3	35	400	0.46	144.63	52.50	1378.81	0.47	81.69	30.58	802.98	0.44	77.17	28.45	747.12	0.55	127.63	45.35	1190.87	0.44	93.83	29.56	776.39
	4	35	400	0.51	157.06	56.40	1481.31	0.47	81.65	30.61	803.89	0.44	77.08	28.46	747.55	0.55	125.63	44.76	1175.53	0.40	101.94	31.96	839.37
	5	35	400	0.42	134.78	48.76	1280.46	0.47	69.33	26.64	699.65	0.40	71.95	25.51	670.01	0.51	116.86	41.43	1088.14	0.40	100.13	31.12	817.32
	6	35	400	0.47	145.27	52.10	1368.36	0.47	68.62	26.43	694.17	0.39	71.72	25.46	668.71	0.50	114.94	40.89	1073.85	0.37	101.38	31.34	823.15
	7	35	400	0.42	134.47	49.90	1310.61	0.39	71.71	26.72	701.75	0.39	71.72	26.72	701.78	0.51	115.66	41.77	1097.04	0.39	87.75	27.75	728.65
	8	35	400	0.46	144.59	53.23	1398.05	0.39	71.57	26.74	702.27	0.39	71.57	26.74	702.22	0.50	113.92	41.28	1084.19	0.39	86.94	27.46	721.07
	9	35	400	0.44	139.56	50.52	1326.64	0.47	76.92	29.11	764.51	0.42	74.42	26.99	708.76	0.53	121.95	43.36	1138.85	0.42	96.71	30.29	795.43
	10	35	400	0.49	150.94	54.10	1420.73	0.47	76.17	28.89	758.68	0.41	74.30	26.99	708.77	0.52	120.05	42.82	1124.64	0.38	99.83	31.09	816.55
	avg	35	400	0.45	142.09	51.81	**1360.69**	0.44	74.10	27.91	**733.07**	0.41	73.32	26.87	**705.77**	0.52	118.62	42.47	**1115.22**	0.40	94.32	29.57	**776.62**
60K	1	35	400	0.48	150.56	56.21	1771.49	0.47	82.42	31.49	992.40	0.47	82.42	31.49	992.37	0.61	145.66	52.71	1661.22	0.47	98.22	31.85	1003.68
	2	35	400	0.53	166.09	61.45	1936.46	0.47	82.16	31.47	991.63	0.47	82.16	31.47	991.62	0.60	142.32	51.66	1628.16	0.47	97.81	31.76	1000.94
	3	35	400	0.56	174.90	63.06	1987.32	0.56	88.94	34.46	1086.12	0.52	91.33	34.52	1087.93	0.66	169.73	60.33	1901.37	0.52	107.79	34.76	1095.54
	4	35	400	0.61	197.60	70.30	2215.63	0.56	87.99	34.15	1076.35	0.52	91.13	34.50	1087.13	0.66	165.45	58.96	1858.11	0.51	108.81	35.13	1107.02
	5	35	400	0.51	158.09	56.86	1791.91	0.56	72.93	29.37	925.57	0.47	82.76	30.08	947.88	0.61	147.89	52.57	1656.83	0.47	110.62	35.13	1107.01
	6	35	400	0.56	176.03	62.62	1973.56	0.56	72.12	29.12	917.65	0.47	82.39	29.97	944.64	0.60	144.25	51.44	1621.04	0.46	109.49	34.80	1096.69
	7	35	400	0.51	157.60	58.23	1835.24	0.47	82.42	31.52	993.37	0.47	82.42	31.52	993.39	0.61	145.66	52.78	1663.28	0.47	98.22	31.89	1004.91
	8	35	400	0.56	174.83	63.96	2015.54	0.47	82.16	31.50	992.67	0.47	82.16	31.50	992.67	0.60	142.32	51.73	1630.19	0.47	97.81	31.80	1002.20
	9	35	400	0.53	166.11	59.73	1882.33	0.56	80.96	31.74	1000.19	0.50	86.78	32.26	1016.61	0.64	157.99	56.26	1773.12	0.50	104.61	33.52	1056.46
	10	35	400	0.58	186.19	66.13	2084.03	0.56	80.05	31.44	990.88	0.50	86.54	32.21	1015.19	0.63	154.12	55.05	1734.74	0.49	103.92	33.35	1050.88
	avg	35	400	0.54	170.80	61.86	**1949.35**	0.53	81.22	31.63	**996.68**	0.49	85.01	31.95	**1006.94**	0.62	151.54	54.35	**1712.80**	0.48	103.73	33.40	**1052.53**
70K	1	35	400	0.57	178.57	66.48	2444.20	0.55	96.89	37.97	1396.22	0.55	96.89	37.97	1396.19	0.71	190.99	70.70	2599.51	0.57	120.95	40.32	1482.27
	2	35	400	0.62	201.61	75.36	2770.65	0.55	96.43	37.88	1392.57	0.55	96.43	37.88	1392.57	0.70	189.58	69.04	2538.43	0.55	118.03	39.39	1448.42
	3	35	400	0.65	202.28	75.94	2792.24	0.65	93.87	37.63	1383.52	0.61	101.11	39.34	1446.29	0.78	202.88	76.02	2795.13	0.63	142.14	47.24	1736.75
	4	35	400	0.71	221.94	84.57	3109.45	0.65	92.70	37.21	1368.27	0.61	99.88	38.89	1429.93	0.76	201.76	74.94	2755.20	0.62	138.35	46.00	1691.24
	5	35	400	0.59	185.40	67.51	2481.97	0.65	76.94	34.36	1263.18	0.55	94.50	35.46	1303.70	0.72	187.17	69.10	2540.72	0.57	125.03	40.83	1501.19
	6	35	400	0.65	204.90	75.82	2787.61	0.65	75.99	34.13	1254.75	0.55	93.31	35.03	1287.79	0.71	183.65	66.68	2451.72	0.56	121.56	39.74	1461.04
	7	35	400	0.59	187.93	70.10	2577.33	0.55	96.89	38.02	1397.91	0.55	96.89	38.02	1397.93	0.71	190.99	70.81	2603.30	0.57	121.06	40.42	1486.05
	8	35	400	0.65	211.25	78.48	2885.65	0.55	96.43	37.92	1394.33	0.55	96.43	37.92	1394.33	0.70	189.58	69.15	2542.32	0.55	118.03	39.45	1450.64
	9	35	400	0.62	194.10	71.05	2612.46	0.65	85.40	34.93	1284.25	0.58	97.67	37.40	1375.24	0.75	194.87	72.49	2665.41	0.60	131.75	43.50	1599.48
	10	35	400	0.68	213.23	79.73	2931.38	0.65	84.34	34.56	1270.82	0.58	96.49	36.98	1359.48	0.73	193.08	70.27	2583.58	0.59	128.35	42.42	1559.70
	avg	35	400	0.63	200.12	74.50	**2739.29**	0.61	89.59	36.46	**1340.58**	0.57	96.96	37.49	**1378.34**	0.73	192.45	70.92	**2607.53**	0.58	126.53	41.93	**1541.68**
80K	1	35	400	0.65	200.27	76.56	3217.22	0.63	110.77	45.46	1910.26	0.63	110.77	45.46	1910.20	0.81	206.01	88.28	3709.53	0.67	158.02	54.29	2281.32
	2	35	400	0.71	216.88	86.28	3625.35	0.63	109.18	44.91	1887.29	0.63	109.18	44.91	1887.29	0.80	207.36	82.96	3486.08	0.65	152.19	52.30	2197.49
	3	35	400	0.74	217.57	89.96	3780.18	0.75	99.32	41.88	1759.63	0.70	107.51	42.90	1802.83	0.89	209.38	105.24	4422.20	0.74	174.20	60.10	2525.16
	4	35	400	0.81	230.21	125.10	5256.80	0.75	97.96	41.37	1738.30	0.70	106.01	42.32	1778.40	0.87	208.06	88.87	3734.42	0.73	168.93	58.24	2447.09
	5	35	400	0.68	195.35	75.30	3164.16	0.75	81.41	105.95	4452.10	0.63	101.33	39.00	1638.58	0.82	191.71	77.71	3265.38	0.68	156.39	54.73	2299.82
	6	35	400	0.75	210.60	87.81	3689.94	0.75	80.30	107.48	4516.26	0.63	99.89	38.45	1615.47	0.81	190.18	74.77	3141.91	0.66	151.51	51.51	2164.60
	7	35	400	0.68	217.95	82.44	3464.23	0.63	110.77	45.64	1917.56	0.63	110.77	45.63	1917.52	0.81	206.00	88.42	3715.19	0.67	157.87	54.33	2282.97
	8	35	400	0.74	229.17	98.28	4129.73	0.63	109.18	45.09	1894.69	0.63	109.18	45.09	1894.69	0.80	207.36	83.10	3491.97	0.65	152.19	52.39	2201.41
	9	35	400	0.71	206.03	81.04	3405.16	0.75	90.36	40.18	1688.43	0.67	104.27	40.93	1719.84	0.85	200.35	87.58	3679.86	0.71	164.98	56.67	2381.19
	10	35	400	0.78	220.14	101.99	4285.77	0.75	89.13	39.76	1670.83	0.66	102.80	40.36	1696.09	0.84	198.91	79.97	3360.38	0.70	159.95	54.91	2307.50
	avg	35	400	0.72	214.42	90.48	**3801.85**	0.70	97.84	55.77	**2343.54**	0.65	106.17	42.51	**1786.09**	0.83	202.53	85.69	**3600.69**	0.69	159.62	54.95	**2308.85**
90K	1	35	400	0.73	225.06	93.33	4411.72	0.71	116.59	50.10	2368.39	0.71	116.59	50.10	2368.34	0.91	219.49	176.92	8363.26	0.77	198.25	71.81	3394.70
	2	35	400	0.80	230.18	135.32	6396.75	0.71	114.75	49.41	2335.92	0.71	114.75	49.41	2335.92	0.90	216.90	137.57	6503.20	0.75	192.01	69.40	3280.59
	3	35	400	0.84	227.65	144.34	6823.20	0.84	105.44	48.19	2277.82	0.79	114.72	47.84	2261.54	1.00	216.34	194.83	9209.78	0.85	190.71	69.27	3274.58
	4	35	400	0.91	233.96	221.65	10477.86	0.84	103.85	47.54	2247.19	0.78	112.94	47.10	2226.57	0.98	214.78	145.50	6878.18	0.84	184.11	65.91	3115.49
	5	35	400	0.76	208.10	101.70	4807.71	0.84	86.43	245.32	11596.69	0.71	109.22	44.40	2098.90	0.92	196.49	132.69	6272.63	0.78	168.60	60.30	2850.54
	6	35	400	0.84	216.64	153.03	7233.94	0.84	85.13	247.98	11722.37	0.71	107.47	43.67	2064.33	0.91	194.75	106.57	5037.79	0.76	162.76	58.05	2743.98
	7	35	400	0.76	228.76	99.91	4723.14	0.71	116.59	50.85	2403.63	0.71	116.59	50.85	2403.60	0.91	219.48	179.78	8498.31	0.77	198.25	72.38	3421.62
	8	35	400	0.83	234.85	151.86	7178.52	0.71	114.75	50.15	2370.66	0.71	114.75	50.15	2370.66	0.90	216.90	140.65	6648.64	0.75	192.01	69.88	3303.36
	9	35	400	0.80	221.36	125.27	5921.78	0.84	95.94	99.86	4720.68	0.75	111.76	45.92	2170.60	0.96	206.12	157.12	7427.41	0.81	179.18	64.33	3040.81
	10	35	400	0.88	227.50	187.65	8870.38	0.84	94.49	99.99	4726.78	0.75	109.99	45.19	2136.06	0.94	204.48	123.68	5846.41	0.80	172.99	61.96	2929.04
	avg	35	400	0.81	225.41	141.41	**6684.50**	0.79	103.40	98.94	**4677.01**	0.73	112.88	47.46	**2243.65**	0.93	210.57	149.53	**7068.56**	0.79	183.89	66.33	**3135.47**

8-Hour Non-Peak Traffic Results

High Lane Level

				4-phase SPUI				3-ph TUDI no overlap				3-ph TUDI w/overlap				4-ph TUDI no overlap				4-ph TUDI w/overlap			
ADT	run	speed	r-to-r dist	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K	v/s	cycle	delay	'01 cost of delay $1K
40K	1	35	400	0.23	100.88	38.05	1143.22	0.23	56.11	19.74	593.18	0.23	56.18	19.77	594.00	0.29	80.34	28.85	866.82	0.28	84.90	25.32	760.85
	2	35	400	0.25	103.95	39.18	1177.02	0.22	56.10	19.80	594.89	0.22	56.10	19.80	594.87	0.29	79.83	28.77	864.39	0.27	84.90	25.27	759.17
	3	35	400	0.27	105.54	38.90	1168.73	0.27	59.36	21.01	631.16	0.25	57.95	20.22	607.61	0.32	83.38	29.63	890.29	0.26	84.90	25.34	761.22
	4	35	400	0.29	109.15	40.11	1205.15	0.27	59.36	21.05	632.48	0.25	57.95	20.27	609.00	0.31	82.93	29.57	888.44	0.26	85.33	25.41	763.55
	5	35	400	0.24	102.44	37.54	1127.97	0.27	59.36	21.06	632.87	0.23	61.58	20.96	629.69	0.29	83.90	29.47	885.34	0.26	84.90	25.42	763.60
	6	35	400	0.27	105.74	38.69	1162.47	0.27	59.36	21.11	634.24	0.22	61.15	20.84	626.07	0.29	82.84	29.23	878.14	0.26	85.33	25.50	766.04
	7	35	400	0.24	102.34	38.36	1152.40	0.23	56.18	19.78	594.24	0.23	56.18	19.78	594.23	0.29	80.34	28.87	867.27	0.28	84.90	25.34	761.21
	8	35	400	0.27	105.52	39.50	1186.73	0.22	56.10	19.81	595.14	0.22	56.10	19.81	595.14	0.29	79.83	28.79	864.83	0.27	84.90	25.28	759.54
	9	35	400	0.25	103.99	38.20	1147.67	0.27	59.36	21.03	631.73	0.24	58.25	20.06	602.64	0.30	81.95	29.09	874.02	0.26	84.90	25.36	762.00
	10	35	400	0.28	107.41	39.36	1182.67	0.27	59.36	21.07	633.07	0.24	57.82	19.94	599.07	0.30	81.50	29.03	872.27	0.26	85.33	25.44	764.38
	avg	35	400	0.26	104.70	38.79	**1165.40**	0.25	58.07	20.55	**617.30**	0.23	57.93	20.14	**605.23**	0.30	81.68	29.13	**875.18**	0.27	85.03	25.37	**762.16**
50K	1	35	400	0.29	109.03	41.02	1540.57	0.28	60.51	21.70	815.00	0.28	60.51	21.70	815.01	0.36	89.41	32.15	1207.55	0.32	84.90	25.80	969.07
	2	35	400	0.32	113.63	42.61	1600.29	0.28	60.47	21.75	816.78	0.28	60.47	21.75	816.78	0.36	88.69	32.01	1202.07	0.32	84.90	25.73	966.32
	3	35	400	0.33	116.02	42.54	1597.62	0.33	65.33	23.56	884.78	0.31	63.21	22.47	843.92	0.40	94.30	33.51	1258.28	0.31	87.18	26.53	996.35
	4	35	400	0.36	121.53	44.29	1663.33	0.33	65.33	23.61	886.53	0.31	63.19	22.51	845.49	0.39	93.55	33.34	1252.23	0.30	88.42	26.76	1004.92
	5	35	400	0.30	111.37	40.64	1526.10	0.33	64.76	23.62	887.19	0.28	64.00	22.05	828.09	0.37	89.96	31.80	1194.42	0.30	88.17	26.79	1006.04
	6	35	400	0.33	116.31	42.25	1586.86	0.33	64.19	23.47	881.42	0.28	63.51	21.90	822.59	0.36	89.14	31.63	1187.78	0.30	88.42	26.72	1003.65
	7	35	400	0.30	111.22	41.54	1560.18	0.28	60.58	21.74	816.52	0.28	60.58	21.74	816.57	0.36	89.41	32.17	1208.31	0.32	84.90	25.82	969.69
	8	35	400	0.33	116.00	43.16	1620.88	0.28	60.47	21.76	817.28	0.28	60.47	21.76	817.29	0.36	88.69	32.03	1202.86	0.32	84.90	25.75	966.96
	9	35	400	0.32	113.67	41.55	1560.34	0.33	65.33	23.59	885.79	0.30	61.87	21.65	812.94	0.38	92.04	32.68	1227.42	0.30	88.15	26.77	1005.43
	10	35	400	0.35	118.87	43.21	1622.87	0.33	65.33	23.63	887.56	0.30	61.84	21.68	814.15	0.38	91.30	32.53	1221.71	0.30	88.42	26.72	1003.60
	avg	35	400	0.32	114.76	42.28	**1587.90**	0.31	63.23	22.84	**857.89**	0.29	61.96	21.92	**823.28**	0.37	90.65	32.39	**1216.26**	0.31	86.84	26.34	**989.20**
60K	1	35	400	0.35	118.72	44.56	2008.22	0.34	65.66	23.99	1081.33	0.34	65.66	23.99	1081.32	0.44	100.90	36.34	1637.83	0.35	84.90	26.30	1185.37
	2	35	400	0.38	125.30	46.76	2107.32	0.34	65.57	24.03	1082.97	0.34	65.57	24.03	1082.97	0.43	99.78	36.45	1642.58	0.36	84.90	26.21	1181.24
	3	35	400	0.40	128.80	46.99	2117.63	0.40	72.63	26.69	1202.71	0.37	69.52	25.17	1134.32	0.47	108.52	38.55	1737.40	0.37	88.57	27.45	1236.92
	4	35	400	0.43	137.10	49.57	2233.78	0.40	72.62	26.73	1204.81	0.37	69.47	25.20	1135.83	0.47	107.30	38.23	1723.00	0.34	92.18	28.33	1276.89
	5	35	400	0.36	122.00	44.32	1997.44	0.40	66.98	25.05	1128.89	0.34	66.65	23.25	1047.93	0.44	101.71	36.01	1622.64	0.34	93.84	28.78	1296.79
	6	35	400	0.40	129.24	46.63	2101.27	0.40	66.34	24.87	1120.79	0.34	66.04	23.06	1039.09	0.43	100.46	35.69	1608.25	0.33	92.06	28.08	1265.47
	7	35	400	0.36	121.79	45.34	2043.38	0.34	65.73	24.04	1083.54	0.34	65.74	24.04	1083.55	0.44	100.90	36.37	1639.14	0.35	84.90	26.32	1186.32
	8	35	400	0.40	128.77	47.63	2146.64	0.34	65.57	24.05	1083.73	0.34	65.57	24.05	1083.74	0.43	99.78	36.09	1626.56	0.36	84.90	26.23	1182.23
	9	35	400	0.38	125.33	45.59	2054.36	0.40	72.63	26.72	1204.37	0.36	67.59	24.08	1085.07	0.46	104.96	37.29	1680.60	0.36	91.00	28.08	1265.38
	10	35	400	0.42	133.05	48.00	2163.16	0.40	72.62	26.77	1206.51	0.36	67.52	24.10	1086.02	0.45	103.76	36.99	1667.00	0.33	92.04	28.16	1268.95
	avg	35	400	0.39	127.01	46.54	**2097.32**	0.38	68.63	25.30	**1139.97**	0.35	66.93	24.10	**1085.98**	0.45	102.81	36.80	**1658.50**	0.35	88.93	27.39	**1234.56**
70K	1	35	400	0.40	130.24	48.77	2564.31	0.39	71.85	26.76	1406.72	0.39	71.76	26.72	1404.86	0.51	115.78	41.78	2196.47	0.39	87.78	27.73	1458.07
	2	35	400	0.44	139.64	51.92	2729.80	0.39	71.62	26.74	1405.75	0.39	71.62	26.74	1405.73	0.50	114.03	41.28	2170.59	0.39	86.97	27.44	1442.90
	3	35	400	0.46	144.76	52.55	2762.67	0.47	81.77	30.61	1609.19	0.44	77.23	28.47	1497.08	0.55	127.79	45.40	2387.07	0.44	93.87	29.58	1555.12
	4	35	400	0.51	157.22	56.46	2968.49	0.47	81.73	30.64	1611.01	0.44	77.14	28.49	1497.94	0.55	125.78	44.82	2356.26	0.40	101.97	31.97	1681.09
	5	35	400	0.42	134.88	48.79	2565.31	0.47	69.34	26.65	1401.34	0.40	72.00	25.53	1342.41	0.51	116.98	41.48	2180.78	0.40	100.17	31.14	1637.15
	6	35	400	0.47	145.40	52.15	2741.74	0.47	68.64	26.44	1390.36	0.39	71.76	25.48	1339.79	0.50	115.06	40.93	2152.07	0.37	101.43	31.36	1648.91
	7	35	400	0.42	134.57	49.94	2625.71	0.39	71.76	26.74	1406.10	0.39	71.76	26.74	1406.06	0.51	115.78	41.82	2198.57	0.39	87.78	27.76	1459.52
	8	35	400	0.46	144.71	53.28	2801.23	0.39	71.62	26.76	1406.95	0.39	71.62	26.76	1406.94	0.50	114.03	41.33	2172.74	0.39	86.97	27.47	1444.34
	9	35	400	0.44	139.67	50.55	2657.96	0.47	76.94	29.12	1531.18	0.42	74.48	27.01	1420.12	0.53	122.09	43.41	2282.59	0.42	96.75	30.30	1593.27
	10	35	400	0.49	151.08	54.15	2846.87	0.47	76.19	28.90	1519.50	0.41	74.35	27.01	1420.13	0.53	120.18	42.87	2254.03	0.38	99.84	31.10	1635.26
	avg	35	400	0.45	142.22	51.86	**2726.41**	0.44	74.14	27.94	**1468.81**	0.41	73.37	26.90	**1414.11**	0.52	118.75	42.51	**2235.12**	0.40	94.35	29.59	**1555.56**
80K	1	35	400	0.46	144.23	53.89	3238.20	0.45	79.11	30.01	1803.13	0.45	79.21	30.05	1805.47	0.58	135.92	49.14	2952.96	0.45	94.41	30.42	1828.15
	2	35	400	0.51	157.68	58.42	3510.23	0.45	78.89	30.00	1802.61	0.45	78.89	30.00	1802.58	0.57	133.03	48.25	2899.42	0.45	94.04	30.31	1821.33
	3	35	400	0.53	165.22	59.68	3586.14	0.54	87.66	33.67	2023.01	0.50	86.86	32.61	1959.22	0.63	155.38	55.22	3318.16	0.50	103.07	33.02	1984.11
	4	35	400	0.58	184.28	65.74	3949.95	0.53	86.75	33.38	2005.48	0.50	86.70	32.60	1958.58	0.62	151.96	54.15	3253.66	0.48	106.55	34.12	2050.17
	5	35	400	0.49	150.79	54.32	3264.06	0.54	71.89	28.52	1713.87	0.45	79.42	28.67	1722.44	0.59	137.66	48.90	2938.08	0.45	106.38	33.57	2016.87
	6	35	400	0.53	166.18	59.25	3560.37	0.53	71.10	28.28	1699.34	0.45	79.10	28.58	1717.38	0.58	134.64	47.98	2882.80	0.43	106.91	33.70	2025.13
	7	35	400	0.48	150.36	55.63	3342.43	0.45	79.11	30.04	1804.89	0.45	79.11	30.04	1804.88	0.58	135.80	49.16	2953.79	0.45	94.41	30.46	1830.31
	8	35	400	0.53	165.15	60.52	3636.78	0.45	78.89	30.03	1804.41	0.45	78.89	30.03	1804.41	0.57	133.03	48.31	2902.89	0.45	94.04	30.35	1823.49
	9	35	400	0.51	157.71	56.81	3413.73	0.54	79.76	30.94	1859.32	0.48	82.93	30.61	1839.49	0.61	145.92	51.94	3120.97	0.48	102.39	32.60	1958.70
	10	35	400	0.56	174.76	62.23	3739.12	0.53	78.92	30.68	1843.64	0.47	82.72	30.58	1837.71	0.60	142.78	50.97	3062.91	0.46	102.46	32.60	1959.15
	avg	35	400	0.52	161.64	58.65	**3524.10**	0.50	79.21	30.55	**1835.97**	0.46	81.38	30.38	**1825.22**	0.59	140.61	50.40	**3028.57**	0.46	100.47	32.12	**1929.74**
90K	1	35	400	0.52	161.58	60.25	4072.82	0.51	88.15	34.05	2301.98	0.51	88.15	34.05	2302.05	0.65	164.19	59.50	4022.38	0.51	106.83	35.01	2366.62
	2	35	400	0.57	181.09	66.85	4519.17	0.50	87.81	34.00	2298.55	0.50	87.81	34.00	2298.56	0.64	159.62	58.02	3922.40	0.50	104.03	34.18	2310.39
	3	35	400	0.60	192.43	69.18	4676.45	0.60	91.00	35.77	2418.07	0.56	97.83	37.60	2541.76	0.71	198.17	70.47	4763.47	0.57	121.55	39.70	2683.67
	4	35	400	0.65	217.53	79.73	5389.89	0.60	89.98	35.43	2394.72	0.56	96.72	37.21	2515.47	0.70	191.90	68.40	4623.84	0.56	119.00	39.59	2676.11
	5	35	400	0.55	170.96	61.34	4146.21	0.60	74.63	30.94	2091.50	0.51	88.55	32.53	2198.79	0.66	167.21	59.52	4023.40	0.51	117.51	37.76	2552.32
	6	35	400	0.60	193.89	68.73	4646.36	0.60	73.75	30.67	2073.59	0.51	88.09	32.39	2189.41	0.65	162.24	57.92	3915.37	0.50	114.52	36.85	2490.86
	7	35	400	0.54	170.34	62.83	4247.14	0.51	88.15	34.09	2304.60	0.51	88.15	34.09	2304.65	0.65	164.19	59.58	4027.78	0.51	106.93	35.09	2372.36
	8	35	400	0.60	192.33	70.17	4743.10	0.50	87.81	34.04	2301.19	0.50	87.81	34.04	2301.19	0.64	159.62	58.93	3983.86	0.50	104.03	34.22	2313.43
	9	35	400	0.57	181.10	64.93	4389.38	0.60	82.83	33.03	2232.77	0.54	93.53	35.14	2375.61	0.69	181.30	64.62	4367.93	0.54	114.45	37.14	2510.67
	10	35	400	0.63	207.25	73.32	4956.43	0.60	81.86	32.70	2210.79	0.53	93.22	35.07	2370.52	0.68	175.83	62.84	4247.98	0.53	112.10	36.44	2463.11
	avg	35	400	0.58	186.85	67.73	**4578.69**	0.56	84.60	33.47	**2262.78**	0.52	90.99	34.61	**2339.80**	0.67	172.43	61.98	**4189.84**	0.52	112.10	36.60	**2473.95**

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