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# **Development of Guidelines for Driveway Location and Median Configuration in the Vicinity of Interchanges**

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16. Abstract Access connections and signalized intersections in the vicinity of an interchange can greatly affect the operations of the arterial and the freeway mainline. Previous studies on this topic have been focused on operations (e.g., arterial delays) and policy issues (e.g., right-of-way acquisition). The focus of this study was primarily on operations issues, but the intent was to take a more comprehensive approach, simultaneously considering several factors that influence arterial operations near an interchange. The main objective was to develop some quantitative tools and guidance for the location of signalized intersections in the vicinity of interchanges. The main sections of this report include the following: a literature review that provides a summary of technical studies, state practices, and national guidelines for access management; a discussion of diamond interchange design issues and their relationship to arterial operations; an overview of available simulation and analytical methods for the analysis of arterial operations in interchange areas; an overview of the relationship between signal spacing and other related factors on progression quality; and the development of tools for the assessment of arterial operations in the vicinity of interchanges. The recommendations for minimum signal spacing near interchanges derive firstly, from the consideration of progression quality at the arterial and secondly, from the estimation of the operating speed and the desired level of service. A software tool was developed that can be used by analysts to assess, at the planning level, the effect of several roadway, traffic, and control variables on arterial operations. The tool has two main features: 1) it will provide an assessment of the adequacy of a given signal spacing with respect to progression quality, and 2) it will provide an estimate of the average travel speed between the interchange off-ramp and first downstream signal, as a function of arterial and driveway operational characteristics and signal distance. The research findings indicate that a minimum signal distance of ¼ mile is sufficient for a range of conditions considering arterial speeds and progression quality; however, more restrictive guidelines of ½ mile should be applied in cases where the anticipated development will reach high levels. In general, the findings are supportive of the guidelines provided in FAC Rule 14-97.			
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## **Disclaimer**

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

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- The content of Chapters 2, 3, and 4 were developed from contributions by Mr. Francis de la Rosa (former graduate student) as part of his Master's report, as supervised by Drs. Scott Washburn and Fazil Najafi.
- The content of Chapter 5 was developed from contributions by Ms. Thuha Nguyen (former graduate student) as part of her Master's project, as supervised by Dr. Scott Washburn and Prof. Ken Courage.

The remainder of the content in the report was developed from contributions by Ms. Alexandra Kondyli serving as a graduate research assistant, as supervised by Dr. Scott Washburn.

Note: The focus of this project was reoriented towards signal spacing considerations in the vicinity of interchanges, with the consideration of driveway factors, as a result of certain circumstances and direction provided by the project manager. The effect of median configuration on arterial operations was also originally included in the analytical model development, but was ultimately excluded for reasons discussed in Chapter 6.

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## Chapter 1: Introduction

Business development in the vicinity of interchanges is often very dense. These businesses, particularly in more rural areas, often consist of gas stations, restaurants, and other similar service/convenience facilities. These types of businesses serve a significant percentage of the passing freeway traffic.

To serve these businesses, driveways must be installed along the arterial. As the area continues to grow, large businesses (e.g., “big-box” retail) are often attracted to the area. This often leads to the necessity to install signalized intersections along the arterial to handle the increased traffic, particularly turning traffic. Alternatively, existing driveway locations may be converted to a signalized intersection.

Driveways and/or signalized intersections installed close to interchange ramp intersections can interfere with the efficient movement of traffic at the interchange. The spacing of signalized intersections and resultant progression quality has probably the largest impact on arterial operations. However, the number of unsignalized driveways in the vicinity of an interchange, particularly if combined with non-restrictive medians, can also have a significant effect on arterial operations.

Excessive queuing is generally a by-product of inefficient arterial operations. This can become a significant safety issue, particularly for interchange off-ramps. Queues that extend upstream to the freeway mainline present a very hazardous situation. Additionally, excessive queues at driveway locations will promote riskier gap acceptance behavior by motorists.

The Access Management Program of FDOT has developed some guidelines based on their experience and preliminary work done by others; however, they are still not comprehensive and not always based on quantitative findings. Thus, this sometimes puts Access Management review staff in the position of having to use subjective judgments to base access permit review decisions on. The objective of this project was to develop some additional quantitative and objective-based guidance with respect to signalized intersection spacing and the impact of driveways on arterial operations in the vicinity of interchanges. An additional product of this project are two analytical tools implemented in a software format for use in estimating the impact of signalized intersection spacing on arterial performance in the vicinity of interchanges, based on traffic, roadway, and control input variables.



## **Report Organization**

Chapter 2 provides an overview of some of the previous studies and/or reference manuals relevant to the area of access management in the vicinity of interchanges. Chapter 3 provides a conceptual overview of the impacts of the most significant design issues for diamond interchanges with regard to access management issues. Additionally, some alternatives to the diamond interchange are discussed. Chapter 4 provides an overview of the more commonly available simulation and analytical methods for evaluating arterial operations at interchanges. Chapter 5 gives an overview of the theoretical basics of signal spacing for the consideration progression quality. It also introduces a simple quantitative tool for providing an estimate of the adequacy of signal spacing with regard to two-way signal progression. Chapter 6 details the development of a model for the estimation of speed reductions due to signal spacing, number of driveways, and traffic characteristics. Chapter 7 summarizes the major findings of the project and provides recommendations for possible inclusion in future access management guidelines/handbooks from the Florida Department of Transportation.

## Chapter 2: Literature Review

*A Policy on Design Standards—Interstate System* (1), (AASHTO) provides some access spacing guidelines in the vicinity of interchanges that are followed by many state agencies. The current guidelines are, however, far from comprehensive and not always based on quantitative findings. Many of the guidelines that were developed are just suggested values and should be adjusted to reflect local conditions. Thus, this often leaves access management review staff having to use more subjective judgments to base access review decisions on. This chapter reviews some important studies that developed access spacing guidelines near interchange areas used by various state and national agencies.

### Overview of State Practices

Every state handles access management differently. Some state agencies have their own design policies on access spacing while others rely on research from other state agencies or even use AASHTO guidelines. AASHTO (1) states that control should be extended beyond the ramp terminal at least 100 feet in urban areas and 300 feet in rural areas. These distances satisfy concerns with congestion, but in some areas there needs to be longer lengths of access control. Some states follow AASHTO guidelines while other states have their own policies and performed their own research to develop their own guidelines. Guidelines by state agencies are in some cases less than the access spacing required to provide adequate weaving distances and storage for turns. National Cooperative Highway Research Program (NCHRP) Report 420: Impact of Access Management Techniques (2) summarizes separation distances used by various state DOTs discussed later. They range from 300 feet to 1,000 feet in rural areas and from 100 to 700 feet in urban areas.

Some widely used access management guidelines come from the NCHRP Report 420 (2) and also the Access Management Manual developed by the Transportation Research Board (3). Many states develop their own access management spacing guidelines while others rely on the research performed by others. Some other state DOTs that have an access management program and have produced their own access management manuals include Oregon, Florida, Wisconsin, Texas, Ohio, and Iowa. Some of these manuals are still basic in nature. The most significant research has come from Oregon, Florida, NCHRP Report 420,

and the TRB Access Management Manual. More details about these significant references are given in the following sections.

### **Oregon Department of Transportation Guidelines**

Some of the earliest and most comprehensive research on access management near interchanges was sponsored by the Oregon Department of Transportation. The following sections detail the research conducted by Layton (4) for the Oregon Department of Transportation to find: the distance to first major intersection; distance to the first driveway; and the distance to the first median opening. This research provides a very detailed discussion of the logic behind minimum access spacing requirements near interchange areas. However, each area type has fluctuations in volume level that may require more subjective judgment when designing access spacing distances. These guidelines are only recommended minimum values and in some situations may require greater distances.

#### Distance to First Major Intersection

According to Layton, the minimum spacing to the nearest major intersection from the off-ramp should take into account the distance required for weaving maneuvers to take place from the off-ramp to the left turn bay at the intersection. Also, since the weaving maneuvers must be completed before the vehicle arrives at the end of the queue, the spacing to the nearest major intersection is the weaving distance plus the queue length at the intersection.

The weaving distance is determined from a series of curves developed by Jack Leisch (5), seen in Figure 1, and the queue distance at the left turn bay is estimated based on the assumption that vehicles arrivals follow a Poisson (i.e., random) distribution. An assumption is made that 50% of the left turning volume at the major intersection is from the off-ramp and the other 50% is from the mainline volume. The total weaving volume ( $V_w$ ) is then calculated by adding the total through volume ( $V_I$ ) with the volume from the ramp turning left ( $\%LT \times [V_1 + V_2] / 2$ ).

$$V_w = V_I + \%LT \times [V_1 + V_2] / 2 \quad [1]$$

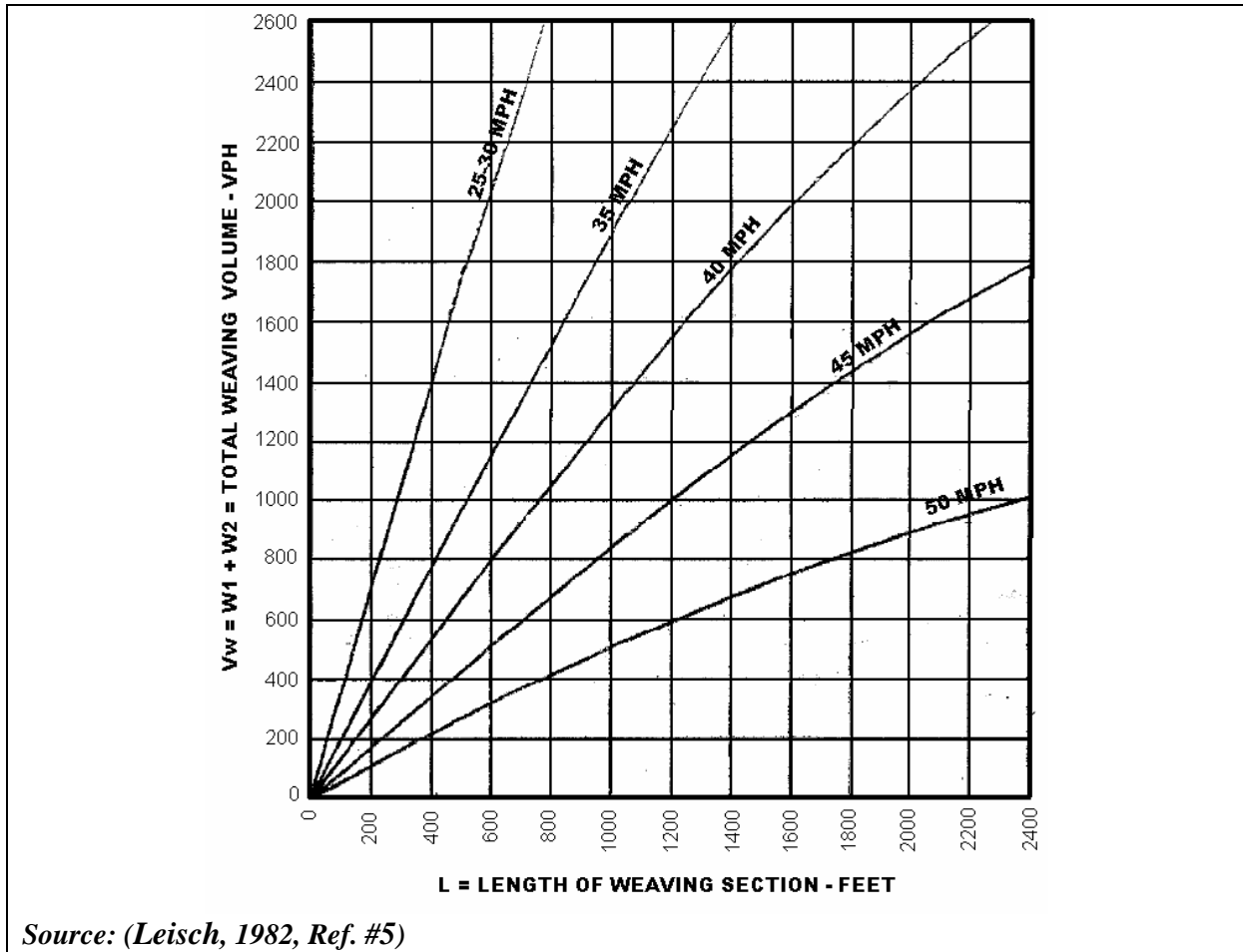
where:

$V_I$  = Cross road volume (veh/hr)

$V_2$  = Ramp volume (veh/hr)

$\%LT$  = Percentage of total vehicles turning left

Once the weaving volume and speed are known, the length of the weaving section can be determined using Figure 1. Table 1 shows the required weaving distance for different area types and volume levels, calculated with 10% and 20% left turns, taken from the series of graphs developed by Leisch.



**Figure 1: Analysis of service road weaving condition**

**Table 1: Development of weaving distances for four lane cross roads with 10% and 20% left turns**

Area Type	Volume Level	Cross Road Volume (veh/hr/ln)	Off Ramp Volume (veh/hr)	Weaving Volumes (veh)		Weaving Distance (ft)	
				10% LT	20% LT	10% LT	20% LT
Urban (35 mph)	High	800	600	1710	1820	900	920
	Moderate	700	500	1495	1590	790	830
	Low	600	400	1280	1360	660	710
Suburban (45 mph)	High	500	400	1070	1140	1300	1380
	Moderate	400	300	855	910	1030	1100
	Low	300	200	640	680	750	820
Rural (55 mph)	High	300	150	637	675	2100	2200
	Moderate	200	100	425	450	1350	1500
	Low	100	50	212	225	600	650

*Source: (Layton, 1996, Ref. #4)*

The queuing distance is calculated assuming Poisson-distributed arrivals and typical volumes and signal operations shown in Table 2. Table 3 summarizes the weaving distance plus the queue distance to give the minimum distance to the nearest major intersection. According to Layton, the minimum spacing is about 1,320 ft or ¼ mi for moderate volumes for typical urban, suburban, and rural conditions. A spacing of 2,000 ft or ½ mi would accommodate all conditions except high volume urban conditions.

**Table 2: Queue sizes by probabilistic Poisson analysis with 10% left turns**

Area Type	Typical Volume, 2 lanes (veh/hr)	Typical Ramp, 2 lanes (veh/hr)	Cycle (sec)	Through Green (sec)	Left Turn Green (sec)	Queue Size (veh)	Queue Distance (ft)
Urban (35 mph)	1600	600	120	60	13	25	625
	1400	500	120	52	12	24	600
	1200	400	120	44	11	23	575
Suburban (45 mph)	1000	400	90	42	10	15	375
	800	300	90	42	10	12	300
	600	200	90	42	10	9	225
Rural (55 mph)	600	150	60	25	-	7	175
	400	100	60	25	-	5	125
	200	50	60	25	-	3	75

*Source: (Layton, 1996, Ref. #4)*

**Table 3: Minimum spacing to first signalized intersection**

Area Type	Volume Level	Weaving Distance (ft)		Queuing Distance (ft)		Minimum Spacing to Next Major Signalized Intersection (ft)	
		10% LT	20% LT	10% LT	20% LT	10% LT	20% LT
Urban (35 mph)	High	900	970	625	600	1525	1570
	Moderate	790	830	600	600	1390	1430
	Low	660	710	575	575	1235	1285
Suburban (45 mph)	High	1300	1380	375	400	1675	1780
	Moderate	1030	1100	300	325	1330	1425
	Low	750	820	225	250	975	1045
Rural (55 mph)	High	2100	2200	175	150	2275	2350
	Moderate	1350	1500	125	125	1475	1625
	Low	600	650	75	75	675	725

*Source: (Layton, 1996, Ref. #4)*

#### Distance to the First Access/Driveway

According to Layton, the distance to the first driveway on the right from the off-ramp should take into account three things: stopping sight distance, maximum egress capacity, and decision sight distance. The stopping sight distance must be used as a criterion because the driver coming from the off-ramp must have enough distance to see the situation and stop for vehicles turning right. The maximum egress capacity should also be taken into account to reduce delay along the arterial. The maximum egress capacity is based on research performed by Major and Buckley (6), reporting that driveways spaced at distances greater than 1.5 times the distance to accelerate from zero to the speed of traffic will reduce delay to vehicles in the traffic stream. The decision sight distance must be taken into account because the drivers must be given enough distance to perceive and react to any unexpected or unusual situations. Also if drivers are unfamiliar with the area they should have enough distance to sort out where they need to go. Layton summarized these distances in a table, included here as Table 4. These values were based on AASHTO (7) guidelines.

In describing these table values, Layton states “The spacing to the first drive or access road must take account of decision sight distance. A spacing of 660 ft provides a distance slightly greater than the decision sight distance for stopping on both rural and urban roads (590 ft and 620 ft). Decision sight distance provides an increase in perception reaction time as the situation complexity increases, therefore, the perception-reaction time is longer for urban

areas with the increased complexity of traffic operations and land use.” and “The braking distance is greater on higher speed rural facilities than urban. Consequently, the decision sight distances for stopping for both rural and urban facilities sums to about 660 ft. Also, this is half of 1320 ft. (1/4 mi.) which places the drive/access approach halfway between the ramp terminal and the nearest signalized intersection, or major intersection.” The distance to the first driveway/access is shown as the distance “X” in Figure 2.

**Table 4: Sight distance criteria comparison**

Area	Speed (mph)	Stopping Sight Distance (ft)	Maximum Egress (ft)	Decision Sight Distance for Stopping (ft)	Decision Sight Distance for Avoidance Maneuver (ft)
Urban	35	250	450	620	710
Suburban	45	400	860	640	810
Rural	55	550	1500	590	870

*Source: (Layton, 1996, Ref. #4)*

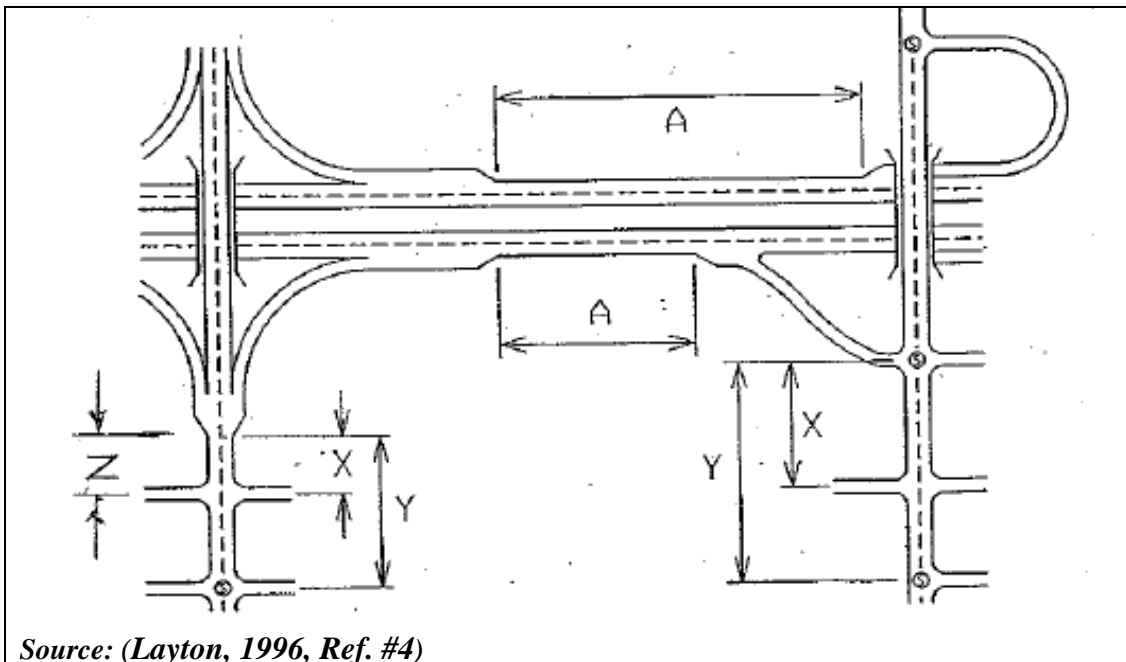
#### Distance to the First Median Opening

Layton found that the first median opening from the off-ramp, providing access to the first driveway on the left, requires an adequate distance for weaving maneuvers. Table 5 shows the minimum weaving distance to the first median opening from the off-ramp tapers, based on Figure 1. A distance of 1200 ft to 1250 ft could serve typical rural and suburban locations. The spacing between the nearest access and the on-ramp should be placed at the decision sight distance for speed/path or direction changes. Layton recommended that median openings should not be spaced any closer than 1000 ft from the off-ramp as it can potentially disrupt operations. The distance to the first median opening is shown as the distance “X” in Figure 2.

**Table 5: Minimum weaving distance to first median opening**

Area Type	Volume Level	Through Volume, 2 Lanes (veh/hr)	Typical Ramp Volume (veh/hr)	Total Weaving Volume (veh/hr)	Weaving Distance (ft)
Urban (35 mph)	High	2000	800	2001	1050
	Moderate	1600	600	1601	830
	Low	1200	400	1201	620
Suburban (45 mph)	High	1000	400	1001	1200
	Moderate	800	300	801	950
	Low	600	200	601	700
Rural (55 mph)	High	600	150	601	2220
	Moderate	400	100	401	1250
	Low	200	50	201	520

Source: (Layton, 1996, Ref. #4)



Source: (Layton, 1996, Ref. #4)

**Figure 2: Spacing to first driveway/access from off-ramp**



## Florida Department of Transportation Guidelines

The Florida Department of Transportation's Access Management Program has developed some guidelines based on their experience and preliminary work done by others. The Florida Department of Transportation's current standard used for access separation in interchange areas since 1988 is Florida Administrative Code (FAC) Rule 14-97 for the adopted classification system and standards for state highway systems and Rule 14-96 which describes the connection permitting application process and procedures (8).

The following are the major references developed by FDOT that are used for guidelines on access spacing near interchange areas:

- Median Handbook – Chapter 2.3 (9)
- Interchange Handbook – Chapter 2.9.2 (10)
- Driveway Handbook – Chapter 9.4 (11)

These handbooks all refer to FAC Rule 14-97, Table 6, for guidelines on access spacing in interchange areas. These, however, are minimum standards and under certain circumstances a greater distance might be required. This rule also does not establish a minimum distance to the nearest signalized intersection or median opening from an off-ramp terminal. This leaves access management staff having to use more subjective judgments for access spacing near interchange areas.

Rule 14-97 (8) states:

*“Connections and median openings on a controlled access facility located up to 1/4 mile from an interchange area or up to the first intersection with an arterial road, whichever distance is less, shall be regulated to protect the safety and operational efficiency of the limited access facility and the interchange area. The 1/4 mile distance shall be measured from the end of the taper of the ramp furthest from the interchange...the first connection from the end of the ramp taper shall be at least 660 feet where the posted speed limit is greater than 45 MPH or 440 feet where the posted speed limit is 45 MPH or less...the minimum distance to the first median opening shall be at least 1320 feet as measured from the end of the taper of the egress ramp.”*

**Table 6: FAC Rule 14-97.003**

Access Class	Facility Design Features	Minimum Connection Spacing* (ft)	Minimum Median Opening Spacing, Directional (ft)	Minimum Median Opening Spacing, Full* (mi)	Minimum Signal Spacing* (mi)
2	Restrictive with Service Roads	1320 / 660	1320	0.5	0.5
3	Restrictive	660 / 440	1320	0.5	0.5
4	Non-Restrictive	660 / 440	N/A	N/A	
5	Restrictive	440 / 245	660	0.5 / 0.25	0.5 / 0.25
6	Non-Restrictive	440 / 245	N/A	N/A	0.25
7	Both	125	330	0.125	0.25
<b>Source: (FDOT, Ref. #8)</b> <i>* Greater than 45 mph / Less than or equal to 45 mph</i>					

The access standards go from the most restrictive (Class 2) to the least restrictive (Class 7). Class 1 (not shown here) are freeways with no access features other than interchanges. The arterial access class is a function of the extent of abutting roadway development and the degree of access control that is offered. For example, access classes 2 and 3 are associated with typically high-speed controlled access facilities, where the abutting land is or will be controlled to maximize the operation of the through movement. Access classes 6 or 7 usually describe roadways in urbanized areas, where the existing land use and roadway sections are built to the maximum possible extent, and there is little intent for providing high-speed travel.

This rule allows a minimum spacing of 660 ft to the first access with a restrictive median while other stated guidelines require a 750 ft minimum spacing to the first access/driveway. FDOT's Driveway Handbook (11) includes some recommendations regarding minimum driveway spacing before an on- or off-ramp. These guidelines place the driveway at a minimum of 750 ft to 900 ft, depending on the area type of the facility.

## Other Relevant Studies

FAC Rule 14-97 is currently general in nature and could be enhanced by further research to develop more comprehensive guidelines. FDOT has been moving in that direction more recently with several research projects, including this one. The Center for Urban Transportation Research (CUTR), located at the University of South Florida, has been the most active research contractor with FDOT on this topic. A relatively recent study by Land and Williams (12) on the subject of access management near interchange areas offers several recommendations to the current rules and also performed some case studies on inadequately managed access.

According to Land and Williams, the current rules and regulations regarding access management should be strengthened to preserve the interchange areas. The authors recommend additions in Chapter 163, F.S. stating that local governments should be required to incorporate state access management regulations into their land development regulations for state highways.

Additions in the Rule 9J-5, FAC are also suggested, stating that the interchange areas and FIHS facilities should be identified as a priority for controlling connections and access points. Moreover, it is recommended that all interchange areas should be identified in the Intergovernmental Coordination Element of local comprehensive plans, along with a demonstration of how intergovernmental compatibility will be achieved. The intergovernmental coordination of interchange areas should be sought even without a change in the current rule, through a written comment during plan review/update.

Land and Williams also propose changes in the FDOT Rules 14-96 and 14-97, regarding:

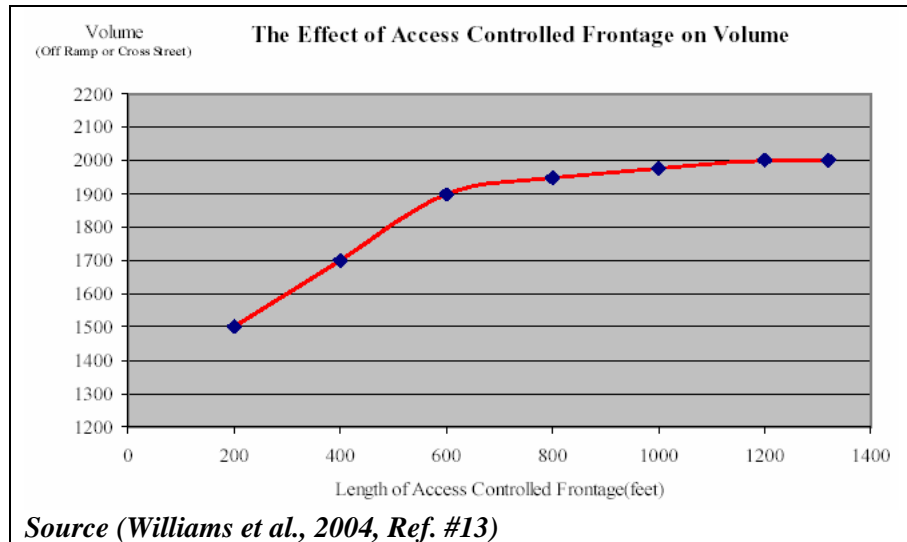
- Increase of the area for regulating minimum connections and median openings to  $\frac{1}{2}$  mile from the interchange area, and denote that area as “area of special concern” in these rules.
- On state roads within the interchange area, FDOT should have the option to attach conditions to its Notice of Intent to Permit, in order to ensure developer cooperation in preserving the functional integrity of the interchange area.
- The variance from the 660 ft should be approved only if the applicant can prove unmitigating circumstances.

The authors also proposed additions and changes to the *Interchange Request Development and Review Manual*. The one most related to the subject of this report is that it is advised to not approve a new interchange when the cross street connection is less than 1320 ft, or when no parallel roadway facilities are available. The other recommendations by Land and Williams with respect to this manual were mostly related to agency coordination issues and general policy advice.

A very recent study by CUTR (13) examined the cost-effectiveness of purchasing additional limited access right-of-way (ROW) at interchange areas, in terms of safety, operational, and fiscal benefits.

As part of the operational analysis, simulation was used to test the effects of limiting access near the freeway interchange. The simulated network included one direction of freeway, a small segment of the arterial, the off-ramp and the downstream traffic signal. The evaluation of the effects of access control was based on measurements of the queue length on the interchange off-ramp and the vehicle hours of delay for the network. The goal of the simulation runs was to observe the traffic volumes on the off-ramp and arterial that would cause the interchange to “fail operationally”, for a wide range of signal distances (200 ft to 1320 ft at 200 ft increments). The term “fails operationally” was defined to mean that the queue at the off-ramp backs onto the freeway mainline.

The findings of this research help to identify the effects of the length of the access controlled frontage on the traffic back-ups on the interstate and the estimated delay savings between varied lengths of access control frontage. The combination of volume and length of access control frontage for which the interchange “fails operationally” is shown in Figure 3. This figure also provides the capacity gains from increasing the access spacing, and using a given annual volume growth rate the projected delay under alternate access spacings can be obtained. Thus, the cost benefit evaluation of the interchange is a function of the delay reduction (in vehicle-hours) between different lengths of access frontage over a 20-year design life.



**Figure 3. The Effect of Access Controlled Frontage on Volume**

McShane et al. (14) studied the effect of several variables related to access management on arterial performance. The primary performance measure of interest in this study was the average travel speed of the arterial thru vehicles. The research approach consisted of testing the effect on the performance measure (dependent variable) as a result of varying either one independent variable or a combination of two independent variables. Simulation was the tool used to investigate these relationships. For the development of simulation scenarios the varying factors used are the number and design of driveways, driveway volume and location, median configurations, arterial flow rates, number of arterial lanes and presence of arterial left-turn bays.

In addition, adequate green time at the signal and excellent progression was assumed, the signal distance was fixed at  $\frac{1}{4}$  mile, and when lefts-in/out were allowed, the percentage split was 70/30. A summary of the research findings is as follows:

- **Driveway right turns only vs. right/left turns:** The no-opening configuration benefits the arterial average speed, when compared to the full median opening, as left turns are not allowed. The elimination of left turns has more effect on the speed of the opposing traffic stream.
- **Impact of acceleration/deceleration lanes:** The average speed of thru vehicles was compared under four different scenarios: no driveways; one driveway with neither acceleration or deceleration lanes; one driveway with both acceleration and

deceleration lanes; and one driveway with only a deceleration lane. The driveway volume remained fixed at 180 veh/h. It was shown that the presence of such lanes have a positive impact on the arterial thru vehicle average speed. The deceleration lane was shown to have a much more significant impact on these speeds than an acceleration lane.

- **Impact of left-turn bay (on the opposing direction):** Four cases were tested: no deceleration lane and no median left turn bay with no driveway volume; no deceleration lane and no median left turn bay with a fixed driveway volume; a deceleration lane but no left turn bay with a fixed driveway volume; and both a deceleration lane and left turn bay with a fixed driveway volume. The fixed driveway volume was 180 veh/h. It was shown that the presence of a deceleration lane for right turns into a driveway has a significant benefit to the arterial thru traffic (for the primary direction) and a median left-turn bay has a significant benefit to the arterial thru traffic in the opposing direction.
- **Impact of the number of driveways:** The effect of driveways was tested on a six-lane undivided arterial. The authors developed scenarios of 0, 2, and 4 driveways, each carrying volume of 180 veh/h and fixed spacing of 150 ft. The results indicate a speed decrease when going from the zero driveways to 2 and further decrease at 4 driveways, for the whole range of arterial thru volume. The impact of increasing number of driveways increases with increasing arterial volume.
- **Impact of driveway volume:** The impact of driveway volume was tested by considering 2 driveways on the arterial link (on the side of the primary direction of travel), each handling a volume of 60, 120 and 180 veh/h, versus the no-driveway scenario. The findings indicate that the driveway volume reduces the arterial thru vehicles' average speed, and most significantly in the primary direction of travel.
- **Impact of driveway location and driveway volume dispersion:** As driveways were moved closer to the downstream signalized intersection, there were adverse effects on the opposing direction. The authors note that these were due to the difficulty imposed to left turning vehicles due to intersection queues, with consequent effects on the thru traffic in the opposite direction. It was also observed that as driveways were moved closer to the signalized intersection, the effect on queues within the driveways was

significant, because these vehicles got trapped by the intersection queues. The research results from the dispersion of traffic among the driveways (i.e., distribution of a fixed amount of driveway traffic among a varying number of driveways) are reported as complex due to several mechanisms that were at work.

## National References

This section provides an overview of the content of the two most significant references at the national level as they relate to access management in the vicinity of interchanges.

### NCHRP Report 420: Impacts of Access Management Techniques

National Cooperative Highway Research Program Report 420 (2) contains some of the more recent studies on access management. This report, along with Layton's research (4), is widely known and referenced by state agencies when dealing with the subject of access management in interchange areas. Chapter 9 primarily deals with the subject of access separation at interchanges and refers to previous research from Oregon for the guidelines.

From NCHRP 420 (2), the important elements to be considered for computing access separation distances are shown in Figure 4 and also discussed below.

- **Weaving Distance:** The weaving distance is the distance required to weave across the through lanes and into the left turn lane. Oregon and Florida use Figure 1 as the basis for required weaving distances. An adequate weaving distance is about 700 to 800 ft for two lane roads and 1,200 to 1,600 ft for multilane roads.
- **Transition Distance:** The transition distance is the distance required to transition into the left turn lane. An adequate transition distance to move into the left turn lane is 150 to 250 ft.
- **Left-Turn Storage:** The left-turn storage is the distance required to store left turning vehicles. Left turn storage length is typically 200 ft to 300 ft depending upon demand which can be considered using the following equation:

$$L = \frac{R \times V \times 25}{N_c} = R \times l \times 25 \quad [2]$$

Where:

$L$  = Length of left turn storage in ft

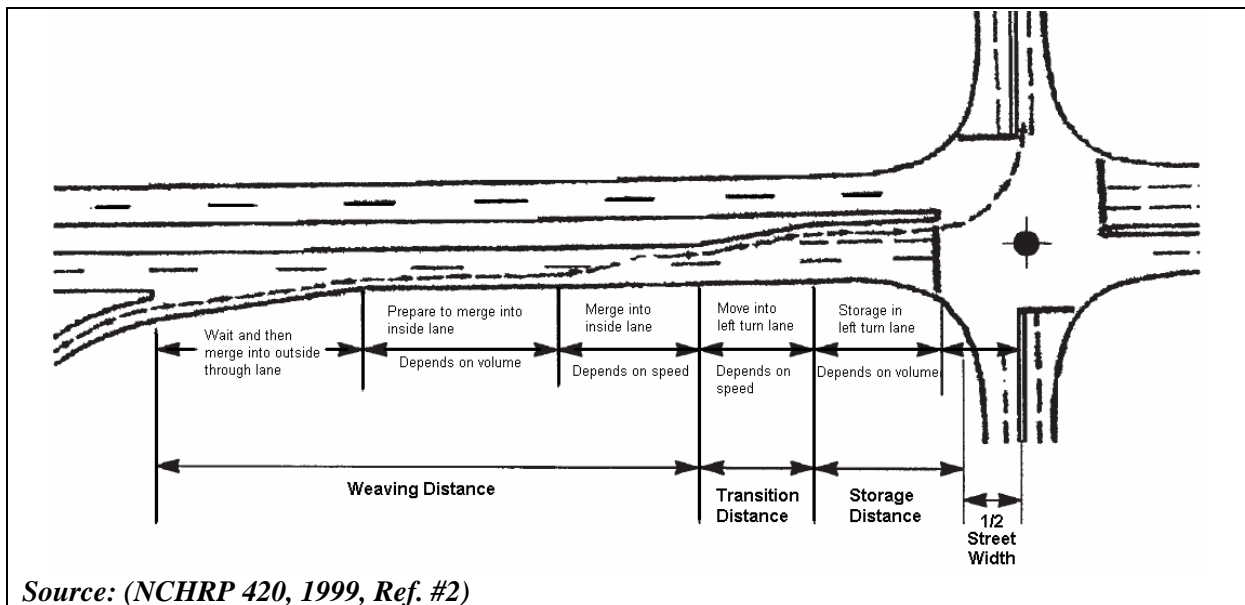
$R$  = Randomness factor for less than 5 percent failure; equal to 2.0 for random operations, or equal to 1.5 for operations where traffic tends to platoon

$V$  = Number of left turns in veh/hr

$N_C$  = Number of cycles per hour

$l$  = Number of left turns per cycle

- **Street Width Distance:** The street width is the distance from the stop line to the centerline of the intersecting road. This distance is normally about 50 ft.
- **Perception-Reaction Distance:** The perception-reaction distance is calculated at about 2.5 ft/sec and adds roughly 125 ft. This distance may be desirable to add in some situations, especially congested areas for drivers unfamiliar with the area.



**Figure 4: Factors influencing access separation distances**

Tables 7 and 8 summarize NCHRP Report 420 guidelines on access spacing in interchange areas for four-lane cross roads and two-lane cross roads, respectively. These access spacing distances are taken from Oregon's research, but modified slightly. NCHRP Report 420 changed the recommended minimum spacing to the first driveway to 750 ft from 660 ft.



**Table 7: Minimum spacing standards applicable to freeway interchanges with four lane cross roads**

Access Type	Area Type		
	Urban (45 mph)	Suburban (45 mph)	Rural(55 mph)
First Access from Off-ramp	750	990	1,320
First Median Opening	990	1,320	1,320
First Access Before On-Ramp	990	1,320	1,320
First Major Signalized Intersection	2,640	2,640	2,640

*Source: (NCHRP 420, 1999, Ref. #2)*

**Table 8: Minimum spacing standards applicable to freeway interchanges with two lane cross roads**

Access Type	Area Type		
	Urban (45 mph)	Suburban (45 mph)	Rural (55 mph)
First Access	750	990	1,320
First Major Signalized Intersection	1,320	1,320	1,320

*Source: (NCHRP 420, 1999, Ref. #2)*

NCHRP Report 420 also performed some background research on access spacing guidelines used by other state DOTs. The researchers discovered that generally the guidelines that are used are less than the access spacing required to ensure good progression and provide adequate weaving and storage for left turning traffic. Table 9 summarizes the minimum access spacing guidelines in interchange areas used by various state DOTs. They range from 300 ft to 1000 ft in rural areas and 100 ft to 700 ft in urban areas.

**Table 9: Minimum access separation distances near interchange areas used by state DOTs**

State	Rural	Urban
Alabama	300 feet (AASHTO)	100 feet (AASHTO)
California	125 m	125 m
Illinois	500 to 700 feet	500 to 700 feet
Iowa	200 m primary highway 100 m other road or street	50 m
Kentucky	300 feet (AASHTO)	100 feet (AASHTO)
North Dakota	300 feet (AASHTO)	100 feet (AASHTO)
Ohio	600 feet diamond interchange 1,000 feet cloverleaf interchange	600 feet diamond interchange 1,000 feet cloverleaf interchange
Oregon	300 feet from frontage road 500 feet from ramp	
Pennsylvania	300 feet (AASHTO)	100 feet (AASHTO)
South Carolina	500 feet desirable 300 feet minimum	300 feet desirable 150 feet minimum
Texas	300 feet (AASHTO)	100 feet (AASHTO)
Utah	300 feet	150 feet
Virginia	200 feet	200 feet
West Virginia	300 feet (AASHTO)	100 feet (AASHTO)
Washington	300 feet	300 feet
Wisconsin	1,000 feet major road 500 feet minor road	500 feet
Wyoming	300 feet	150 feet

*Source: (NCHRP 420, 1999, Ref. #2)*

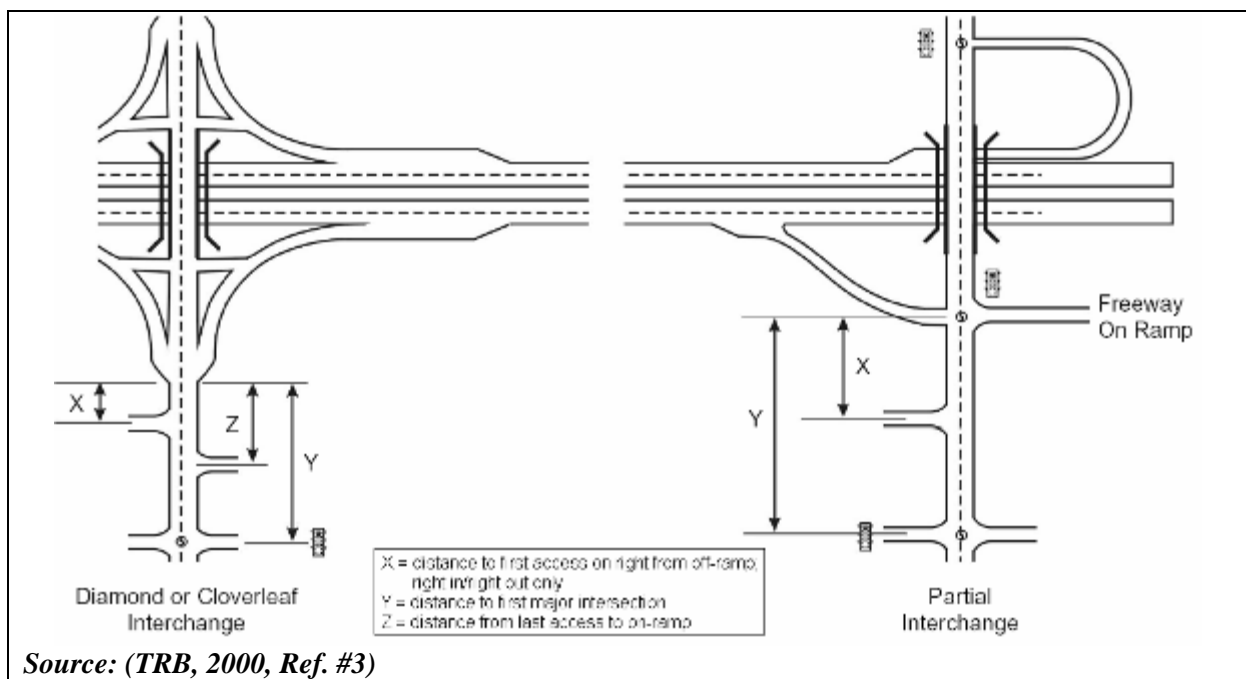
### TRB Access Management Manual

The Transportation Research Board has produced a manual on access management (3) that is the most current, nationally used manual on access management. The manual is a compilation of information from the biennial TRB Access Management conferences, along with insights from practitioners and state agencies to provide one source that summarizes the latest access management practices. The manual details many aspects of access management from access permitting to access spacing and also discusses state and local access management strategies. This manual also briefly discusses access management near interchanges.

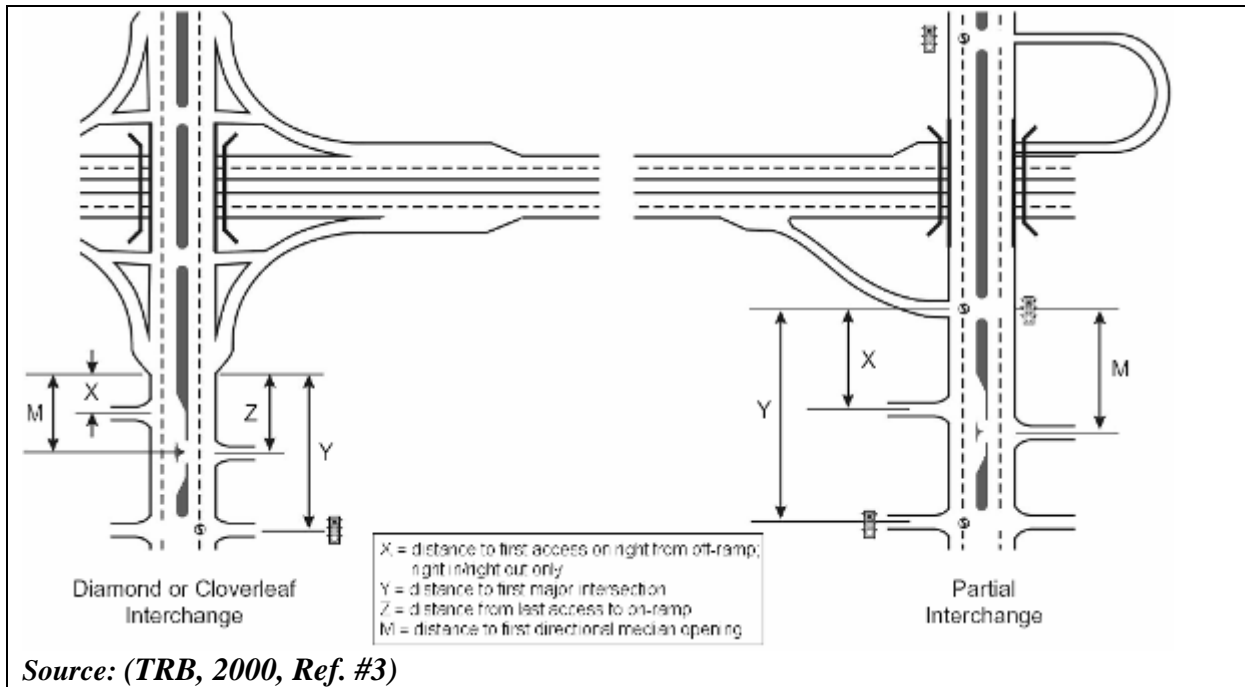
Chapter 9, titled *Access Spacing*, sets forth considerations in establishing access spacing distances. This chapter specifically has a section titled *Interchange Area Management* that briefly discusses access spacing near interchange areas. This section refers

to research from Oregon (4) and NCHRP Report 420 (2). This chapter also discusses the effects of signal spacing and the optimum spacing for various speeds that could minimize delay. A more detailed discussion on the impact of signal spacing on progression quality is presented in Chapter 5.

This manual provides more detailed illustrations on minimum access spacing in interchange areas in reference to Tables 7 and 8 from NCHRP Report 420 (2). These illustrations are shown in Figures 5 and 6. From these illustrations, the access spacing guidelines can be more easily applied to different interchange configurations such as diamond or cloverleaf interchanges or cloverleaf interchanges. The figures also illustrate the application of the access spacing guidelines to two-lane or multilane roads, as well as signalized and free-flow off-ramps.



**Figure 5: Illustration of minimum spacing for freeway interchange areas with two-lane crossroads**

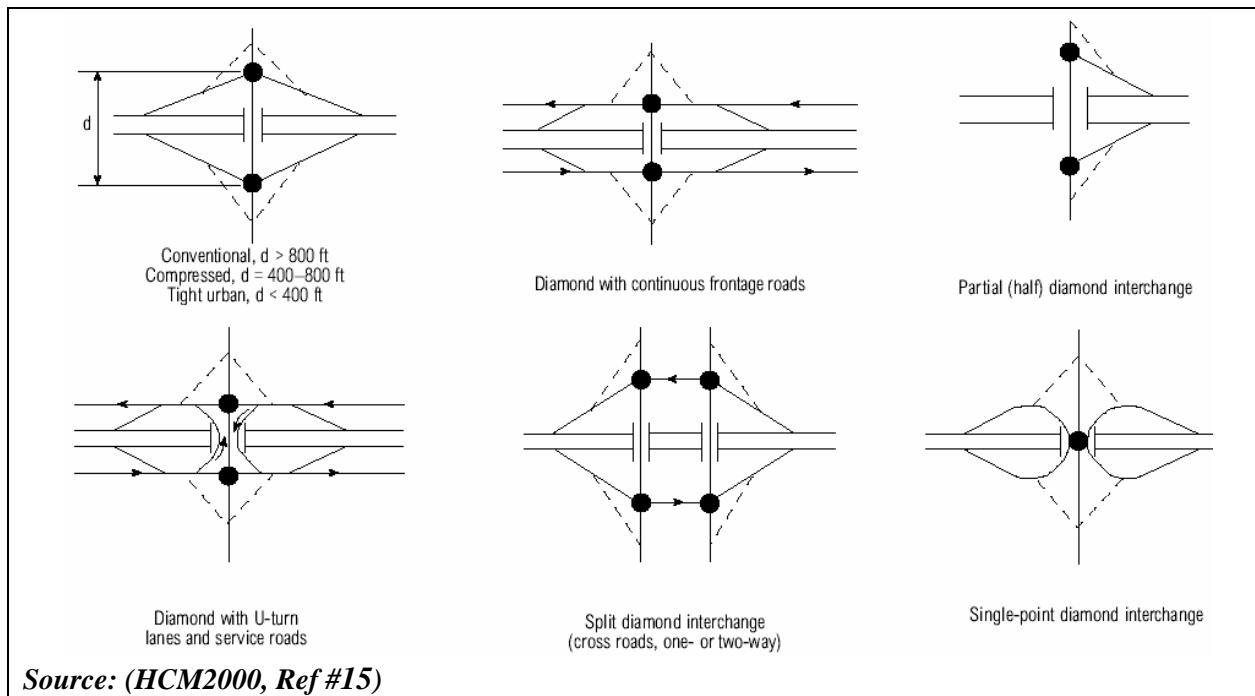


Source: (TRB, 2000, Ref. #3)

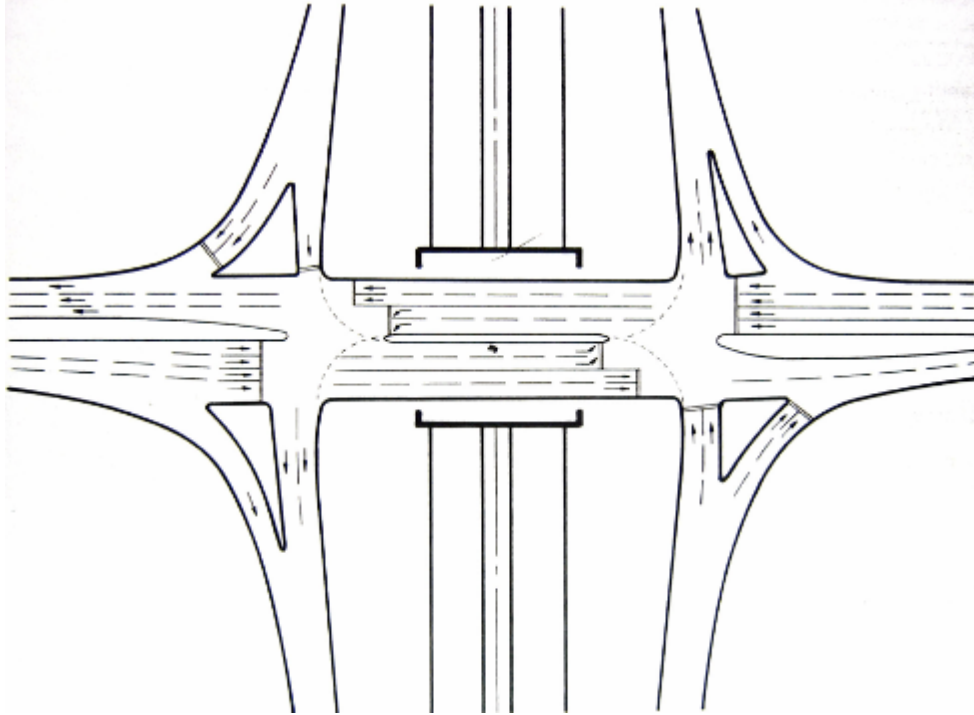
**Figure 6: Illustration of minimum spacing for freeway interchange areas with multilane crossroads**

## Chapter 3: Diamond Interchange Design Issues and their Impact on Access Management

There are several types of service interchanges, but the diamond interchange is the most common type of interchange in the United States. It consists of four legs and two intersections that, for heavier traffic volumes, require signalization and coordination for efficient traffic operations. Figure 7 shows the different types of diamond interchanges. Some state DOTs prefer using a tight diamond interchange (TDI), Figure 8, or a single point urban interchange (SPUI), Figure 9, design for situations requiring minimal right of way. When arterial weaving distance in terms of access management is a concern, some state DOTs have taken advantage of the minimal right of way requirements of the TDI and the SPUI to increase weaving distances.

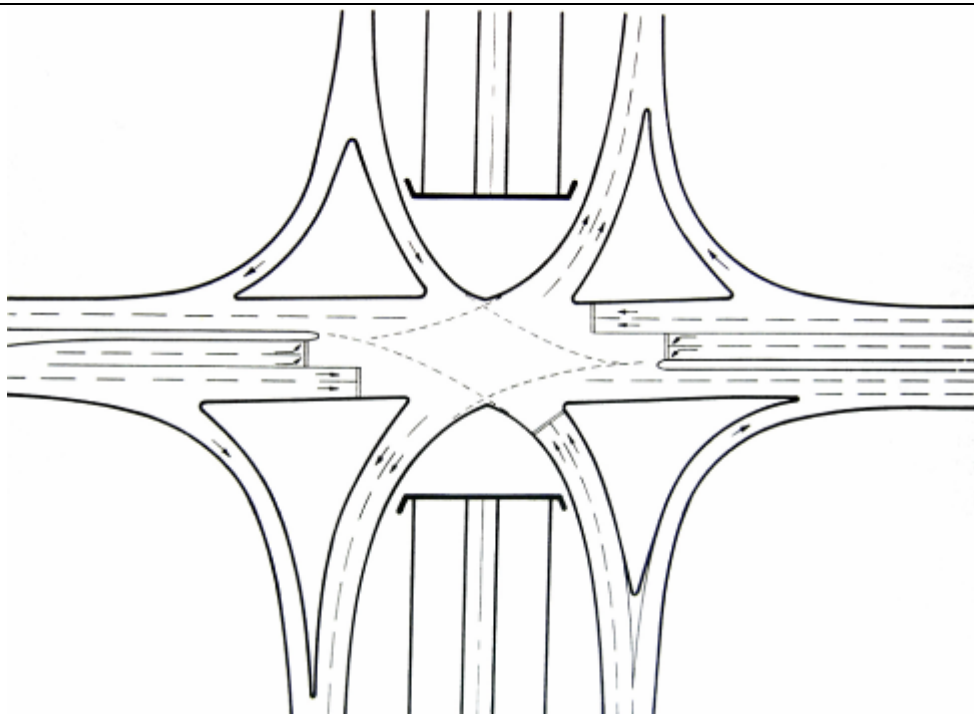


**Figure 7: Types of Diamond Interchanges**



*Source: (Leisch et al., 1989, Ref. #16)*

**Figure 8: Tight Diamond Interchange**



*Source: (Leisch et al., 1989, Ref. #16)*

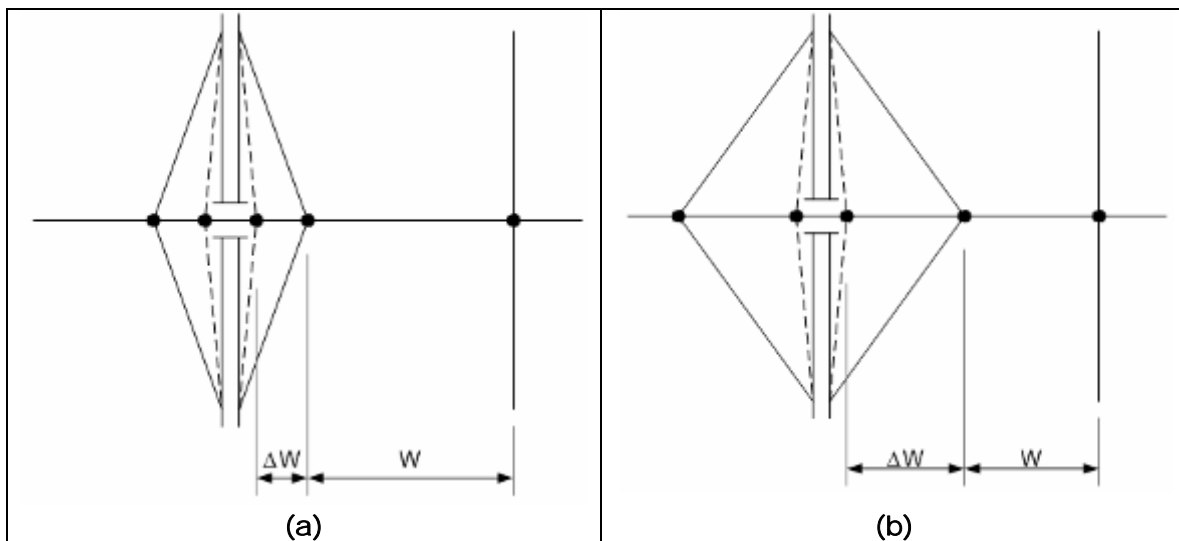
**Figure 9: Single Point Urban Interchange**

Guidelines for the spacing of driveways and signalized intersections are helpful during new construction of interchanges. Once an interchange has already been constructed, however, the access spacing guidelines are only helpful to determine if the current access spacing is inadequate. In many cases the distance to the nearest major intersection from the interchange off-ramp is less than adequate and some state DOTs have resorted to modifying the interchange in addition to access management strategies along the arterial to help improve traffic operations caused by inadequate access management. In cases where there is a conventional diamond interchange, some state agencies have resorted to bringing the ramps in closer to the freeway mainline, creating a TDI. This provides the drivers making a right turn from the off-ramp and a left turn at the downstream intersection a longer distance to weave into the left turn lane.

There are, however, some tradeoffs when modifying the interchange. The internal link spacing is the distance between the two intersections created by the four legs on a diamond interchange shown in the upper left of Figure 7 as the variable  $d$ . By decreasing the internal link spacing, there is a gain of longer weaving distances, but in turn the reduced storage distance between the two intersections may result in several operational problems, such as inadequate internal storage, reduced bandwidths, poor progression, and increased delay. Also, intersections spaced very closely will require complex signal operations. In cases where the interchange is not a diamond, but rather a full or partial cloverleaf, or even a multilevel interchange, modifying the interchange to a SPUI or a TDI is not an option. The SPUI or TDI cannot handle the volume levels that a multilevel interchange and a full or partial cloverleaf can handle. Once an interchange is converted to a TDI, the loss of land between the freeway mainline and ramps precludes the possibility of later adding any loop ramps to accommodate heavy left turn volumes.

By bringing the interchange ramps in closer to freeway mainline, the potential gain in weaving distance would depend upon the situation. Figure 10 illustrates the potential gain in weaving distance for two situations, where  $W$  is the weaving distance,  $\Delta W$  is the potential gain in weaving distance, and the dashed line is the TDI. If a conventional diamond interchange with an internal link spacing of about 800 ft were to be modified to a TDI or a SPUI with an internal spacing of about 400 ft, there is potentially a gain of about 200 ft or more of additional weaving distance on either side. In situations where the nearest intersection is already located too close to the off-ramps, 200 ft is not much of an increase in the weaving distance, shown in

Figure 10a, considering the cost of interchange modifications. However, in other situations where the internal link spacing of a conventional diamond interchange is spaced far apart, a conversion to a TDI or SPUI could significantly increase the weaving distance on either side shown in Figure 10b. An interchange modification could be a worthwhile expense depending upon the current length of the internal link spacing and the distance to the nearest major intersection.



**Figure 10: Potential gain in weaving distance (a) small and (b) large**

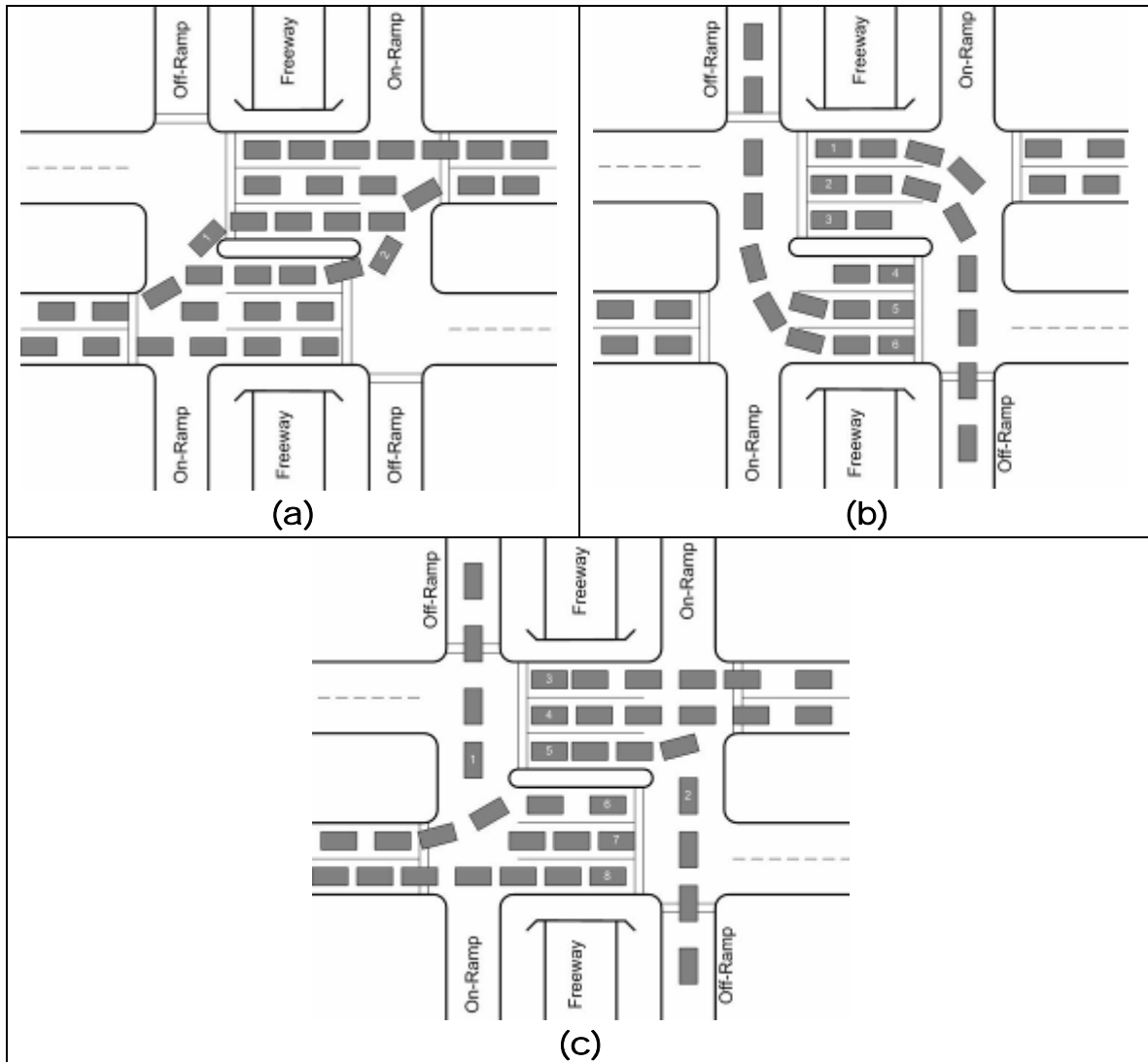
As defined in the Highway Capacity Manual (15), a conventional diamond interchange has ramps that are spaced greater than 800 ft apart and a tight diamond has ramps spaced less than 400 ft apart. Typically the internal link spacing for a TDI is between 200 ft and 400 ft. Depending upon certain factors (e.g., internal link spacing, average vehicle length, inter-vehicle spacing, and stop bar setback) there would be approximately an available storage of 5 vehicles before the queue extends into the upstream intersection.

There are many issues to consider when deciding to modify a diamond interchange. A tight diamond configuration will require very complex signal operations due to the significant decrease in internal queue storage. An improper signal timing sequence and coordination plan can lead to poor progression of traffic, which can lead to excessive queuing. There are some situations where the short storage distance can cause queues along the arterial to extend beyond the storage length and block the upstream intersections, also known as spillback, shown in Figure 11. Spillback is defined as full or partial blockage of an intersection by one or more



vehicles that do not make it through the downstream intersection during the green time for that phase. The next arriving vehicle may not react in time to stop before the intersection is blocked because the vehicle arrived on a green light and most drivers are not likely to stop on a green light. Gridlock is essentially a worse case scenario, but it is more likely with the shorter internal link spacing because of the complications in signal timing and the need for short cycle lengths to keep the red times for the internal link movements at a minimum. The different scenarios where spillback can cause gridlock and vehicles essentially end up blocking each other are described below.

- Figure 11a illustrates how the left turn movements from the arterial onto the freeway on-ramps can block each other. If the internal queues between the ramps extend upstream to the next intersection and block it, the left turn movements will not be able to move. The ability of the first left turning vehicle, vehicle 1, to advance is dependent on a gap between the queues blocking the intersection. However for this gap to be created, the first vehicle in that queue, 2, must advance, but cannot because this driver is in the same situation as vehicle 1.
- Figure 11b shows what can happen with a short storage distance to the left turning vehicles coming from the off-ramps. As the vehicles on the off-ramp turn left and accumulate on the main street, the queue can extend into the intersection and block the arterial once the light turns green for the arterial movements. Again a gridlock situation occurs as vehicles 1-3 cannot advance unless a gap is created, but the gap is dependent upon the first vehicles in that queue, 4-6, that also cannot advance.
- Figure 11c shows how spillback along the arterial can block the off-ramp left turn movements. Even if the green time on the off-ramps is enough for the demand, spillback would allow only a few vehicles to advance, depending upon street width, and those vehicles would block the intersection. Again, a gridlock situation can occur because a gap must be created to allow the vehicles from the off-ramp, 1 and 2, to enter the arterial, but the gap is dependent upon the first vehicle in the queue which is also blocked by the vehicles that cannot enter the arterial on the other side of the freeway, 3-5 and 6-8.

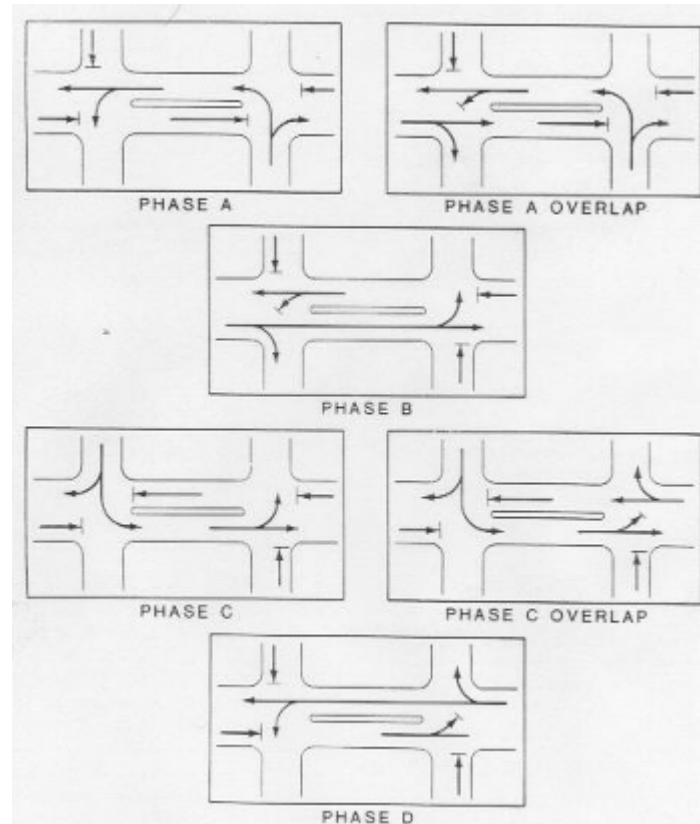


**Figure 11: Gridlock caused by a short internal storage length and improper signal coordination for (a) arterial left turn lanes, (b) off-ramp left turns, and (c) a combination of both**

A SPUI, seen in Figure 9, is an alternative to a TDI that consolidates the on-ramps and off-ramps of a diamond interchange into a single intersection. This eliminates the possibility of the internal gridlock situation, discussed earlier, caused by the short internal storage length. A SPUI also eliminates the issue of complex coordination between the two closely spaced signals. Choosing which type (SPUI or TDI) to build in a particular situation can be challenging as it depends on several factors and both have advantages and disadvantages. The following paragraphs summarize previous research that compared the two interchange forms. Some recommend the use of a SPUI while others recommend the use of the TDI.

Research by Leisch et al. (16) highlighted some differences between the two interchange forms, SPUI and TDI, which should be considered when deciding between the two interchange forms.

- Due to the large open pavement area for the single intersection, a SPUI would require a clearance interval of about two seconds larger. The SPUI has clearance distances approaching 250 ft compared to a TDI at about 125 ft. This causes the SPUI to have a greater lost time per phase than the TDI. However, depending upon the number of phases and cycle length, the proportion of total lost time during the cycle may not be larger.
- A SPUI typically has a three phase cycle and a TDI has a four phase overlap cycle that essentially operates as one cycle, shown in Figure 12. Typically three-phase operations perform slightly better than four-phase operations because there is one less phase to incur lost time.
- Using TRANSYT-7F as an analysis tool, a TDI was found to require shorter cycle lengths than a SPUI. This is likely a result of having to clear the internal queues more frequently to prevent upstream intersection spillback.
- Since the SPUI has a larger turning radius for left turns, the saturation flow rates are similar to those of the through movements and are only reduced by 5% to 15% depending upon flow conditions.
- The construction costs are much higher on the SPUI compared to the TDI because a SPUI requires a longer bridge span, larger bridge deck, higher retaining walls, and more earthwork. Typically, construction costs for a SPUI are about \$1 million to \$2 million more and sometimes there are cases where it is \$4 million more than a TDI.
- Leisch et al. concluded that TDIs are more efficient than SPUIs for most traffic volumes except when off-ramp left turn volumes are high compared to traffic on the crossroad.



*Source: (Leisch et al., 1989, Ref. #16)*

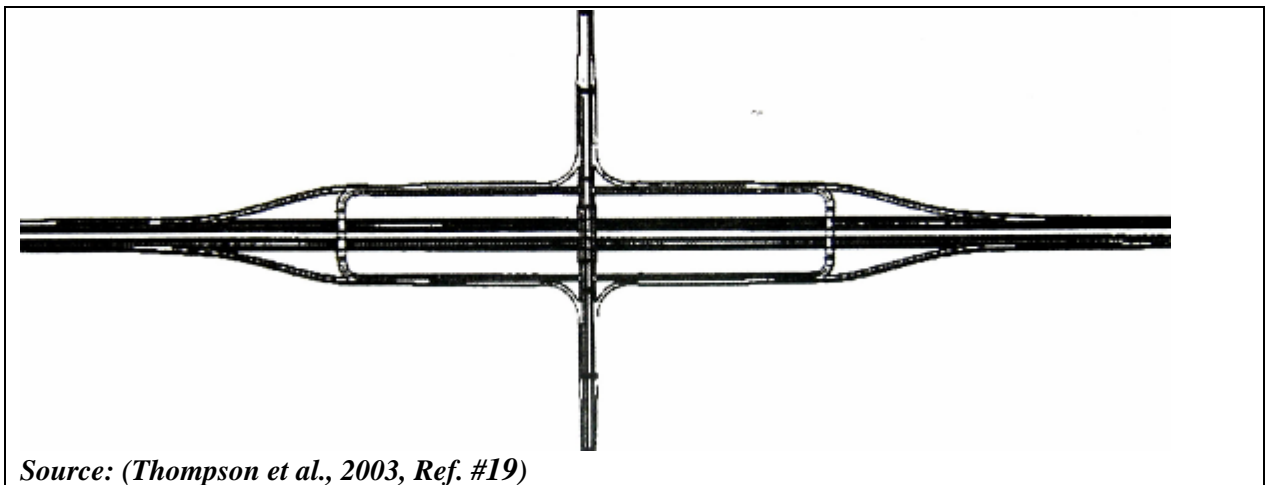
**Figure 12: Signal Phasing For TDI**

Research by Jones et al. (17) compared the TDI and SPUI designs using computer analysis methods. They used a variety of software to compare operations between the two interchange forms, including Synchro 5.0, PASSER III-98, and CORSIM. The SPUI was found to have higher average travel speeds, a lower percentage of stops, and higher capacity than a tight diamond. They also found that with the same volume levels, a TDI could reach capacity while a SPUI would still be under capacity and operating at moderate conditions.

Bared et al. (18) performed traffic simulation analysis comparing TDI and SPUI designs to create planning models for estimating operational parameters of the SPUI and TDI. It was concluded that if the left-turn volumes from the off-ramp are less than 900 veh/hr, delay and stops are not significantly different between the two interchange forms. However, if the left-turn volumes are higher than 900 veh/hr, it was found that a TDI has significantly higher delay and a higher percentage of stops due to the two intersections instead of a single intersection.

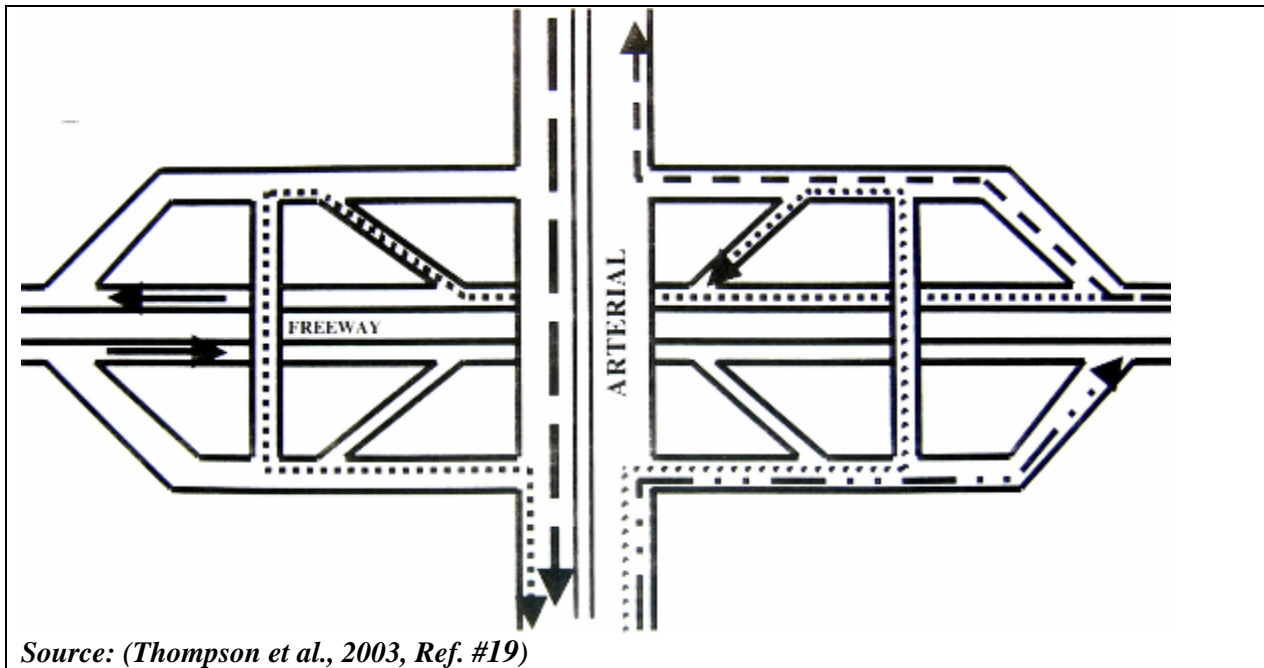
There are many other types of interchanges that are not as commonly used but, have close ramp spacing. The Michigan urban diamond interchange (MUDI), Figure 13, and the W-

interchange, Figure 14, also have two closely spaced intersections, but these two interchange forms do not require traffic signals because the on-ramps and off-ramps are right turn only. These interchange forms handle high volume situations very well and are a possible modification to full or partial cloverleaf interchange designs. Thomson et al. (19) compared the W-Interchange with the other conventional interchange forms that require minimal right of way. The MUDI and the W-interchange are more expensive than TDIs and SPUIs, but offer better traffic operations due to the free-flow on-ramps and off-ramps. The MUDI is a combination of a diamond interchange and Michigan's median U-turn intersection design, thereby removing left turns from the interchange by rerouting them to directional crossovers so they may turn right instead. However, this rather complicated design requires four crossover bridges in addition to the main freeway overpass. The W-interchange is similar to the MUDI in that it removes left turns by rerouting them, but only requires two crossover bridges by relocating the locations of some on and off ramps to reduce the need of two crossover bridges, shown in Figure 14. These two interchange types are new designs and rather expensive to construct, but operate significantly more efficiently and have several advantages over the other interchange designs. These interchange designs should only be considered in very high volume situations due to the high construction costs.



*Source: (Thompson et al., 2003, Ref. #19)*

**Figure 13: Michigan Urban Diamond Interchange**



*Source: (Thompson et al., 2003, Ref. #19)*

**Figure 14: W-Interchange**

## **Chapter 4: Modeling and Analysis Methods for Interchange Areas**

The use of computer modeling for roadway design and traffic operations is a growing trend among practitioners. Whether it is stochastic time-based modeling of individual vehicles or is based on analytical methods, computer modeling provides practitioners feedback on the efficiency of the operations of their design before implementing in the field. Computer modeling is not only far more efficient than performing calculations manually, for complex systems it may be the only feasible method for solving the problem. Analytical techniques are based primarily on mathematical formulas. A number of analytical methods for estimating a wide array of performance measures for a variety of transportation facilities are contained in the Highway Capacity Manual. This chapter provides an overview of different simulation programs and analytical methods that can be applied to the analysis of arterial operations in interchange areas.

### **Simulation Methods**

The analysis of arterial operations in an interchange area is one of the most complex traffic operations analysis situations in the field of traffic engineering. With so many factors, and their various interactions, influencing traffic operations for these facilities, a simulation approach is often the only method that will allow one to investigate the operations issues in sufficient detail. Computer models can be used for a comprehensive analysis of interchange design as well as the access management of the nearby driveways.

There are two commonly used types of simulation programs: microscopic and macroscopic. Microscopic programs model individual vehicles and are typically more computationally intensive. Macroscopic programs work with aggregate measures of traffic flow measures, such as flow, speed, and density, which improves computational performance, but reduces the detail of representation.

There are several simulation programs that can be used to analyze access management alternatives and interchange design features. The following gives an overview of the more popular simulation/computer analysis programs and their applicability to arterial access management and interchange design.

- **CORSIM (CORridor SIMulation):** Developed by the Federal Highway Administration (20), CORSIM is a time-based, stochastic, microscopic simulation program. This program has been one of the most extensively used and tested in the United States. As with all microscopic simulation programs, this programs requires a large number of input parameters to be specified. The traffic operations can be animated (through the companion TRAFVU application) and the output can be post-processed to measure any operational parameter of interest. Previous studies in access management performed by Tindale and Coxen (21) used CORSIM to make determinations for access controls as well as to give an understanding of the issues related to queuing, access, and circulation within the site. Research mentioned earlier by Jones et al. (17) and Bared et al. (18) used CORSIM to analyze and compare a TDI and SPUI operations.
- **VISSIM:** Developed by a German Company called Planung Transport Verkehr (PTV), VISSIM is distributed in Corvallis, Oregon by Innovative Transportation Concepts (ITC). VISSIM is a microscopic behavior-based traffic simulator and it has a variety of animation capabilities such as providing 2D and 3D visualization of the network. VISSIM can simulate a wide variety of geometric and operational conditions such as interchanges, and merging and weaving areas. The simulator can be used for a series of applications; however it is more data intensive than CORSIM. Previous studies in access management by Dale and Woody (22) used VISSIM as a tool to compare operations with and without access management control as well as illustrate some safety issues.
- **Synchro/SimTraffic:** Synchro/SimTraffic was developed by Trafficware Corporation. Synchro is a macroscopic simulation program and SimTraffic is a companion microscopic simulation and traffic animation program. Synchro is capable of optimizing cycle lengths, phase splits, and offsets. It is also can provide detailed time-space diagrams for assessing green bands and progression quality. Synchro performs capacity analysis according to the HCM2000 methodologies for signalized



and unsignalized intersections. Version 6 added the ability to implement curved links, making it easier to represent interchanges with loop ramps.

- **PASSER III-98 (Progression Analysis and Signal Systems Evaluation Routine):**

PASSER III-98 was developed by the Texas Transportation Institute. It is a deterministic, macroscopic traffic modeling software program developed specifically for diamond interchange signal optimization. For example, one of its strengths is in identifying the optimal internal offset between the two signals. A major limitation of this software, however, is that it does not take into account the impacts of queue buildups that can spill back into upstream intersections as it assumes vertical queuing. It has an automated report generator useful for comparing results.

- **TRANSYT-7F:** TRANSYT-7F is a deterministic, macroscopic simulation program used to optimize traffic signal timing for traffic networks, arterials, or complex intersections. TRANSYT-7F is capable of optimizing cycle lengths, phase splits, and offsets. It offers a tremendous amount of flexibility in specifying the objective function for optimization. Release 10 offers some enhanced features such as interaction with CORSIM for optimization and traffic animation. A wide variety of lane configurations and timing plans can be accommodated in TRANSYT-7F, giving it the flexibility to handle a wide variety of signalized interchange configurations. Several studies (e.g., 15, 16, 17) have used TRANSYT-7F to compare SPUI and TDI interchange designs.

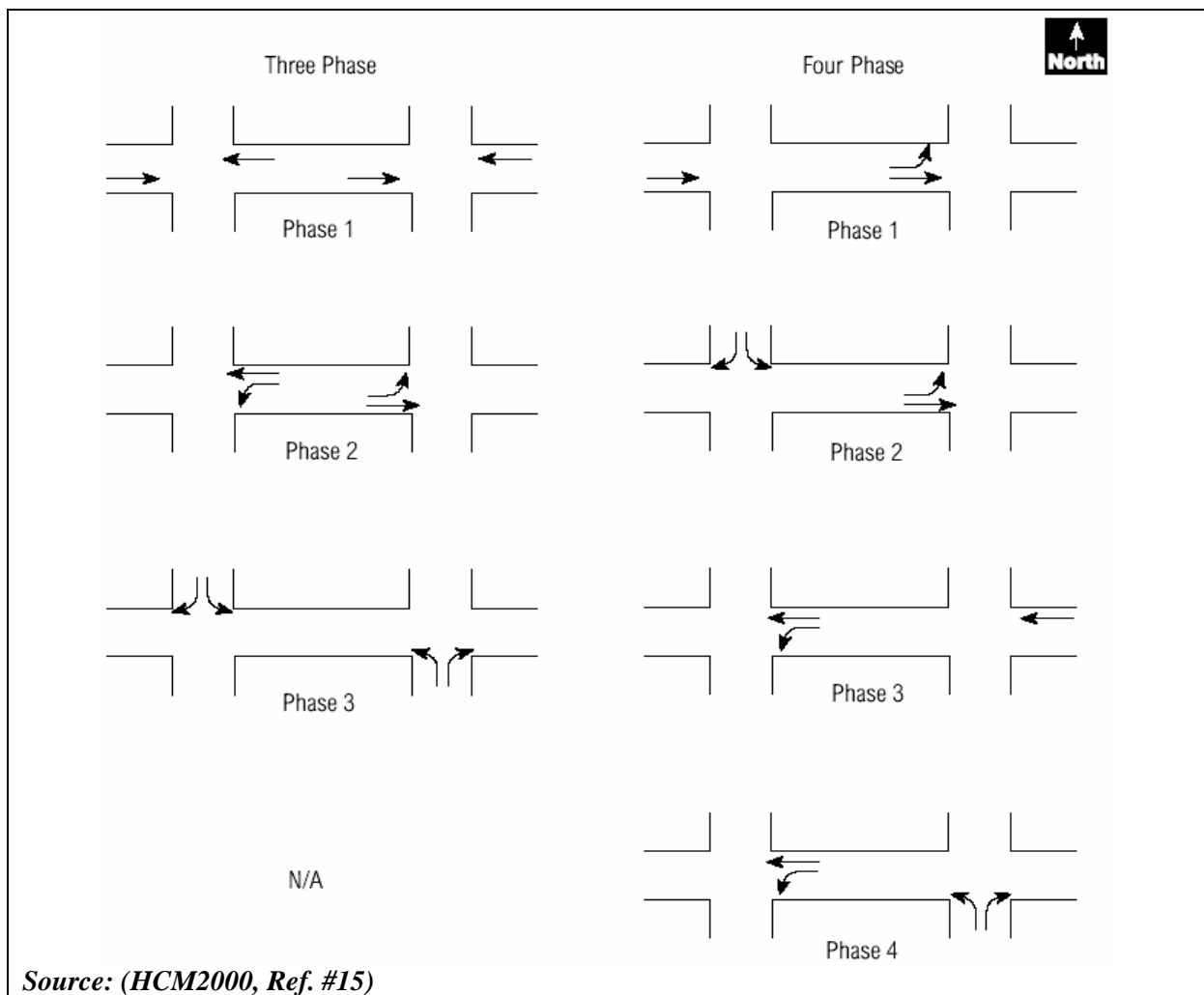
## **Analytical Methods**

Current analytical methods for the analysis of interchange operations are rather limited. The HCM2000 is still probably one of the best sources, but its methods are still inadequate for a detailed operational analysis of the cross-street arterial operations.

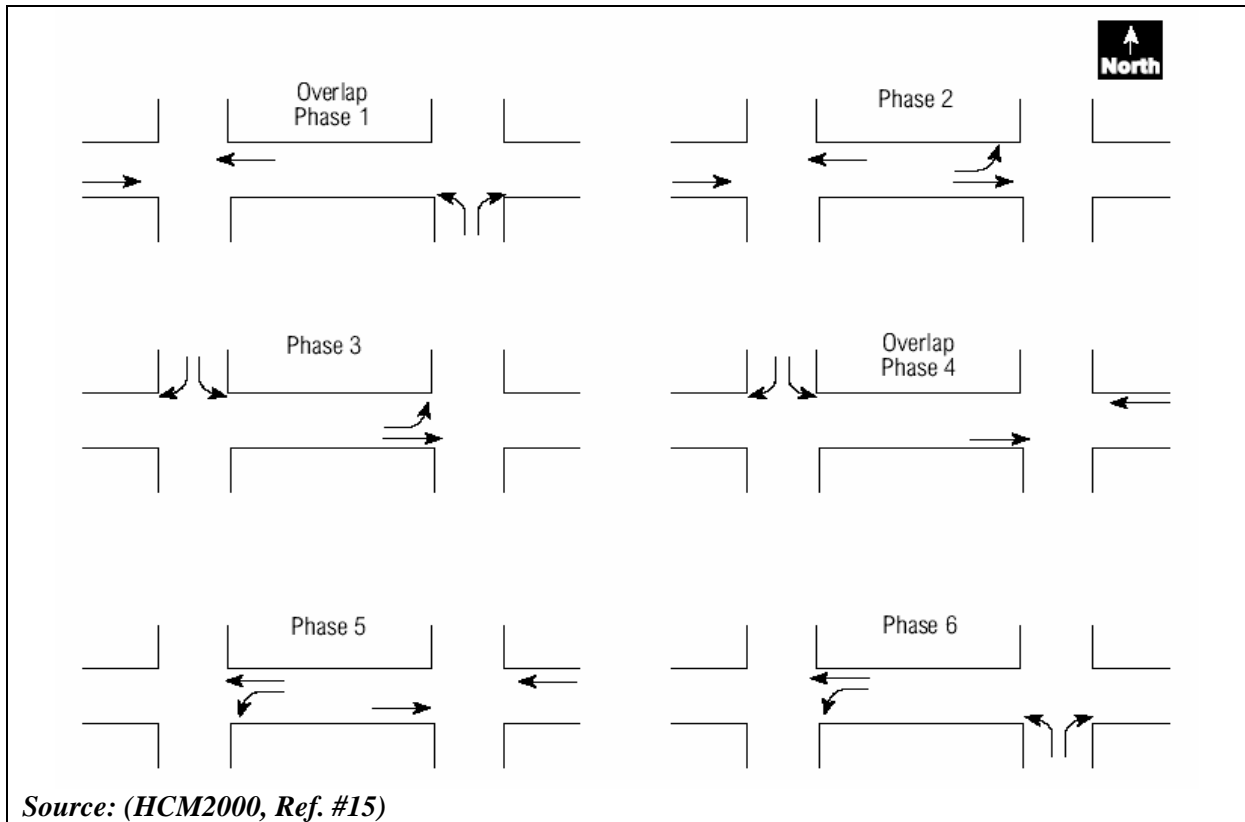
Current guidance in the HCM2000 for interchange analysis is contained in Chapter 26 (15). The chapter primarily focuses on two-intersection diamond interchanges, but is conceptual in content. This chapter is very general and does not focus on the impact of the interactions between the two intersections. The impacts of access management issues in the

vicinity of an interchange are also not addressed. The current procedure essentially aggregates the signal delay estimates at the individual ramp intersections and then uses this aggregate delay value as a basis for the interchange level of service.

Chapter 26 also provides signal phasing and timing strategies for closely spaced intersections such as the ones from diamond interchange ramps. Figure 15 and Figure 16 show the recommended phasing sequence for the closely spaced intersections of a diamond interchanges that can be applied for use on any two closely spaced intersections.



**Figure 15: Three and Four Phase Plan**



Source: (HCM2000, Ref. #15)

**Figure 16: Four Phase Plan with Overlap**

A recently completed NCHRP project (3-60) (23), performed by a research team led by Dr. Lily Elefteriadou from the University of Florida resulted in the development of a more comprehensive interchange analysis method. This new methodology will ultimately replace the current one in Chapter 26 of the HCM2000. A brief overview of the new methodology is as follows.

This NCHRP project was focused on the development of improved methods for capacity and quality of service analysis of different types of interchange configurations, such as diamond, cloverleaf and SPUI. The methodology considers both planning and preliminary design as well as final design and operational analysis. The main focus of the research is on the surface streets. The methodology can account for the expected queue length on the ramps; however it cannot evaluate its impact on freeway operations. Additionally, the methodology can estimate the operational effects of two closely-spaced signalized intersections (such as in diamonds and two-quadrant partial cloverleafs) and the impact of the downstream queued vehicles on the upstream intersection; however, it does not consider the effect of a downstream closely-spaced intersection on the interchange operations.

The final design and operational analysis begins with the collection of all pertinent input information such as geometry, traffic demands and signalization information. An important feature of the analysis is the developed lane utilization models. Depending on the interchange type, the lane utilization model adjustment factors account for the uneven distribution of volume among lanes of the same lane group on the upstream approaches. This unbalanced lane distribution is typically more severe than at isolated intersections, due to the generally high turning movements at interchange areas.

The effect of a downstream (internal link) queue on the upstream approaches (both surface street and ramps) is considered by estimating the additional lost time experienced at the upstream intersection (external link). The methodology developed considers the duration of overlaps between the phases of the two intersections, as well as the estimated queue lengths on the internal links.

The lost time incurred on the downstream (internal) link is expressed in terms of the demand starvation, which accounts for the amount of green time during which there is no queue present to be discharged from the internal link and there are no arrivals from either of the upstream approaches due to signalization. The methodology goes on to compute the adjusted effective green times for all approaches by including the lost times incurred for both internal and external approaches.

The next step of the methodology is the determination of the level of service by estimating the queue storage ratio, the degree of saturation and the control delays. If for any given lane group the queue storage ratio and/or the degree of saturation exceed 1.0, then the LOS for every origin-destination (OD) pair which travels through this particular lane group will be F. The methodology defines the LOS for each OD movement as the total average control delay experienced by the appropriate demands that travel through the interchange. The LOS criteria for each OD of an interchange are greater than those for an individual signalized intersection to reflect the fact that some movements travel through two intersections.

Also, the authors recommend the use of simulation tools for the analysis of complex situations at interchanges, such as impacts of interchange operations on the surrounding streets, the oversaturated conditions with interacting queues between two intersections.

## Chapter 5: Signal Spacing Considerations for Progression Quality

Signal progression is one of the most significant concerns for arterial operations. Signal progression (or sometimes called coordination) directly affects the percentage of vehicles that arrive during the green indication, which directly affects the progression adjustment factor (*PF*) term in the signal delay equation from the Highway Capacity Manual (HCM) (15). The *PF* is multiplied directly with the uniform delay component of the overall signal delay equation (16-9). A paper by Washburn et al. (24) describes the impact of progression quality on arterial performance in more detail.

If unsignalized driveways are placed near an interchange, it is very important to consider the spacing of those driveways in the context of signal coordination as well, as it is possible that they may need to be signalized in the future due to development trends.

The following section describes the fundamental relationship between signal spacing, vehicle speed, and cycle length

### Fundamental Relationship between Signal Spacing, Average Vehicle Speed, and Cycle Length

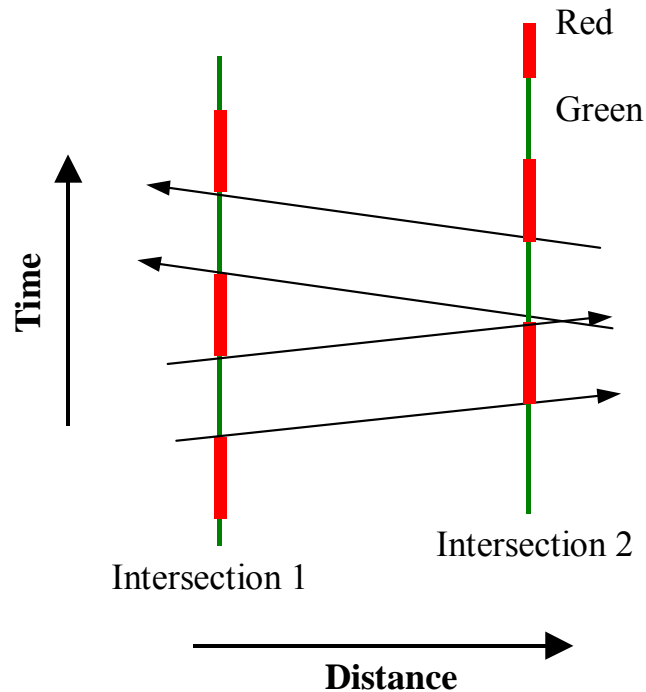
The relationship between signal spacing and vehicle speed is most easily illustrated by considering a one-way arterial. For two intersections separated by a certain distance, the green offset at the downstream intersection relative to the upstream intersection should just be equal to the travel time between the two intersections. Consider the following example:

- Two intersections on a one-way arterial are separated by 880 ft. The average running speed of vehicles along this segment is 30 mi/h. Thus, the green offset between the two intersections is calculated as follows:

$$\frac{880 \text{ ft}}{30 \text{ mi/hr} \times \frac{5280 \text{ ft/mi}}{3600 \text{ sec/hr}}} = 20 \text{ seconds} \quad [3]$$

However, for an arterial with traffic in both directions, the setting of the offset for ideal progression in one direction may lead to very poor progression in the other direction. Figure 17 (a time-space diagram) illustrates a situation in which the offset has been set to

obtain ideal progression from intersection 2 to intersection 1. However, this situation also results in very poor progression in the direction of intersection 1 to intersection 2. This scenario assumes the same cycle length and g/C ratios at both intersections.



**Figure 17. Time-Space Diagram Illustrating Good Progression for Only One Direction**

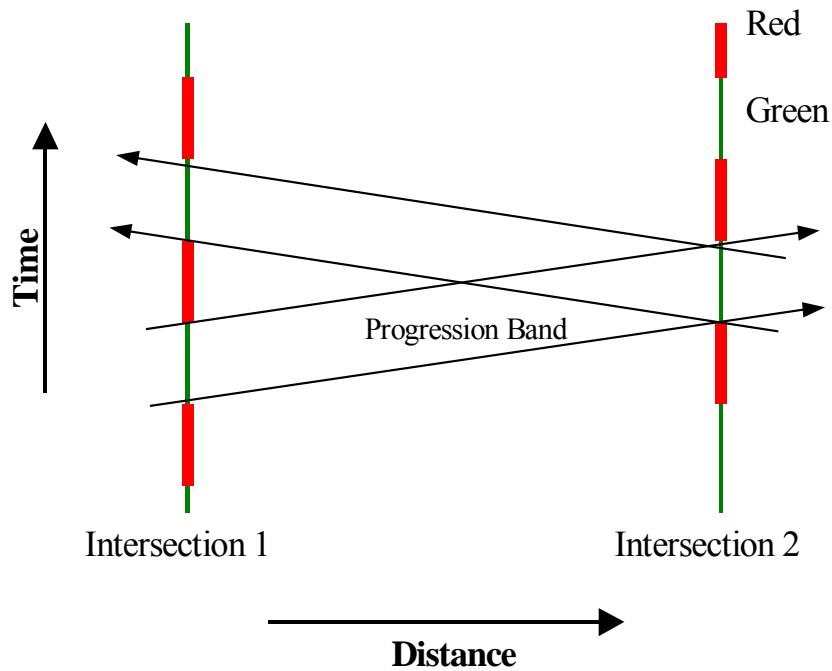
To obtain ideal progression for the above scenario in both directions, the cycle length must be factored into the equation. For ideal progression in both directions, the cycle length (for both intersections) should be twice the travel time from Intersection 1 to Intersection 2. Consider another example:

- Two intersections on a two-way arterial are separated by 2640 ft (1/2 mile). The average running speed of vehicles along this segment is of 40 mi/h (58.7 ft/sec).

The travel time between the two intersections is  $\frac{2640 \text{ ft}}{58.7 \text{ ft/sec}} = 45 \text{ seconds}$ . Therefore,

the necessary cycle length to obtain ideal progression in both directions is 90 seconds (45 x 2).

This situation is illustrated in Figure 18.



**Figure 18. Time-Space Diagram Illustrating Good Progression for Both Directions**

The progression band, indicated by the double arrows, shows the direction of traffic flow. Every vehicle that gets through the green at the first intersection will also make it through the green at the second intersection. In the other direction, every vehicle that gets through the green at the second intersection will also make it through the green at the first intersection. Thus, the progression is ideal in both directions of traffic flow. When calculating offsets, the progression bands should be as close to the available green as possible in order to obtain the best progression.

Table 10 provides the required signal spacing to provide for ideal two-way progression. The values in this table are based on an assumed  $g/C$  ratio of 0.5.

**Table 10. Required Signal Spacing for Given Travel Speed and Cycle Length (in feet)**

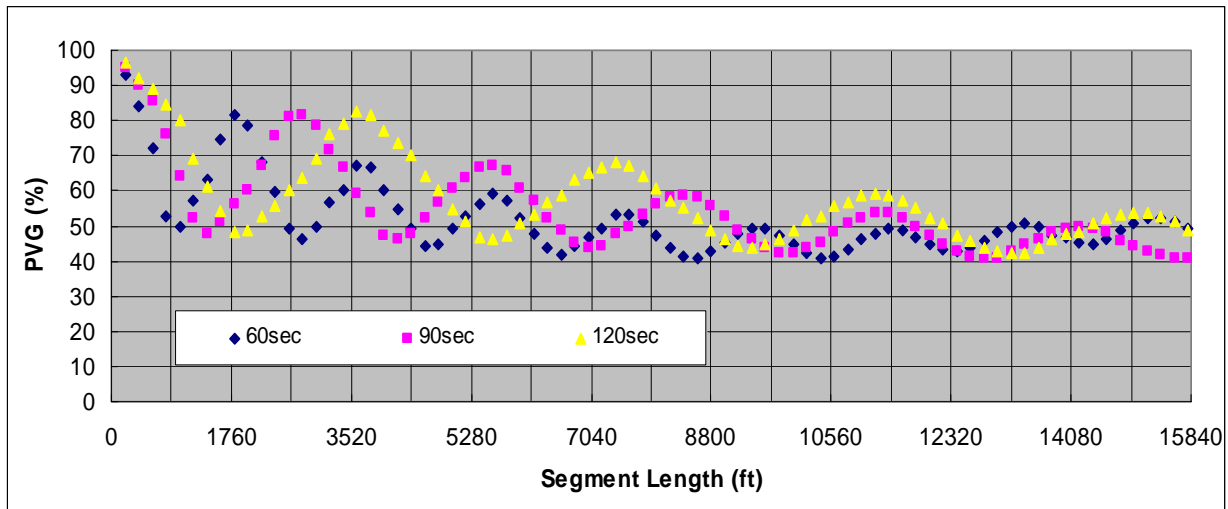
Cycle Length (sec)	Speed (mi/h)						
	25	30	35	40	45	50	55
60	1100	1320	1540	1760	1980	2201	2421
75	1375	1650	1925	2201	2476	2751	3026
90	1650	1980	2311	2641	2971	3301	3631
105	1925	2311	2696	3081	3466	3851	4236
120	2201	2641	3081	3521	3961	4401	4841
135	2476	2971	3466	3961	4456	4951	5446
150	2751	3301	3851	4401	4951	5501	6051
165	3026	3631	4236	4841	5446	6051	6657
180	3301	3961	4621	5281	5941	6602	7262

As can be seen in the table, as speed and/or cycle length increases, the distance required for ideal two-way progression also increases. As it relates to this project, it is useful to consider these spacing values in the context of different arterial development scenarios, such as low, medium, and high.

For low development conditions, such as in rural areas, the traffic volumes are relatively low, cycle lengths are usually between 60 and 90 seconds, and travel speeds range from 45 to 55 mi/h. For these conditions, longer signal spacing is required for two-way progression, in the range of 2000 to 3600 ft. For medium development conditions, traffic volumes are moderate, cycle lengths range from 90-150 seconds, and travel speeds vary from 35 to 45 mi/h. For these conditions, ideal signal spacing for progression ranges from 2300 to 5000 ft. For high development conditions, traffic volumes are high, cycle lengths vary from 150 to 180 seconds, and travel speeds range from 25 to 35 mi/h (except under saturated conditions when travel speeds can become very low). Under these conditions, a signal spacing anywhere between 2700 and 4600 ft may be necessary.

Figure 19 shows some sample theoretical curves for Percent Vehicles arriving on Green (PVG) based on the signal spacing, vehicle speed, and cycle length relationship shown above for two-way arterials. The 60-second, 90-second, and 120-second cycle lengths use a signal spacing of 1760 ft, 2640 ft, and 3520 ft, respectively. The graphed PVG values represent the average of the peak and off-peak directional PVG values. The sinusoidal nature of the curves is a function of high average PVG values resulting from ideal spacing of intersections and low average PVG values resulting from worst-case spacing of intersections.





**Figure 19. Percent of Vehicles Arriving on Green (PVG) for Various Cycle Lengths**

When planning new construction, this relationship can help guide the location of new signalized intersections. For an analysis of an existing arterial, however, the objective is to usually find a common cycle length and offset values that produce the best progression through the existing intersections.

### **Other Factors Influencing Progression Quality**

#### Effective Green / Cycle Length Ratio ( $g/C$ )

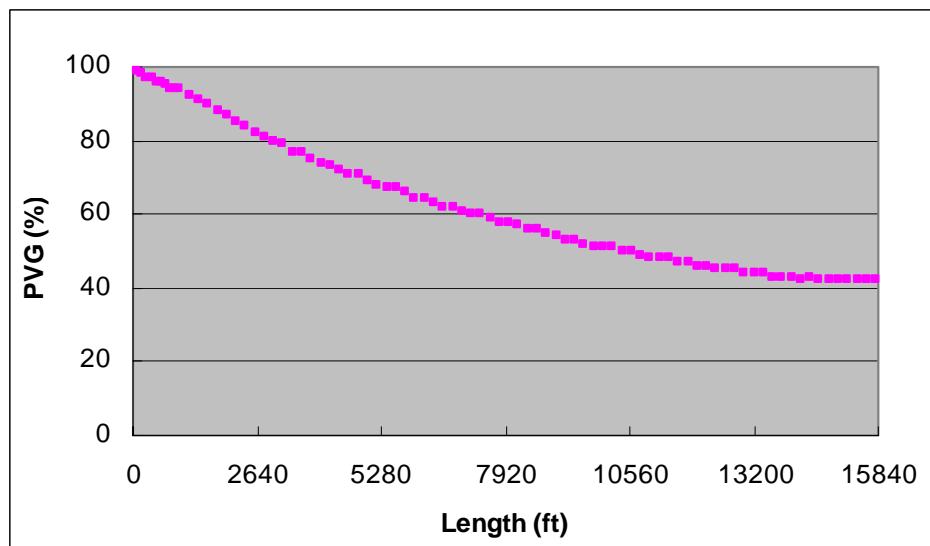
The influence of the  $g/C$  ratio is straightforward. To illustrate, two hypothetical examples will be discussed. First, consider an intersection approach that actually had a 100%  $g/C$  ratio (i.e., constant green). For this situation, the PVG would also be 100% because there would never be a red indication for which to stop. Next consider the opposite case, where an approach has a  $g/C$  ratio of 0% (i.e., constant red). In this case the PVG would also be 0%, because there would never be a green indication for vehicle arrivals. Thus, for the case of uniform arrivals, the PVG will be equal to the  $g/C$  ratio. Of course, for many situations, a condition of truly uniform arrivals does not exist. However, as will be discussed for the platoon dispersion variable, the  $g/C$  ratio can serve as a boundary condition for certain situations.

#### Platoon Dispersion

When queued vehicles depart an intersection after the start of a green phase, they are usually closely spaced. These closely spaced groupings of vehicles are referred to as

platoons. One of the goals of signal coordination is to maintain these platoons of vehicles and allow them to arrive at successive downstream intersections on the green. However, as the platoons progress along the length of an arterial between signals, individual drivers within these platoons will begin to adjust their speeds, and the platoon will begin to spread (i.e., disperse). The more length of roadway between signals, the more pronounced this dispersion will become, eventually reaching a point at which the flow of traffic along the arterial will become more random or even uniform in nature. Thus, platoon dispersion is primarily a function of roadway length between signals. However, the character of the development surrounding the arterial will also have an effect on platoon dispersion, as intersecting driveways, presence of curbside activities (e.g., parking, bus stops, etc) will also increase traffic “friction” and have a platoon dispersing effect.

Hillier and Rothery (25) performed empirical studies of platoon dispersion. The platoon dispersion model developed by these researchers has been incorporated into the TRANSYT-7F simulation program. This model follows a negative exponential form, an example of which is shown in Figure 20. Note how the function appears to level off just above 40 percent. This corresponds to the  $g/C$  ratio of 0.4 used in this experiment, which is the expected PVG value when the traffic arrival flow pattern begins to approximate a uniform pattern.



**Figure 20. Percent of Vehicles Arriving on Green (PVG) for One-way Street (D Factor = 1.0)**

### Percent Turns from Exclusive Lanes (PTXL)

From a strictly theoretical perspective, the percentage of turns from exclusive lanes (predominantly left turn lanes) should not impact the PVG for the through vehicles. However, from a practical perspective, an approach that has a significant percentage of turns onto the cross street is likely to have a significant percentage of traffic turning onto the major street from the cross street. And since the arterial is likely timed for progression of the major street through movements, the traffic coming from the cross street is likely to arrive during the red of the downstream intersection. Thus, this will lead to an overall lowering of the PVG for the downstream approach. Mid-block entry points (e.g., driveways) can also have a similar effect on progression quality. That is, many vehicles turning onto the arterial from a mid-block driveway may not be able to join the progression band, and may even negatively impact the ability of the through vehicles to stay in the progression band.

### Directional Distribution (D-factor)

The distribution of traffic in the peak and off-peak directions plays a role in the determination of offsets to meet a certain objective function. For example, if (hypothetically speaking) 99% of the traffic is in one direction and 1% in the other, the signal offsets will most likely be set without any consideration of the off-peak direction. This would essentially be a one-way arterial situation, for which just the travel time and signal spacing are factors for the determination of offsets. As the split approaches more of an even distribution, it is necessary to consider both directions in the determination of offsets. Although the heavier traffic flow direction will always be favored, the opposing direction will still be a significant factor in the offset calculations.

### **Software Tool**

A software tool has been provided that will allow the analyst to assess potential signal progression quality based upon signal spacing, cycle length, and travel speed. This tool returns an index value (between 0 and 1) that indicates the deviation (i.e., 0% to 100%) from the actual signal spacing to the spacing which would provide ideal two-way progression for the combination of input values. For example, if the relationship between the input values

provide for ideal two-way progression, a value of 0 will be returned. If the input conditions provide for the worst-case two-way progression (i.e., both directions of traffic flow will have to stop at the downstream intersection after departing from the upstream intersection), a value of 1 will be returned. The following discussion illustrates this concept through an example.

For a cycle length of 120 seconds and a travel speed of 40 mi/h (58.7 ft/s), the signal spacing that will give ideal two-way progression is,

$$\text{Signal Spacing} = \frac{120 \text{ s}}{2} \times 58.68 \text{ ft/s} = 3520 \text{ ft} \quad [4]$$

This corresponds to the first peak shown for the 120-second cycle length curve in Figure 19. In this figure, it can also be seen that the first valley of the 120-second cycle length curve occurs at a signal spacing 1760 ft. This corresponds to the worst-case spacing for two-way progression.

For this cycle length and speed, the ideal signal spacing for two-way progression is increments of 3520 ft (i.e., 3520, 7040, 10560, etc.). The worst-case spacing is also in increments of 3520 ft, but starting from a distance of 1760 ft rather than 0.

The spacing index calculation is based on three values: actual signal spacing, the ideal spacing for the given cycle length and travel speed, and the worst-case spacing for the given cycle length and travel speed. It is computed according to the following formula:

$$\text{Spacing Index} = \frac{\Delta Ideal}{(\Delta Ideal + \Delta Worst)} \quad [5]$$

Where:

$$\Delta Ideal = \text{Abs}(\text{Actual Spacing} - \text{Ideal Spacing})$$

$$\Delta Worst = \text{Abs}(\text{Actual Spacing} - \text{Worst Spacing})$$

For this example,  $\Delta Ideal$  equals 0 (3520-3520) and  $\Delta Worst$  equals 1760 (3520-1760). The gives the following result for the spacing index.

$$\text{Spacing Index} = \frac{0}{(0 + 1760)} = 0 \quad [6]$$

For a spacing of 1760 ft, cycle length of 120 seconds, and travel speed of 40 mi/h, the following values result.  $\Delta Ideal$  equals 1760 (Abs[1760-3520]),  $\Delta Worst$  equals 0 (Abs[1760-1760]), and the spacing index is,

$$\text{Spacing Index} = \frac{1760}{(1760 + 0)} = 1 \quad [7]$$

The software tool is described in more detail in Appendix D.

## **Chapter 6: Impact of Signal Spacing and Driveway Factors on Arterial Operations**

The focus of this chapter is on the development of a speed estimation model that can be used to obtain quantitative guidance on the related impacts of signal spacing, driveway and traffic factors on arterial operations downstream of an interchange off-ramp.

### **Research Approach**

This study was based primarily on a simulation approach. CORSIM (CORridor SIMulation) was chosen as the simulation program for use in this project. CORSIM was developed by the Federal Highway Administration (FHWA) and is one of the most extensively used simulation programs in the U.S. CORSIM offers a comprehensive set of input variables and corresponding ranges (20). It is a microscopic and stochastic based simulation program, meaning that traffic is modeled at the individual vehicle level, and traffic flow parameter values are randomly generated from specified probability distributions; thus, each simulation run can result in different output values, even for the same input values.

For a simulation model to have any validity, it must be capable of reasonably replicating field conditions. Other studies (e.g., Webster and Elefteriadou (26), Washburn (24), Kondyli (27)) have shown that CORSIM has this capability. However, like any simulation model, it must be calibrated for each particular study. Thus, the first step in the simulation process was to establish a network for which conditions could be calibrated to measured field conditions. This step entailed the identification of a suitable site for field data collection, the subsequent collection of data from this site, the replication of this site in the CORSIM simulation program, and then the calibration of the simulation model according to the field data. These activities are discussed in the following subsections.

#### Selection of Field Site

It was decided to identify an existing arterial/interchange area which was representative of sites where the number and proximity of driveways in the vicinity of an interchange lead to operational inefficiencies. It was also desirable that such a site was in close proximity to the University of Florida for data collection logistics reasons.

Newberry Road (also known as SR-26) in the vicinity of I-75 was the selected field site. This site is a good example of how arterial/interchange operations can deteriorate as a result of close signal spacing and multiple driveways.

Newberry Road is a major east-west route through the City of Gainesville and provides to/from Interstate-75. This road carries an AADT of approximately 50,000 (from FDOT FTI 2004 CD) and is one of the most heavily traveled roads in the city. One of the major traffic generators in this area is a large shopping center (Oaks Mall) of regional attraction. This route also serves as a primary connection between residential areas of west Gainesville and major employment centers such as the University of Florida and downtown Gainesville.

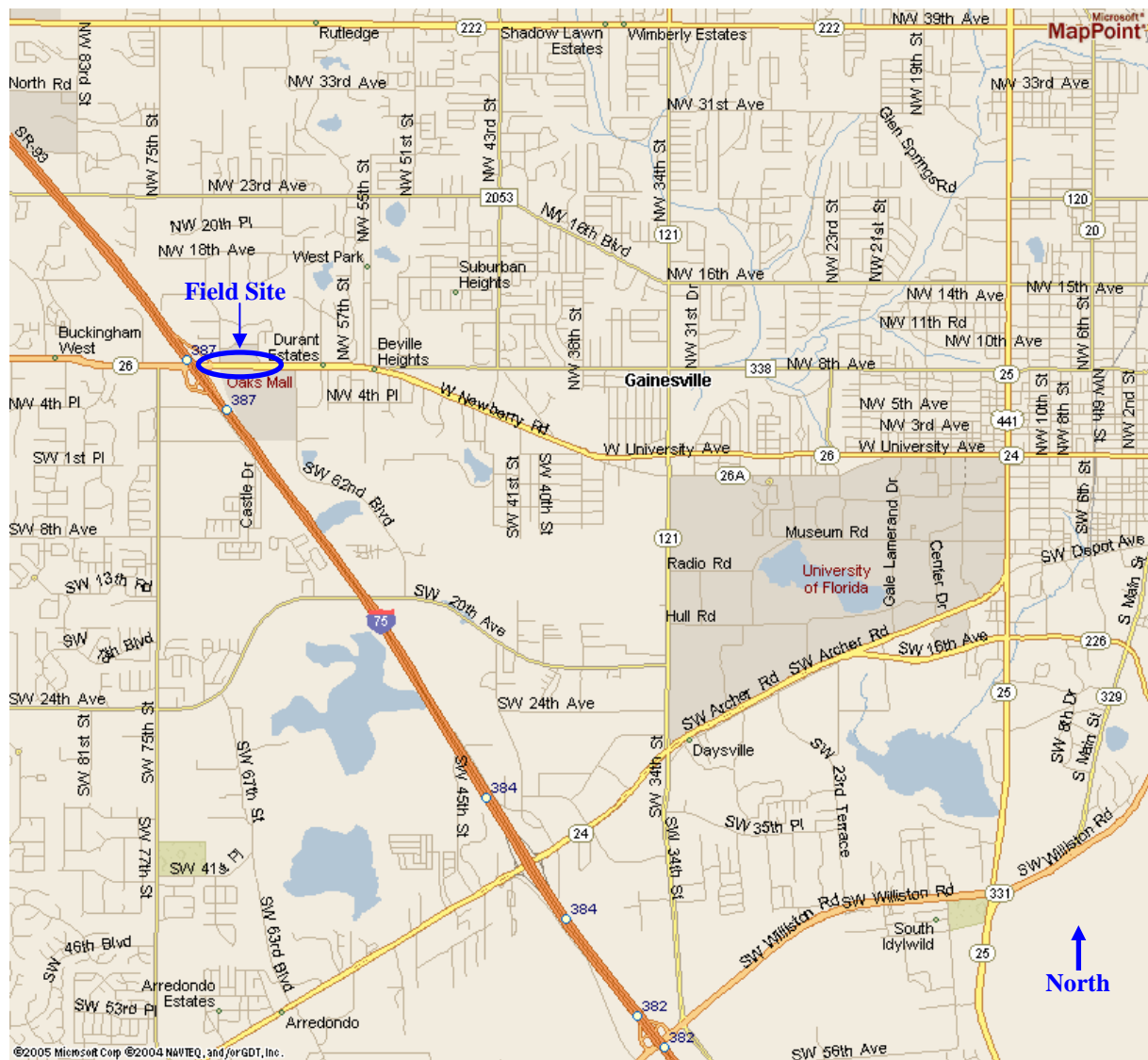
During times of moderate to high traffic demand, the operational quality of traffic flow on this roadway, in the vicinity of I-75, is very poor, with average speeds typically as low as 6 mi/h (posted speed is 35 mi/h). This poor traffic flow is significantly exacerbated by very close signal spacing and a large percentage of turning traffic. Under lighter volume conditions, average speeds reach about 24 mi/h.

From a distance of 0.6 miles to the east, and 0.3 miles to the west of I-75 (NW 62<sup>nd</sup> St to NW 76<sup>th</sup> Blvd), there are 9 signalized intersections (for an average signal spacing of 10 signals per mile). There are also numerous unsignalized driveways. The distance to the first downstream signal (in the EB direction) from NB right-turn off-ramp is approximately 460 ft. The left-turn lane extends for a distance of 210 ft back from this intersection. So when a queue is present in this lane during moderate to heavier traffic periods, there is very little distance available to make the weaving maneuver from the NB off-ramp to this left-turn lane. In addition, these vehicles have to weave across three through lanes. There are also three driveways (2 serving the same gas station) within this distance.

With the mall and a host of other retail, food, and convenience services, a large percentage of traffic turns off of and onto Newberry Rd. These turning movements cause additional inefficiencies at the signals (e.g., exclusive turn phases and resultant lost time, longer cycle lengths) and increased traffic friction at unsignalized access points. The median configuration is mostly closed along this distance, but there are a couple of locations with a directional median opening.

The problems are further compounded by the close spacing of the interchange ramp signals, which results in a lack of queuing storage between those signals. The distance between these signals is approximately 480 ft. In particular, during peak traffic periods, the westbound queue at the west-side ramps intersections often backs through the east side ramps intersections. The challenges with this situation are discussed in more detail in Chapter 3 of the report. Discussion about signal spacing for progression considerations is contained in Chapter 5 of the report.

The specific section of Newberry Road selected for data collection stretches from I-75 on the west to the main Oaks Mall entrance toward the east, spanning a total distance of about 1650 ft. A general site map is shown in Figure 21.



**Figure 21. General Vicinity Map of Newberry Arterial Site**



Along this section of Newberry Road are five signalized intersections, as follows  
(from west to east):

- I-75 NB off-ramp
- I-75 NB on-ramp
- NW 69<sup>th</sup> Terrace
- Oaks Mall West entrance
- NW 66<sup>th</sup> Street

Street level photos of these intersections are shown in Figures 20-23.



(a)



(b)

**Figure 22. Newberry Road and I-75 NB on-ramp (a) and NB off-ramp (b)**



(a)



(b)



(c)



(d)

**Figure 23. Newberry Road and NW 69<sup>th</sup> Terrace (a) WB approach (b) NB approach (c) departing WB traffic and SB approach (d) departing WB traffic**



(a)



(b)



(c)

**Figure 24. Newberry Road and Oaks Mall West Entrance (a) WB approach (b) NB approach (c) SB approach**



(a)



(b)



(c)

**Figure 25. Newberry Road and NW 66<sup>th</sup> Street (a) WB and EB approaches, looking east (b) NB approach on right (c) SB approach from North Florida Regional Medical Center**

Additionally, there are a number of driveways along this section of Newberry Road. Figure 26 shows an example of typical afternoon peak period congestion and a couple of unsignalized driveways.



**Figure 26. Westbound Newberry Road traffic during afternoon peak period**

#### Field Data Collection

For collecting traffic data, it was decided to use video. While the initial effort with camera installation is greater, the video format provides a permanent visual record of conditions that can always be consulted at a later date if necessary. Additionally, when data reduction is done from the video tape, the tape can be paused and rewound, thereby increasing the accuracy of the collected data.

The video data were collected from the site through four video camera installations. The video camera positions are identified by the circles in Figure 27. Each camera was facing toward the east (to the right in the figure).





**Figure 27. Data collection site (I-75 / Newberry Road) and video camera installation sites**

The video data collection equipment consisted of a small color camera attached to an apparatus that could be easily attached to a utility pole, a portable VCR, and a 12-volt power supply (battery). The VCR and battery were housed inside a “suitcase”. Figure 28 shows some photos of the equipment installation along Newberry Road.



**Figure 28. Photos of data collection equipment setup in field**

Figure 29 shows the field-of-view (FOV) for each video camera installation.



**Figure 29. Snapshots of video from each equipment installation site, a) Oaks Mall entrance, across from North Florida Regional Medical Center b) Oaks Mall West Entrance c) NW 69<sup>th</sup> Ave d) I-75 NB on- and off-ramps**

**Note: All cameras are facing to the east**

Video data were collected over several days to obtain a range of operating conditions. A total of about 18 hours of video data were collected over four days. These data collection periods are summarized in Table 11.

All mainline and cross street/driveway/ramp volumes were collected, as well as the percentage of heavy vehicles, by reducing the video data. Additionally, travel times between the two section endpoints were obtained through vehicle matching (manual observation of vehicles in video).

**Table 11. Video Data Collection Schedule**

<b>Location</b>	<b>Video Data Collection Days and Times</b>			
	<b>Friday (4/29/05) (2h 40min)</b>	<b>Saturday (4/30/05) (2h 40min)</b>	<b>Tuesday (5/3/05) (8h)</b>	<b>Wednesday (5/4/05) (5h)</b>
I-75 NB Ramps	3:36:00 pm – 6:16:00 pm	9:19:00 am – 11:59:00 am	10:03:00 am – 6:03:00 pm	7:45:45 am – 12:45:45 pm
NW 69 <sup>th</sup> Terrace	3:42:00 pm – 6:22:00 pm	9:26:00 am – 12:06:00 pm	9:52:00 am – 5:52:00 pm	7:41:30 am – 12:41:30 pm
Oaks Mall West	3:49:00 pm – 6:29:00 pm	9:33:00 am – 12:13:00 pm	9:42:00 am – 5:42:00 pm	7:35:00 am – 12:35:00 pm
NW 66 <sup>th</sup> Street	4:46:00 pm – 7:26:00 pm	9:38:00 am – 12:18:00 pm	9:31:00 am – 5:31:00 pm	7:28:50 am – 12:28:50 pm

The following figures contain volume data along the study corridor, which were obtained through the video reduction of the four data collection periods. Figure 30 corresponds to the measurements taken on Friday April 29<sup>th</sup> 2005 (5:16:00 PM – 6:16:00 PM), Figure 31 corresponds to the volume measurements of Saturday April 30<sup>th</sup> 2005 (10:59:00 AM – 11:59:00 AM) and Figures 12 and 13 show volume measurements for two time periods on Tuesday May 3<sup>rd</sup> 2005 (12:00:00 PM – 1:00:00 PM and 4:30:00 PM – 5:30:00 PM).

For the volume data collection, four categories of vehicle were recorded: passenger car, medium truck, large truck, and bus. The truck and bus categories were combined to arrive at a heavy vehicle percentage for each approach of each signalized intersection. These percentages are summarized in Appendix A.





**Figure 30. Volume data from video reduction taken on Friday April 29<sup>th</sup> 2005 PM peak period (5:16:00 PM – 6:16:00 PM)**



**Figure 31. Volume data from video reduction taken on Saturday April 30<sup>th</sup> 2005 midday period (10:59:00 AM – 11:59:00 AM)**



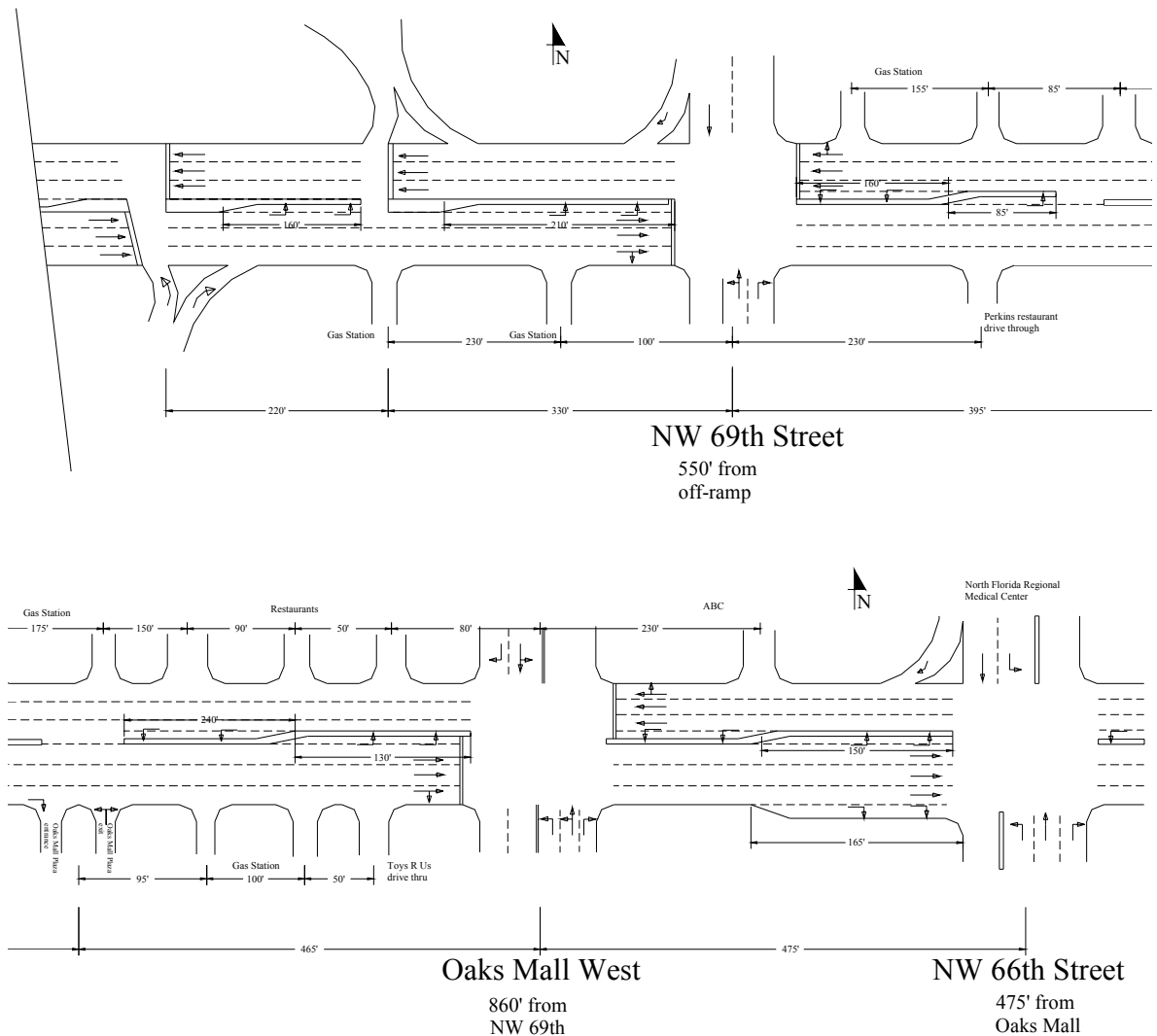


**Figure 32. Volume data from video reduction taken on Tuesday May 3<sup>rd</sup> 2005 midday period (12:00:00 PM – 1:00:00 PM)**



**Figure 33. Volume data from video reduction taken on Tuesday May 3<sup>rd</sup> 2005 PM peak period (4:30:00 PM – 5:30:00 PM)**

Geometric data were obtained from the video, an aerial photo, ground level photos, and site visits. The geometric data (lane configuration, channelization, and distances) for the field site are given in a schematic shown in Figure 34. The posted speed limit through this section of Newberry Road is 35 mi/h.



**Figure 34. Field Site Geometric Data**

Signal timing data were obtained from the City of Gainesville. These timings were confirmed from the video data. The cycle length in the field generally ranged from 120 to 180 seconds, depending on the volume conditions. For higher volume conditions, the cycle length was set at 180 seconds. The signal phasing and timing for the four signalized

intersections along this study site are summarized in Table 12. A graphical depiction of the intersection phasing plans is also included in Appendix B.

**Table 12. Signal Phasing and Timing Summary**

	<b>NB Ramps (Intersection 1)</b>			
	<b>Phase 1</b>	<b>Phase 2</b>	<b>Phase 3</b>	
<b>NEMA Signal Phases</b>	3, OL-D	2, 6, OL-C	2, 6, OL-D	
<b>Green Interval (sec)</b>	27	93	45	
<b>Yellow Interval (sec)</b>	3.5	3.5	3	
<b>All-Red Interval (sec)</b>	2	1	2	
	<b>NW 69<sup>th</sup> Terrace (Intersection 2)</b>			
	<b>Phase 1</b>	<b>Phase 2</b>	<b>Phase 3</b>	
<b>NEMA Signal Phases</b>	1, 5	2, 6	4, 8	
<b>Green Interval (sec)</b>	16	111	38	
<b>Yellow Interval (sec)</b>	3	3.5	3.5	
<b>All-Red Interval (sec)</b>	2	1	2	
	<b>Oaks Mall West (Intersection 3)</b>			
	<b>Phase 1</b>	<b>Phase 2</b>	<b>Phase 3</b>	<b>Phase 4</b>
<b>NEMA Signal Phases</b>	1, 5	2, 6	4	8
<b>Green Interval (sec)</b>	17.5	79	20	43
<b>Yellow Interval (sec)</b>	3	3.5	3.5	3.5
<b>All-Red Interval (sec)</b>	2	1	2	2
	<b>NW 66<sup>th</sup> Street (Intersection 4)</b>			
	<b>Phase 1</b>	<b>Phase 2</b>	<b>Phase 3</b>	<b>Phase 4</b>
<b>NEMA Signal Phases</b>	1, 5	2, 6	3, 7	4, 8
<b>Green Interval (sec)</b>	20	97	26	18
<b>Yellow Interval (sec)</b>	3	3.5	3	3.5
<b>All-Red Interval (sec)</b>	2	1	2	2

As previously mentioned, travel time data were obtained from the video data by matching vehicles at the section endpoint intersections (NB off-ramp and NW 66<sup>th</sup> Street) by manual observation. The clarity of the video was more than adequate to have a high degree of confidence in identifying the same vehicle at different intersections. The travel time data were collected for each direction of travel along the study section, for each of the four hourly periods. The required number of observations was determined according to the following equation:

$$n = \left( s \cdot \frac{z_{\alpha/2}}{\varepsilon} \right)^2 \quad [8]$$

where:

$n$  = minimum number of observations (i.e., travel time matches)

$s$  = estimated sample standard deviation, mi/h

$z_{\alpha/2}$  = constant corresponding to the desired confidence level

$\varepsilon$  = permitted error in the travel time estimate, mi/h

The confidence level was chosen as 90%, which corresponds to a  $z$ -value of 1.645 for a two-tailed test. The permitted error ( $e$ ) was selected to be 10% of the average travel time. Thus, this equation provides the number of travel time matches ( $n$ ) necessary to obtain the average travel time within a 10% error range at a 90% confidence interval. The field measured travel times are summarized in Table 13.

**Table 13. Travel Time (TT) Measurements Obtained from Field Data**

	<b>Friday PM (4/29/05)</b>		<b>Saturday AM (4/30/05)</b>		<b>Tuesday midday (5/3/05)</b>	
<b>Measure</b>	<b>EB</b>	<b>WB</b>	<b>EB</b>	<b>WB</b>	<b>EB</b>	<b>WB</b>
Average TT (sec)	77	228	63	54	67	59
St. Dev. (sec)	32	120	19	17	27	18
$e$ (sec) (10% of TT)	7.7	22.8	6.3	5.4	6.7	5.9
Number of TT matches required	46	75	25	27	44	25
Number of TT matches obtained	47	77	36	38	48	31

### Simulation Model Development

The Newberry Road network simulation model was developed based upon the collected traffic, roadway, and control data from the field. Three different volume conditions were used for calibration of the simulation model—low, medium, and high. These volume conditions were obtained from the three days of data previously mentioned (4/29/05, 4/30/05, and 5/3/05). The geometric representation of the simulation model can be seen in the following screen captures from TRAFVU (the companion traffic animation component to CORSIM).

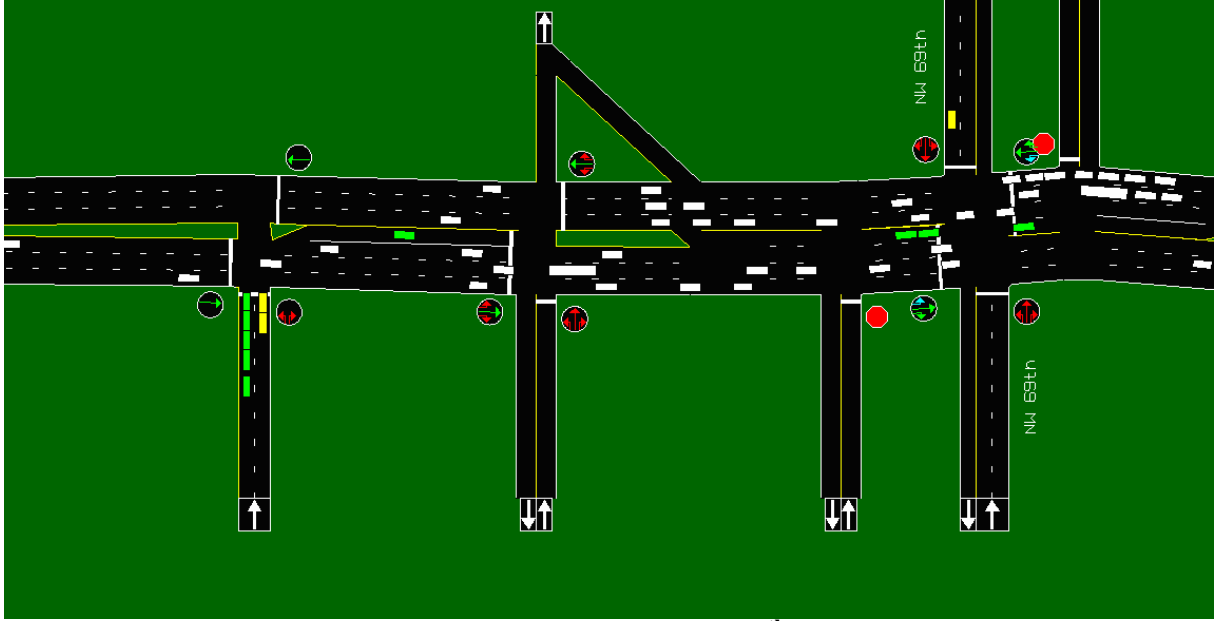


Figure 35. Newberry Road Site (NB off-ramp to NW 69<sup>th</sup> Terrace)

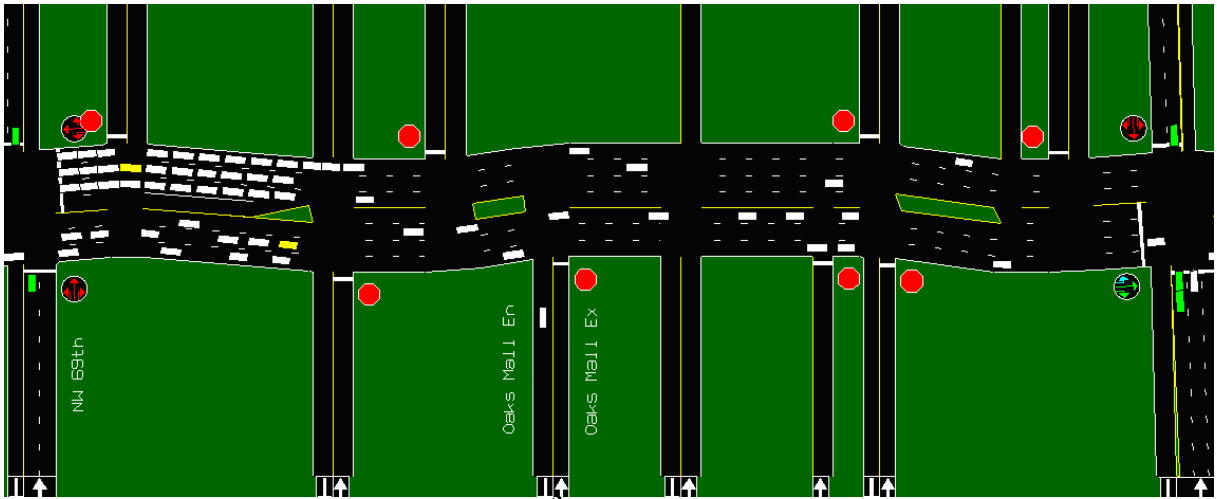
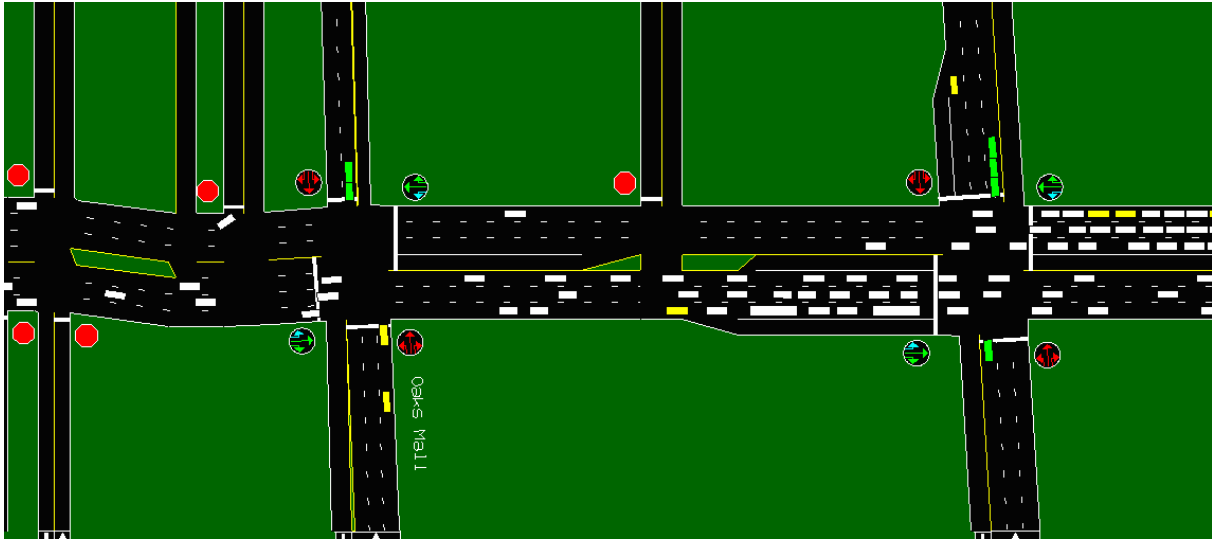


Figure 36. Newberry Road Site (NW 69<sup>th</sup> Terrace to Oaks Mall West)



**Figure 37. Newberry Road (Oaks Mall West to NW 66<sup>th</sup> Street)**

#### Calibration of Simulation Model

The calibration of the Newberry Road network was performed by adjusting simulation parameter settings such that the CORSIM simulated travel times along the network were within +/- 10% (consistent with the range established in the calculations for Table 13) of the field-measured travel times. The travel time results of the calibration effort are given in Table 14.

**Table 14. Travel Time Calibration Results**

	Travel Time and Acceptable Range from Field Data (sec)		CORSIM Travel Times (sec)	
	EB	WB	EB	WB
<b>Friday PM (4/29/05)</b>	77 [69.3, 84.7]	228 [205.2, 250.8]	75.5	247.5
<b>Saturday AM (4/30/05)</b>	63 [56.7, 69.3]	54 [48.6, 59.4]	59.8	55.4
<b>Tuesday (5/3/05) Midday</b>	67 [60.3, 73.7]	59 [53.1, 64.9]	73.1	54.5

#### Variable Selection and Experimental Design

Once the simulation models had been developed and calibrated for the Newberry Road site, the next step was to develop the experimental design and networks for the full simulation study.

The variables to be studied were developed in consultation with the project manager, Mr. Gary Sokolow. There are several roadway and traffic variables that can affect traffic operations along an arterial in the vicinity of an interchange. A summary of the variables selected for inclusion in this study are as follows.

- **Mainline Volume:** The significant impact that traffic volume has on travel speed has been clearly established in past traffic flow theory studies.
- **Driveway Volume:** Traffic turning into and out of driveways along the arterial creates additional movement conflicts and potential increased friction/turbulence to the mainline traffic flow.
- **Mainline Free-Flow Speed:** With increasing free-flow speed of vehicles on the mainline arterial, the greater the reduction in mainline vehicle speed when it is necessary for these vehicles to decelerate for their own turning maneuvers or those of other vehicles.
- **Downstream Signal Distance:** The proximity of the first signalized intersection to the interchange affects the traffic friction/turbulence created by vehicles weaving from the off-ramp to the inside lanes of the arterial, such as for making a left turn at the intersection. The shorter this distance, the greater the turbulence. This becomes an even greater factor with increasing traffic volume and left turn percentage. Signal proximity to the interchange can also affect operations within the interchange area as it relates to signal coordination and queuing issues. However, these issues were beyond the scope of this study.
- **Number of Driveways:** The number of driveways is directly related to the number of conflict points and the potential for increased traffic stream friction/turbulence. Thus, the more driveways (and consequently more driveway traffic) will logically add more disruptions the mainline traffic flow, thus reducing average travel speeds.
- **Downstream Signal Left Turn Percentage:** A higher percentage of traffic exiting from the off-ramp that wants to make a left turn at the downstream signal should lead to an increased amount of friction/turbulence due to those vehicles needing to move across several lanes of traffic so that they can enter the left turn lane at the downstream signal.



- **Median Type:** The median type present at a driveway affects the type of turning movements allowed at that driveway. The number of turning movements allowed at a driveway is directly related to the number of conflict points, which in turn can affect the amount of friction/turbulence present at the driveway/median site. In this study, three median types were considered: No opening (right-in, right-out only), directional opening (right-in, right-out, and left-in), full opening (all turning movements in and out of the driveway allowed).

Signal timing parameters such as cycle length, green splits, phasing sequence, etc., can also significantly impact arterial operations since they impact the intersection operations. However, given the number of parameters associated just for signal operations, it was decided to not include these as additional variables in the experimental design because it would result in an unfeasibly large number of required simulation runs. Alternatively, it was decided to use similar signal timing parameter values for the experimental network as those for the field site of Newberry Road. However, these parameter values were checked to make sure they still provided reasonably efficient signal operating conditions for the given input values.

With these variables selected for consideration in this study, an experimental design was developed such that the relationship of these variables to particular performance measures could be determined. Table 15 summarizes the experimental design for these variables, and their values, which were also selected in consultation with Mr. Gary Sokolow.

**Table 15. Experimental Design Factors and Levels**

#	Independent Variables	Level		
		Low	Med	High
1	Mainline peak direction volume (veh/h/lane)	400	800	1200
2	Mainline free-flow speed (mi/h)	40	45	50
3	Downstream signal distance (ft)	300	900	1500
4	Number of driveways <sup>1</sup>	1	2	3
5	Driveway trips/type (rule 14-96)	B	C	D
6	Downstream signal left turn %	5	10	15

<sup>1</sup> For the downstream signal distance of 300 feet, only 1 driveway will be used. This scenario will also be limited to just the low volume condition.

Table 16 summarizes the parameters and values that were specific to the variable of driveway type and volume (variable #5 in Table 15). The values for the driveway parameters were set according to the classification of the driveway (i.e., B, C, or D).

**Table 16. Driveway Parameter Settings for Experimental Design**

	Level				
Driveway Parameter	Low	Med		High	
Driveway classification	B	C		D	
Driveway volume (veh/h) <sup>1</sup>	30	90		260	
Driveway turning %'s					
— right-in, right-out	100/100	70/100	70/70	70/100	70/70
— left-in, left-out	0/0	30/0	30/30	30/0	30/30
Median configuration	No opening	Directional	Full	Directional	Full
% heavy vehicles	5	5		5	
Driveway separation	Equal distance				

<sup>1</sup> assuming 50% in, 50% out; midpoint of trips/day range, using K-factor of 0.10

<sup>2</sup> No median opening—right-in and right-out allowed; Directional median opening—right-in, right-out, and left-in allowed; Full median opening—all turning movements allowed.

Other variables that had fixed values or were dependent upon other variable settings were as follows:

- Directional split % for mainline volume, peak/off-peak: 60/40
- Mainline/off-ramp volume split %:
  - Low volume condition: 90/10
  - Medium volume condition: 80/20
  - High volume condition: 70/30
- Mainline heavy vehicle %: 5

If a full factorial design (i.e., every possible combination of variable values) were run based upon Table 15, a total of 729 (3<sup>6</sup>) unique combinations would result. However, in consultation with Mr. Sokolow, certain restrictions were placed on the design. It was decided that certain combinations of these variable values were highly unrealistic; and therefore, unnecessary to run.

The primary restriction pertained to the short downstream signal distance condition. For this situation, it was felt that the only realistic scenario would be just one driveway in between the interchange and downstream signal. Additionally, Mr. Sokolow indicated that for this situation, it should be considered with only the low volume and no median opening conditions. Thus, for the 300-ft downstream signal spacing situation, only mainline speed and left turn % were varied.

This results in:

1 (300 ft signal distance)  $\times$  3 (number of speeds)  $\times$  3 (number of left turn %'s)  
= 9 run combinations for this scenario.

For the other downstream signal distances, the following run combinations result:

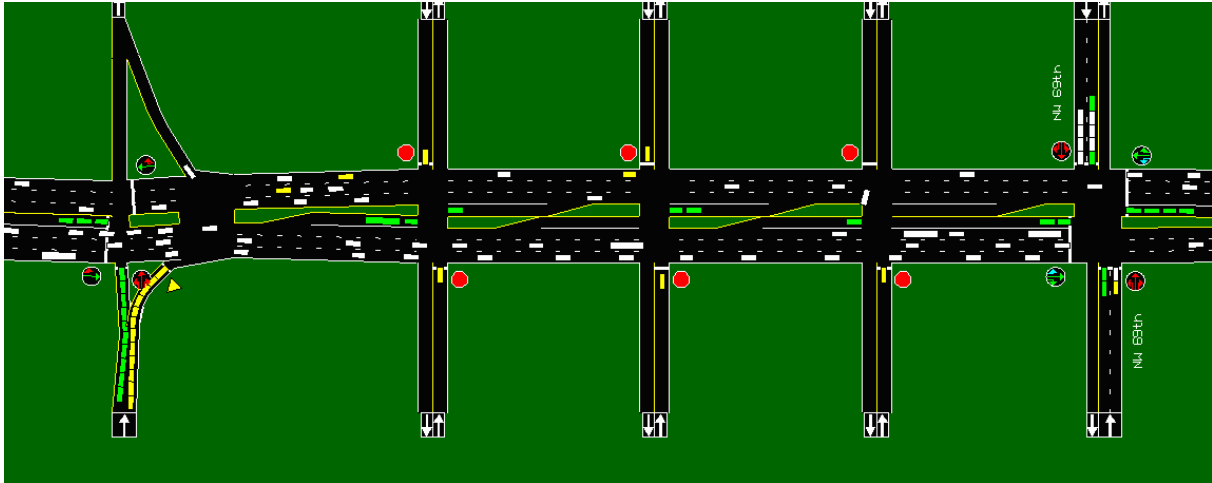
2 (900 & 1500 ft signal distances)  $\times$  3 (number of driveways)  
 $\times$  5 (combinations of median type and driveway and mainline volume)  
 $\times$  3 (number of speeds)  $\times$  3 (number of left turn %'s)  
= 270 run combinations

This gives a total number of run combinations of 279 (9 + 270). Additionally, to account for the stochastic nature of the CORSIM program, ten replications of each factor combination were run using different random number seeds. These ten runs provide an estimate of the variance for each run combination. These variance values are subsequently used in the analysis of variance that results in the regression equation(s). This resulted in a total of 2790 (279 $\times$ 10) separate CORSIM simulation runs being necessary for this experiment.

For each simulation scenario, an additional downstream signal was added to the network. This signal was always ¼ mile (1320) feet downstream of the first signalized intersection. There was also one driveway placed in between the two signals, and had the same roadway and traffic characteristics as the driveways on the first link.

To facilitate the efficient generation of the CORSIM input files (\*.trf), a Visual Basic program was written. The program systematically changed the input values within a base input file based upon a particular run combination. This process was more efficient than generating all the input files manually, and was particularly advantageous for the situation when it was discovered that an input value, or values, needed to be revised.

A screenshot from TRAFVU of one of the experimental design scenarios is provided in Figure 38. This screenshot shows three driveways between the off-ramp and the downstream signal. A graphical summary of the simulation network combinations is provided in Appendix C.



**Figure 38. Example Experimental Network (3 driveways)**

As previously mentioned, the signal timing parameters were not varied as part of this experiment. Instead, the values were set to be consistent with the field site of Newberry Road. These settings are summarized Table 17.

**Table 17. Signal Settings for Experimental Networks**

	NB Ramps (Intersection 1)			
	Phase 1	Phase 2	Phase 3	
NEMA Signal Phases	3, 8 NB LT, NB T/R	2, 6 EB T/R, WB T/R	2,5 EB T/R, EB LT	
Green Interval (sec)	35	105	24	
Yellow Interval (sec)	4	4	3	
All-Red Interval (sec)	2	1	2	
	First Downstream Intersection (Intersection 2)			
	Phase 1	Phase 2	Phase 3	
NEMA Signal Phases	1, 5 WB LT, EB LT	2, 6 EB T/R, WB T/R 1, 5 (perm.) WB LT, EB LT	4, 8 SB T/R, NB T/R 3, 7 (perm.) NB LT, SB LT	
Green Interval (sec)	40	117	7	
Yellow Interval (sec)	3	4	4	
All-Red Interval (sec)	2	1	2	
	Second Downstream (Intersection 3)			
	Phase 1	Phase 2	Phase 3	Phase 4
NEMA Signal Phases	1, 5 WB LT, EB LT	2, 6 EB T/R, WB T/R 1, 5 (perm.) WB LT, EB LT	4, 7 SB T/R, SB LT	8, 3 NB T/R, NB LT
Green Interval (sec)	17.5	79	20	43
Yellow Interval (sec)	3	3.5	3.5	3.5
All-Red Interval (sec)	2	1	2	2

Another Visual Basic program was developed for extracting performance measures values for the CORSIM output files. Visual Basic macros were also written for Microsoft Excel to facilitate organization and analysis of the data. These programs were used to process the data generated from the 2790 CORSIM simulation runs. Once the data were organized into a suitable format, a statistical analysis was performed, as described in the next section.

## **Results and Analysis**

A regression analysis approach was used to investigate the relationship of the experimental design variables (independent variables) with the selected performance measure (dependent variable). The method of regression analysis is specifically designed for the investigation of the relationship between two or more variables that are related in a nondeterministic fashion (28).

An example of a simple linear regression model can be given by the following general formula.

$$y = \beta_0 + \beta_1 x + \varepsilon \quad [9]$$

where:

$y$  = predicted value of dependent variable

$\beta_0$  = estimated constant value

$\beta_1$  = estimated coefficient value for independent variable

$x$  = value of independent variable

$\varepsilon$  = random error term

### Analytical Model Development

The primary performance measure that was investigated was mainline arterial average speed. This measure is recognized as one of the primary measures of arterial performance. The Highway Capacity Manual (15) uses average speed as the primary performance measure for evaluating level of service on signalized arterials.

The advantage to developing a predictive model for average speed is that it can take into account the free-flow speed (which is function of the posted speed limit) of the arterial and the speed value can easily be translated into a delay value by taking the difference in estimated speed for the conditions and the free-flow speed.

After testing various model configurations, in terms of included combinations of variables and interactions, the following best-fit model was identified.

$$\begin{aligned} \text{Avg. Speed} = & 11.57 - 0.4572 (V_a/100) - 0.0099 (Ndr \times Vdr_{out}) - 0.0117 (Ndr \times Vdr_{in}) + \\ & 0.8307 Ndr + 1.7377 (SigDist/100) - 0.0479 (SigDist/100)^2 - \\ & 19.29 (\%LT/100) + 0.405 FFS \end{aligned} \quad [10]$$

where:

Avg. Speed = average speed of vehicles on arterial between off-ramp and first signalized intersection

$V_a$  = arterial mainline volume, in analysis direction (veh/h)

$Ndr$  = number of driveways between off-ramp and first signalized intersection

$Vdr_{out}$  = average outbound driveway volume per lane (veh/h)

$Vdr_{in}$  = average inbound driveway volume per lane (veh/h)

$SigDist$  = distance between off-ramp and first signalized intersection (ft)

$\%LT$  = percentage of left turns at first signalized intersection

$FFS$  = free-flow speed of mainline arterial traffic (mi/h)

The statistical results the of the regression analysis for this model are given in Table 18.

**Table 18. Model Estimation Statistical Results**

Parameter	Coeff.	t-stat
Constant	11.5738	24.36
$V_a$	-0.4572	-53.37
$Ndr \times Vdr_{out}$	-0.0099	-17.48
$Ndr \times Vdr_{in}$	-0.0117	-8.63
$Ndr$	0.8307	11.65
$SigDist$	1.7377	25.96
$SigDist^2$	-0.0479	-16.22
$\%LT$	-19.2916	-24.72
$FFS$	0.4054	54.70

The significance of each variable to the model is indicated by its  $t$ -statistic (coefficient divided by standard error). All of the variables in Table 18 are significant at well over the 99% confidence level, which corresponds to a  $t$ -statistic value of 2.58.

The goodness-of-fit for this linear regression model is based upon the coefficient of determination,  $R^2$ . This value provides a measure of the amount of variance in the dependent variable explained by the model, on a scale of 0-100%. Given that the  $R^2$  value will always increase with the addition of variables, even insignificant ones, an alternative measure is typically used, the adjusted  $R^2$ . The adjusted  $R^2$  will penalize the  $R^2$  value for the presence of irrelevant variables. The adjusted  $R^2$  value for this model is 90.8%. This value indicates that

approximately 91% of the variation in the dependent variable (speed) can be explained by the model.

For the most part, the signs of the variable coefficients in this model are reasonable. The average speed of the arterial traffic will decrease with increasing traffic volume. The combination of the number of driveways variable and the two interaction variables with number of driveways and driveway volume results in overall decrease in average travel speed as the number of driveways and the traffic volume entering and exiting those driveways increases. The parameters for the interaction of the number of driveways and average driveway volume variables indicate that driveways only begin to adversely impact arterial operations once driveway volumes and arterial volumes become significant, as expected. Increasing distance between the off-ramp and the first downstream signal will lead to increasing average travel speed. As expected, the model indicates that an increase in left turn percentage at the first downstream signal will lead to a lower speed. And, of course, higher free-flow speeds lead to higher average travel speeds (for undersaturated conditions).

During the initial model development, indicator variables for the type of median configuration (e.g., directional) were included. Indicator variables are set to a value of one if true, and a value of zero if not true. Indicator variables can be included in a model for the total number of alternatives minus one. So with three median configurations included in the experimental design (none, directional, full), two indicator variables were included in the model testing, one for directional and one for full. The choice of which two to include is arbitrary, as the coefficient values are relative.

In the initial model, the coefficient values for the directional and full median opening variables were positive, indicating that these median types would lead to a higher speed (on the order of 2 mi/h) than that for a median with no opening. Initially, this result seemed counter-intuitive, as it was expected that the no median opening condition would lead to fewer conflicts and a higher speed. However, after further investigation, the reason for this result was discovered. For the simulation scenarios, the amount of driveway traffic (in and out) was not a function of median type. Thus, in the case of changing the median type from directional to no opening, the traffic that turned left into the driveway (for directional) must now turn right into the driveway (for no opening), to keep the driveway volume at a constant value. In the CORSIM simulations, the left turning traffic into the driveway essentially does not cause

any delay to the opposing direction of traffic flow (the analysis direction). However, when this traffic must take a right turn into the driveway from the analysis direction, these vehicles will impact the speed of the analysis direction vehicles due to their need to slow in the travel lane to make the right turn into the driveway. Likewise, vehicles turning left out of a driveway essentially do not impart delay on the analysis direction vehicles; whereas, right turn exiting vehicles do. Thus, assuming a constant driveway volume, the more left turn movements that are allowed in/out of a driveway, the less negative the impact on speed to the analysis direction. However, the left turn maneuvers will have an impact on speed to the other direction of flow (the direction that these vehicles will diverge/merge with), although the effect on speed of a left turn diverge should not be as significant as that of the right turn diverge since an exclusive left turn lane is usually provided. Obviously, an exclusive right turn lane on the mainline will lessen the impact on speed for right turn diverge movements as the deceleration can be accomplished in this lane.

In practice, it is generally found that closed medians will lead to better operations (as well as safety) in both directions of travel. However, given the constraint of constant volume (which is probably unlikely in reality given the median restrictions), this result from the simulation model was reasonable. Since this simulation result was inconsistent with previous experience (per project manager Mr. Sokolow), it was decided to not include the median configuration variables in the final model.

To demonstrate the application of the speed prediction model, three sample calculations are provided in the next section.

### Example Calculations

Table 19 provides a summary of the input values for three example applications of the speed prediction model. All three scenarios use a medium arterial volume level (800 veh/h/lane), driveway volumes of 50 veh/h in and 50 veh/h out, 10% left turns at the signal, and a free-flow speed of 40 mi/h. The differences in the three scenarios are downstream signal distance (high, medium, and low for scenarios 1, 2, and 3, respectively), and the number of driveways (3, 2, and 1 for scenarios 1, 2, and 3, respectively).



**Table 19. Variable Input Values for Sample Calculations**

<b>Variable</b>	<b>Scenario 1</b>	<b>Scenario 2</b>	<b>Scenario 3</b>
$V_a$	2400	2400	2400
$Vdr_{out}$	50	50	50
$Vdr_{in}$	50	50	50
$Ndr$	3	2	1
$SigDist$	1320	880	440
$\%LT$	10	10	10
$FFS$	40	40	40

Scenario 1

$$\begin{aligned} \text{Avg. Speed} &= 11.57 - 0.4572 (2400/100) - 0.0099 (3 \times 50) - 0.0117 (3 \times 50) + 0.8307 (3) \\ &\quad + 1.7377 (1320/100) - 0.0479 (1320/100)^2 - 19.29 (10/100) + 0.405 (40) \\ &= 28.73 \text{ mi/h} \end{aligned}$$

Scenario 2

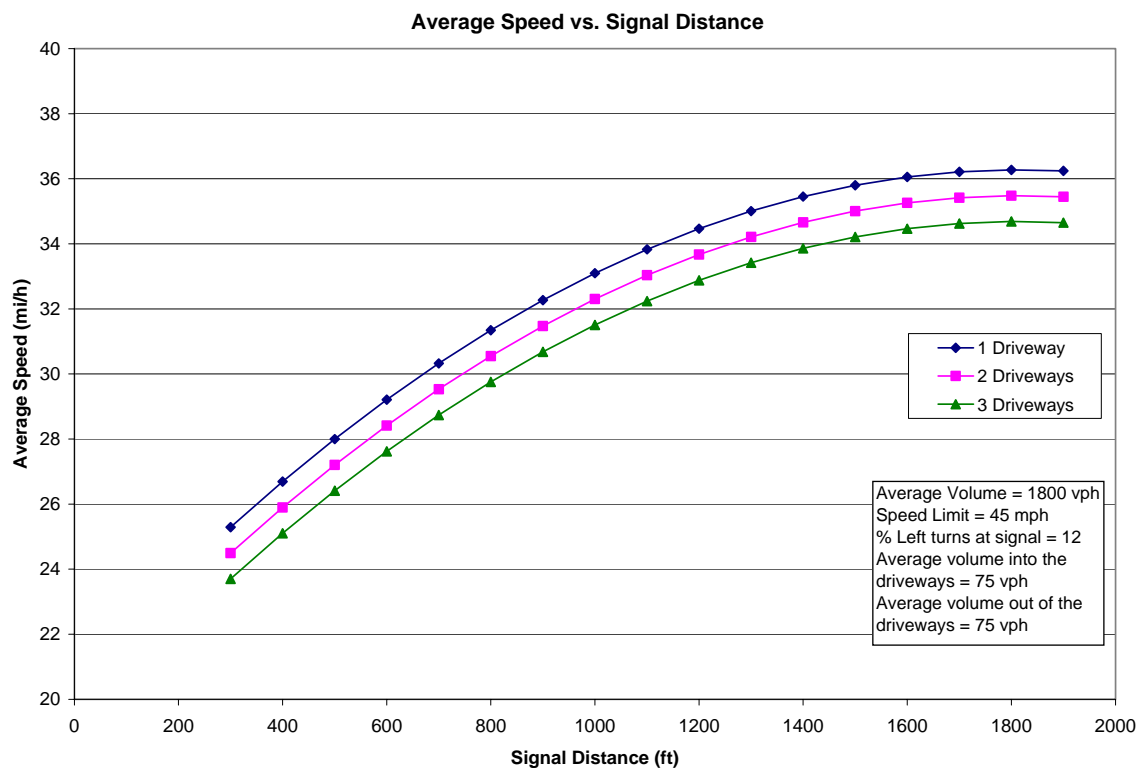
$$\begin{aligned} \text{Avg. Speed} &= 11.57 - 0.4572 (2400/100) - 0.0099 (2 \times 50) - 0.0117 (2 \times 50) + 0.8307 (2) \\ &\quad + 1.7377 (880/100) - 0.0479 (880/100)^2 - 19.29 (10/100) + 0.405 (40) \\ &= 25.97 \text{ mi/h} \end{aligned}$$

Scenario 3

$$\begin{aligned} \text{Avg. Speed} &= 11.57 - 0.4572 (2400/100) - 0.0099 (1 \times 50) - 0.0117 (1 \times 50) + 0.8307 (1) \\ &\quad + 1.7377 (440/100) - 0.0479 (440/100)^2 - 19.29 (10/100) + 0.405 (40) \\ &= 21.36 \text{ mi/h} \end{aligned}$$

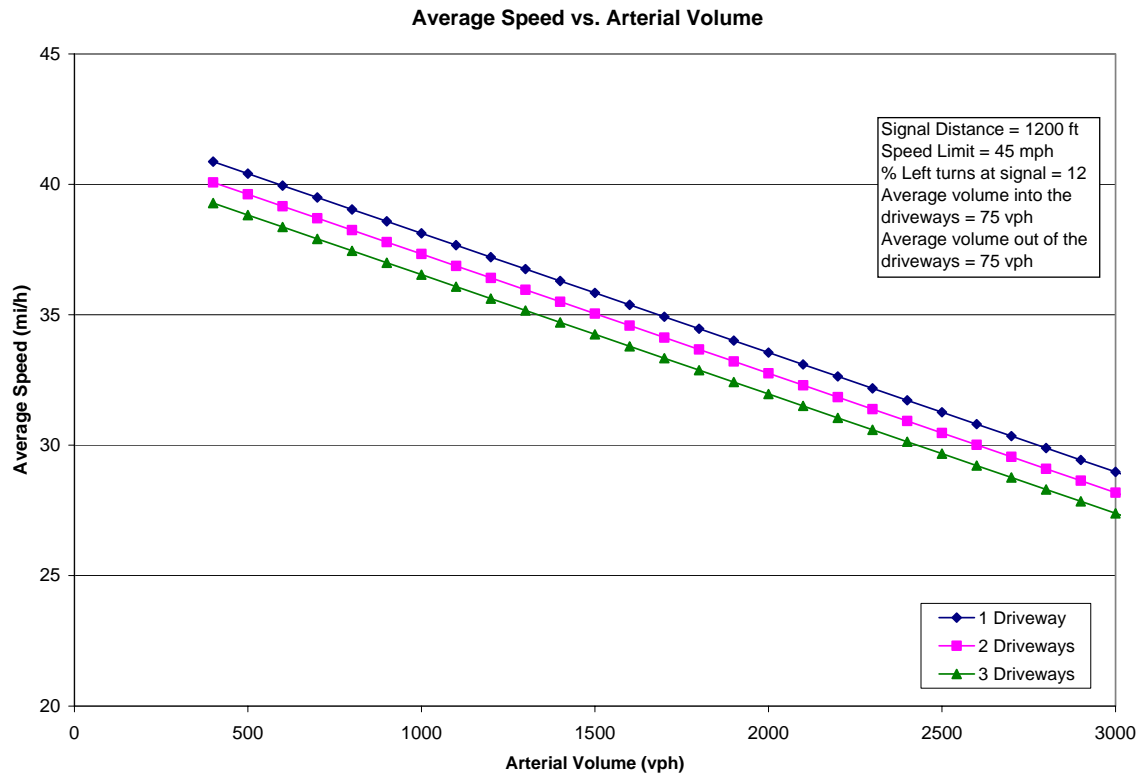
These example calculations illustrate the effect that signal distance can have on the average speed of the arterial traffic. The first scenario also includes two more driveways; whereas, scenario 3 only includes one. The effect on speed due to just the signal distance is given by the two coefficient values of 1.7377 and -0.0479. This relationship is non-linear; thus, the first coefficient pertains to the linear component and the second term pertains to the quadratic component. This non-linear relationship accounts for the diminishing return that signal distance will have on speed beyond a certain value (i.e., one cannot keep increasing speed by increasing signal distance, eventually the speed will reach its maximum under the given conditions for the other factors). For example, the effect on speed for the spacing of 1320 ft is 14.60 mi/h ( $1.7377 (1320/100) - 0.0479 (1320/100)^2$ ) and for 440 ft is 6.72 mi/h ( $1.7377 (440/100) - 0.0479 (440/100)^2$ ), or a net difference of 7.88 mi/h. The relationship

between signal distance and average speed, assuming fixed values for the other parameters, is illustrated in Figure 39. This relationship is non-linear to reflect the fact that at a certain distance, in this case about 1800 ft, the effect of signal spacing on speed becomes negligible. That is, it should not be possible to get ever-increasing speeds with increasing signal spacing. An absolute upper bound to speed is obviously the free-flow speed. It should be noted, however, that the mathematical form of this particular non-linear relationship is quadratic. Thus, it is theoretically possible to get an inverse relationship between speed and distance after 1800 ft as the curve begins to slope downward at that point. But this is just an artifact of the quadratic relationship and it should be assumed that any signal distance greater than 1800 ft has the same impact on speed as a distance of 1800 ft. This is accounted for in the software tool described later. The peak of this quadratic relationship being at 1800 ft is tied to the upper boundary value of 1500 ft used in the simulation runs.



**Figure 39. Relationship between signal distance and average speed**

Figure 40 illustrates the relationship between arterial volume (in analysis direction) and average speed, again assuming fixed values for the other input parameters.

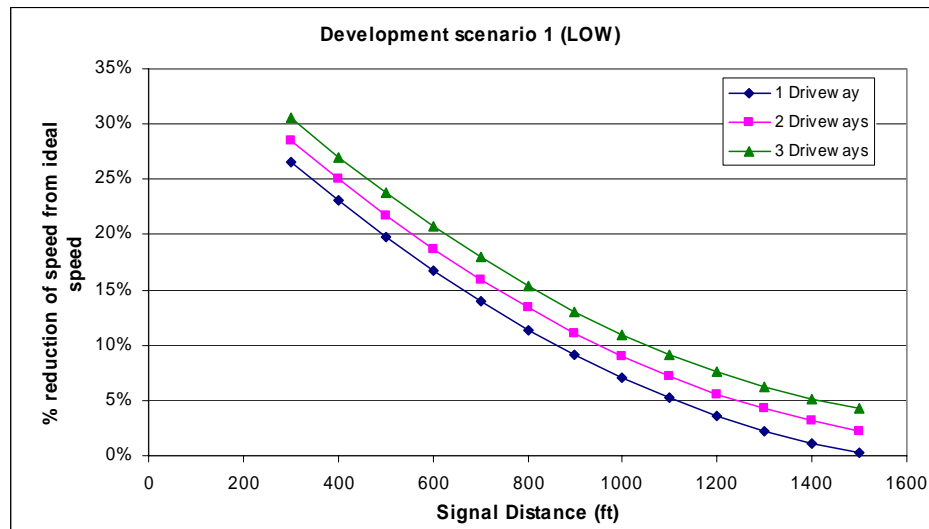


**Figure 40. Relationship between total arterial volume and average speed**

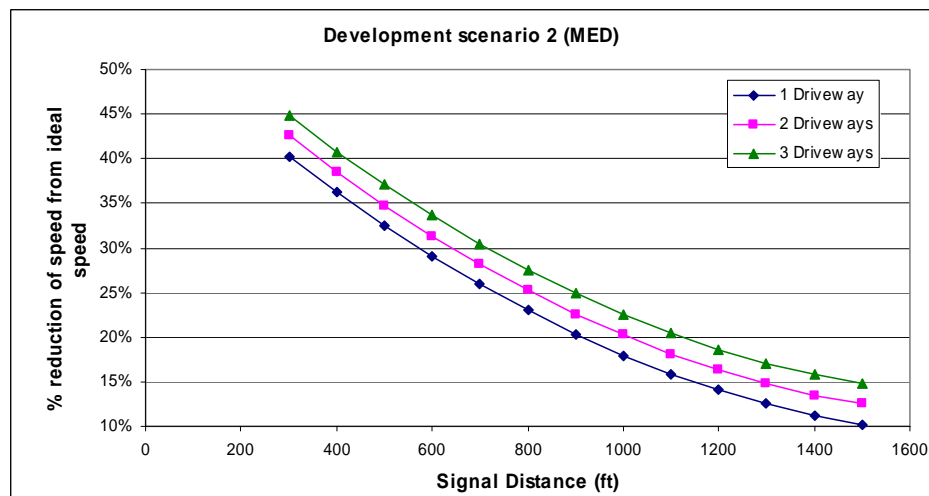
The three graphs shown below illustrate the impact of signal spacing on the percentage in speed reduction from an ideal value, for three different development scenarios. In this report, the ideal speed is defined as the speed limit minus 5 mi/h. This is seen as a reasonable upper limit for speed as any arterial with signalized intersections will almost never average speeds approaching the speed limit (except under near zero traffic demand conditions). The first scenario presented in Figure 41a corresponds to an arterial configuration under low development which typically includes low though vehicle volume and high operating speeds. The medium development scenario shown in Figure 41b represents cases with medium volume and speed levels. The high development scenario of Figure 41c represents arterial configurations with typically higher volumes and lower operating speeds. The input parameter values used to generate the speed-signal spacing relationship graphs are given in Table 20.

**Table 20. Selection of factors inputs for three development scenarios**

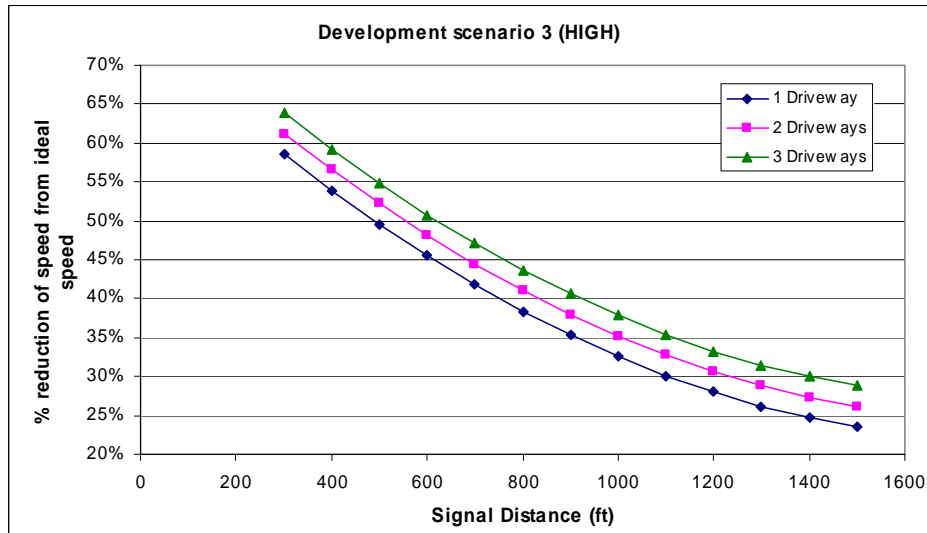
	Development Scenario		
	Low	Medium	High
$V_a$ (veh/h)	1200	2400	3600
$Vdr_{out}$ (veh/h)	75	75	75
$Vdr_{in}$ (veh/h)	75	75	75
% $LT$	5	10	15
$FFS$ (mi/h)	50	45	40



**(a)**



**(b)**



(c)

**Figure 41. Graphs of signal distance and speed reduction from the ideal speed under three different development scenarios. (a) low development. (b) medium development, and (c) high development**

A software tool has been developed to allow an analyst to identify specific percentage reductions in average speed (from an ideal value) for any given signal spacing and other values of input parameters. This software tool is described in more detail in Appendix D.

## Chapter 7: Summary and Recommendations

Arterial operations are dependent on a number of factors. In the vicinity of an interchange, however, some of the most significant are the distance between the freeway off-ramp and the first signalized intersection, the number of driveways (and their volume) in between, and the percentage of traffic turning left at the signalized intersection. The interaction between these variables has a significant influence on the amount of traffic turbulence that gets created in the vicinity of the interchange, which in turn has a significant influence on arterial travel speeds and the ability to efficiently move traffic off of and onto the freeway.

For example, an interchange area that has a short distance to the first signal, one or two driveways in between, and a relatively high percentage of off-ramp vehicles wanting to turn left at the signal will lead to much more traffic turbulence in this area and very inefficient arterial/interchange operations. On the other hand, if the distance to the downstream signal is relatively long, this will provide the left turning vehicles the ability to make much smoother weave movements from the outside to inside lane and have a much less significant impact on overall traffic operations in the vicinity of the interchange.

Signal spacing also has a major impact on progression quality, and it is therefore essential to consider the spacing of the signals adjacent to the interchange in relation to the spacing of the interchange ramp terminal intersections (when signalized). The spacing of the interchange ramp terminals is also an important factor in arterial operations. Closely spaced intersections will require shorter cycle lengths in order to clear the queues from the short internal link storage area, which again will affect the ability to perform signal coordination with the adjacent signals.

Traffic operations in the vicinity of an interchange are very dynamic, and as such, it is necessary to consider several factors when making permitting decisions about driveway locations and median types. This study focused specifically on two issues related to signal spacing in the vicinity of interchanges: progression quality and arterial average speed. The following section discusses the recommendations of this study in these two areas and also provides some general recommendations with regard to the FAC Rule 14-97 considering the results of this study as well as previous related studies.

## **Recommendations**

### Arterial Speed

The multivariate-based model developed in this study provides a first step in allowing the analyst to quantitatively evaluate the tradeoff in operations due to a combination of factors.

One way in which this tool can be applied is in establishing recommended distances to the first downstream signalized intersection from an interchange off-ramp. Based on the concept of identifying percentage speed reductions from an ideal speed for a given combination of input parameter values, signal spacing values can be established for a desired maximum reduction in speed. A table of recommended signal spacing was developed using this method. For the selection of target operating speeds, Exhibit 15-2 from the HCM2000 was used as a guide. This table gives average speed values for given levels of service on signalized arterial streets. The speed values chosen from this table correspond to the threshold between LOS B and LOS C. Additionally, the speed values from street classes I, II, and III were used from this table to correspond with low, medium, and high development levels, respectively. The chosen target speed values correspond to a percent reduction in speed of approximately 30% in all cases when compared to the free-flow speed. If a speed less than free-flow speed is chosen as the comparison case, the target percent reduction in speed will obviously be lower. For example, if free-flow speed minus 5 mi/h is considered to be the upper threshold speed, then the chosen target speeds are on the order of a 20% reduction from ideal speed.

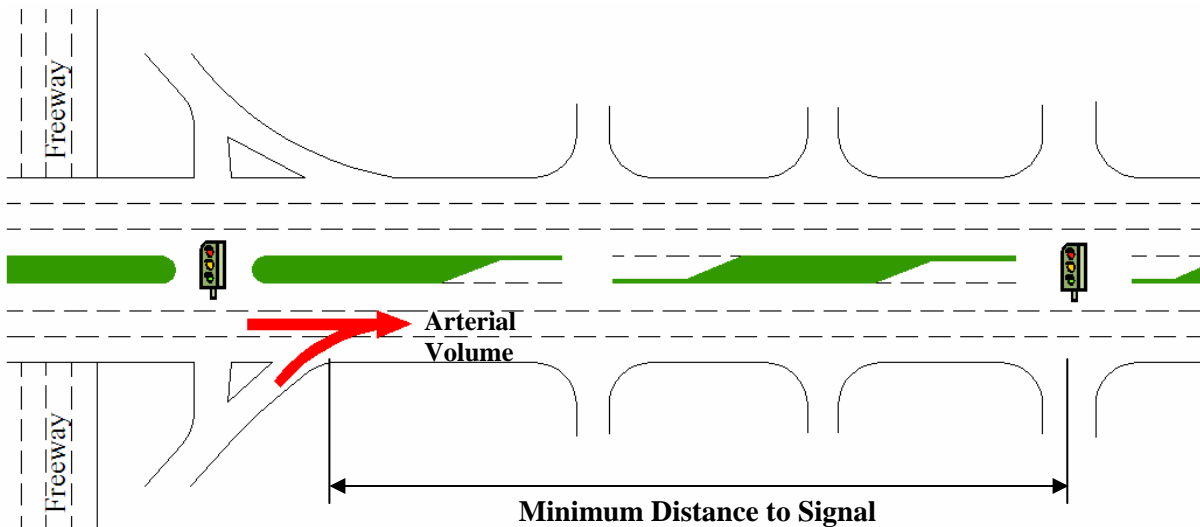
The minimum signal distances presented in Table 21 are measured from the off-ramp taper up to the first downstream signal. The measurement of this distance is consistent with the one described in NCHRP 420 (2), in Layton's research (4), and in TRB's Access Management Manual (3) and is illustrated as distance "Y" in Figure 2 and Figure 5.

The minimum signal distance recommendations by development level are shown in Table 21. For the development of this table, the number of driveways was fixed at two, with an average of 75 vehicles in and 75 vehicles out of each one. The calculated distances are rounded to the nearest 50 ft increment.

**Table 21. Recommended Minimum Signal Distance by Development Level**

Traffic and Development Levels	Free-Flow Speed (mi/h)	Target Speed (mi/h)	Arterial Volume (veh/h)	Minimum Signal Distance (ft)	
				10% LT	20% LT
Low	50	34	1200	850	1050
Medium	40	28	2000	1050	1350
High	35	24	2400	1050	1350

As can be seen from this table, most of the signal spacing values are between 1000 ft and approximately ¼ mile (1320 ft). This ¼ mile distance is consistent with the minimum distances given in FAC Rule 14-97 for several of the arterial classes. It should be noted that the distance of 850 ft would only be feasible if the volume was expected to remain low. However, this expectation is generally not reasonable. And since you usually have the option of implementing longer distances in low development situations, which will likely lead to safer and more efficient operations in the future, it is still recommended to use the longer distances from this table. An illustration of the relevant parameters from Table 21 is shown in Figure 42.

**Figure 42. Illustration of Parameters Relevant to Table 21.**

#### Progression Quality

In addition to the impacts on arterial travel speed due to signal spacing, progression quality is significantly influenced by signal spacing. For rural conditions, however, progression quality is not as critical of a concern. Since traffic demands are relatively low, particularly from the side streets due to the lack of roadside development, the g/C ratios for



the major street can be higher and will thus compensate for any reduced progression quality. Thus, when considering signal spacing for progression quality, the high development scenario should be the design case. Under this scenario progression quality will have the most impact to arterial operations. And with the ever-increasing growth in Florida, rural areas are likely to eventually become urban areas.

For the travel speed and cycle length values typical for high development conditions, signal spacings on the order of 2700 ft or greater are generally necessary for progression considerations. However, as Figure 20 in Chapter 5 shows, for distances greater than 2640 ft, platoon dispersion begins to greatly diminish the effects of signal spacing for progression quality. Thus, 2640 ft should be considered as a practical upper bound for signal spacing with respect to progression quality concerns. As for a practical lower bound, 1320 ft is a reasonable value. This value still provides some flexibility for establishing a reasonable level of progression, particularly when considering the adjacent interchange ramp terminals intersections. Although tight diamond (distance between signals at ramp terminals is less than 400 ft) configurations, as shown in Figure 8, make it very difficult to obtain any reasonable level of progression quality, other configurations may provide some flexibility for coordinating with the adjacent signals. For example, diamond configurations that can obtain a spacing between 660 and 1320 ft can be coordinated to some extent. If the internal interchange link is half the length of the adjacent links (e.g., 660 vs 1320 ft) and the ramp signals can run at one-half the cycle length of the adjacent signals (i.e., ‘double-cycling’), it will still be possible to maintain a reasonable level progression quality.

### General

Previous research has examined other factors to consider for driveway and signal spacing, such as weaving distance and sight distance. In general, there is considerable consistency between them with regard to minimum spacing guidelines for driveways, median openings, and signalized intersections. Of course, some of this consistency is because more recent studies have relied heavily upon, or directly referenced, the results from the earlier studies.

The previous study most similar to this study was the one performed by McShane et al. (14). Both studies had a number of elements in common. The key difference is that the McShane study focused on several univariate, or bivariate, relationships (i.e., one or two

independent variables vs. the dependent variable—arterial travel speed in this case), whereas this study used a multivariate approach (that is, testing the simultaneous effect of multiple independent variables on the dependent variable).

The McShane study did not recommend any specific spacing guidelines for either signals or driveways, but rather was focused on general trends of the relationships between the performance measure and experimental variables. In this regard, the trends identified by the McShane study were consistent with those established in the quantitative model of this study, with the exception of the effect of median configuration. One other difference is that the McShane study investigated the effect of acceleration and deceleration lanes, whereas this study did not.

From CUTR's research (13) on the operational effects of limiting access near freeway ramps, it is concluded that the ROW acquisition is an important factor for defining the viability of an interchange. Their analysis showed the relationship between the length of access controlled frontage and interchange operations failure, for a given arterial volume, and also the estimated delay savings between different lengths of access control frontage over a period of time.

As part of their operational analysis they also examined the capacity gains from increasing the access spacing, by measuring the vehicle hours of delay for the entire network. The authors estimated the vehicle delays under three alternative access spacing scenarios (200 ft, 600 ft and 1320 ft) and over a 20 year design life (they assumed a 3% annual increase in volume). The delay reduction between the 600 ft or 1320 ft and the 200 ft alternative would imply whether a ROW acquisition could potentially extend the operational life of the interchange.

Therefore, the derived guidelines and relationships developed here between the signal spacing and the arterial speed can be used in conjunction with the estimated delay savings from CUTR's report, to identify the feasibility of an interchange project throughout the design period, given an anticipated volume growth factor. (Note: CUTR's report is specific to 4-lane arterials).

The results from NCHRP 420 state that the minimum spacing distance to the first major signalized intersection should be  $\frac{1}{4}$  mile for 2-lane arterials and  $\frac{1}{2}$  mile for 4-lane arterials. These standards, however, are not sensitive to arterial speeds and degree of

development, but they are comparable to the progression quality recommendations given in this report.

The guidelines provided in Rule 14-97 (shown in Table 6) are generally in line with past and current research, and for the purpose of providing general guidance, the values appear to be reasonable. While the results of this project provide tools that allow the analyst to obtain more specific guidance for specific input conditions, it is not recommended that these be used to revise the values of the Rule 14-97 at this time. The results from this study indicate that a minimum signal spacing of  $\frac{1}{4}$  mile may be appropriate for a range of conditions considering operational issues such as arterial speeds and progression quality. However, in many cases the development conditions eventually reach a high level, in which case the more restrictive guidelines of  $\frac{1}{2}$  mile for signal spacing,  $\frac{1}{4}$  mile for median opening spacing, and  $\frac{1}{8}$  mile for driveway spacing become more reasonable values. The signal spacing guidance of  $\frac{1}{2}$  mile ultimately provides the most flexibility for accommodating weaving maneuvers (also considering left-turn lane queuing), arterial travel speeds, and signal progression. At this point, the researchers do not feel it is warranted to make the general guidance any less restrictive. The tools developed in this project can be used to provide an assessment of the impact of certain parameter values on arterial operations, and may prove useful at the planning level, but it is not suggested that these results be used to supersede the values of Rule 14-97. Ultimately, the researchers feel that this table could be expanded to include more independent variables and spacing options, but there is still some additional research that needs to be done before this will be feasible. This could be part of a future study that builds upon this one and attempts to tie together all the different facets affecting this complex issue.

## **Applicable Scope**

### 4-lane vs. 6-lane Arterials

Although the speed estimation model developed in this project is specific to 6-lane arterial cross sections, differences due to a 4-lane cross section can be described qualitatively. With respect to the weaving maneuver from the interchange off-ramp to the left-turn lane of the downstream signalized intersection, a 4-lane arterial requires one less lane change to make that maneuver; thus, this weaving maneuver can be accomplished in less distance than for a

six-lane arterial, all else being equal. For higher traffic volumes, however, it may be more difficult to make the necessary lane changes due to fewer available gaps to merge into. So for higher traffic volume conditions, the reduced availability of gaps for a 4-lane arterial may essentially negate the reduced number of lane changes.

For a 4-lane arterial, the impacts due to vehicles turning right into a driveway may be greater as there is only one other lane for through vehicles to use. Thus, the travel speed in the right-hand lane of a 4-lane arterial is likely to be lower than that for a 6-lane arterial, all else being equal. This difference is likely to become smaller as traffic volumes increase, as the percentage of through vehicles using the right lane will increase since the other lanes will be slowed as well.

For a 4-lane arterial, the use of an open median is more feasible than for a 6-lane arterial. For higher volume conditions, a full median opening will likely lead to greater crash frequencies and rates for a 6-lane arterial due to the greater crossing distance and the extra lane of traffic that must be accounted for in the gap acceptance decision making process. When comparing a 4-lane arterial with a full median opening to a 6-lane arterial with no median opening or a directional median opening, the increased traffic friction created by the median opening is likely to offset any advantage in the reduced number of lane changes required for a weaving maneuver from the off-ramp to the left-turn lane of the first signalized intersection.

### Other Analysis Tools

The general guidelines provided in this report, and in most of the previous related studies, are in a tabular format. Guidelines summarized in tabular format provide a good starting point, but they have obvious limitations. They typically only account for a limited number of variables, and it can be difficult for the analyst to make evaluations across multiple tables. The obvious advantage to the use of tables is the computational ease and required level of knowledge. Although a model for speed estimation was developed that allows an analyst to extend an analysis beyond the general tables, it is still limited in scope, and is therefore intended to be applied at the planning level of analysis.

Given the complexity of operations at interchange areas, and the inherent uniqueness of each site, ideally, an advanced analysis tool should be used to facilitate a comprehensive evaluation. One such tool is microscopic simulation (discussed in Chapter 4). This tool

allows an analyst to examine the simultaneous effect of multiple variables and their interactions. However, the required knowledge level is very high and the data requirements are intensive. In some situations, microscopic simulation may be the only reasonable alternative; for example, in examining alternatives to mitigate existing operational deficiencies under heavily saturated volume conditions.

Nonetheless, the analyst must still use caution when interpreting the output from simulation models. In particular, one aspect of driveway operations that is very difficult to duplicate in a simulation model is the pressure effect. That is, as vehicles queue up behind the first vehicle in queue at a driveway, the driver of that vehicle feels more pressure to enter the arterial. Furthermore, under higher volume conditions when acceptable gaps for entering the arterial are minimal, a driver may begin to consider accepting smaller gaps than normal the longer they wait to enter the arterial. Thus, simulation does not necessarily account properly for driveway exit maneuvers during high congestion, when people feel pressured to get out onto street. The same principles can be applied to drivers waiting to make a left turn into a driveway. Despite the limitations of representing this type of reduced gap size acceptance behavior in a simulation model, it must still be understood that when traffic volumes build and driveway exit and entrance queues build, there likely will be negative impacts to driveway safety as well as arterial speed reductions. Although outside of the scope of this study, a combination of predicted travel speed reduction and driveway inbound and outbound volumes can possibly be used as a surrogate for the potential impact on driveway safety. However, even with the limitations of simulation, it is still one of the best tools available for modeling complex situations.

For interchange planning purposes, an intermediate-level tool may provide the best compromise between analyst experience, data requirements, and analysis accuracy. An example of such a tool is the LOSPLAN suite of software programs provided by the FDOT Systems Planning Office. One program in this suite is designed to perform arterial level of service analyses at a planning level (ARTPLAN). Another program is designed to perform freeway level of service analyses at a planning level (FREEPLAN). Components of these two programs could potentially be combined to provide for the analysis of interchange operations, at a planning level. With current, and proposed future enhancements, several areas critical to the assessment of interchange operations could be addressed, such as off-ramp queue backup,

left-turn bay spillover, arterial weaving, mid-block friction effects, signal spacing, etc. This type of tool would allow an analyst to perform a more comprehensive assessment of traffic operations at an interchange. That is, operations at the interchange ramp terminals and upstream and downstream in both directions for several intersections can be simultaneously considered. Since the input requirements are much less burdensome with a planning-level application, it would be considerably easier to use and apply than a microscopic simulation package. On the other hand, with the reliance on default values for a number of input parameters, it is generally not appropriate for use in detailed operational analyses.

Another type of tool that might prove particularly useful in this area is an expert system. Given that there is still much to be learned in this area, and that there are a relatively limited number of acknowledged experts in this area, such a system could be quite beneficial to those less experienced in this area. One of the main challenges in developing an expert system, however, is to extract the relevant knowledge from the human experts. This knowledge is typically heuristic in nature and often based on “proven” “rules of thumb” rather than absolute certainties. Assuming this can be successfully done, the developed expert system can guide an inexperienced analyst through the logical analysis approach that the expert analyst would typically follow, and considering all of the aforementioned critical issues.

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## Appendix A: Newberry Road Heavy Vehicle Percentages

<b>Friday (4/29/05)</b>	EB	WB	NB	SB
NB Ramp	0.87%	0.97%	1.25%	-
NW 69 <sup>th</sup> Terrace	0.37%	0.94%	0%	0%
Oaks Mall West	0.41%	0.55%	0%	0%
NW 66 <sup>th</sup> Street	0.47%	0.52%	0%	0.64%

<b>Saturday (4/30/05)</b>	EB	WB	NB	SB
NB Ramp	0.83%	0.70%	1.21%	-
NW 69 <sup>th</sup> Terrace	0.30%	0.96%	0.96%	0.90%
Oaks Mall West	0.70%	0.94%	0.28%	0%
NW 66 <sup>th</sup> Street	0.51%	0.66%	0%	3.08%

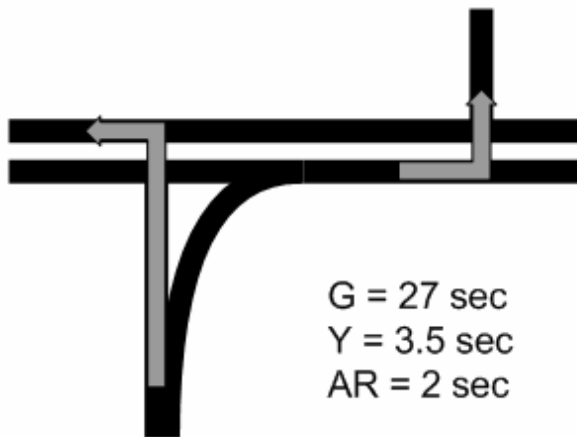
<b>Tuesday (5/03/05) midday</b>	EB	WB	NB	SB
NB Ramp	2.82%	1.31%	7.87%	-
NW 69 <sup>th</sup> Terrace	0.00%	1.58%	0.00%	0.22%
Oaks Mall West	1.44%	1.49%	0.79%	0%
NW 66 <sup>th</sup> Street	1.03%	1.10%	2.65%	0.52%

## **Appendix B: Newberry Road Signal Phasing Diagrams**



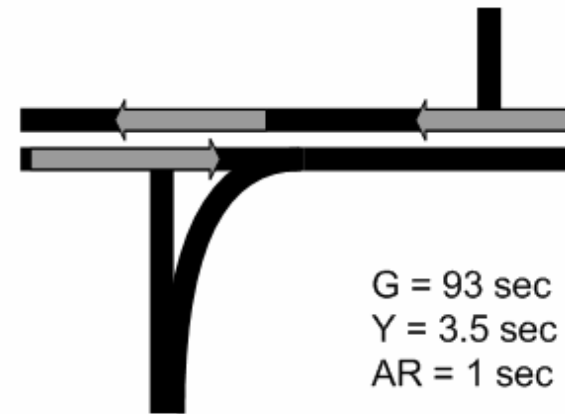
## I-75 NB Ramps (Intersection 1)

Phase 1



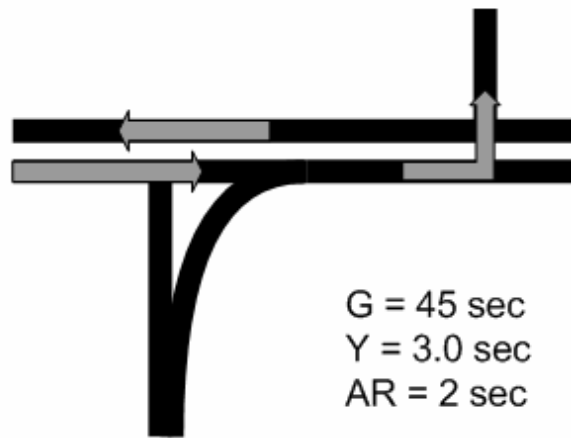
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Y = 3.5 sec  
AR = 2 sec

Phase 2



G = 93 sec  
Y = 3.5 sec  
AR = 1 sec

Phase 3

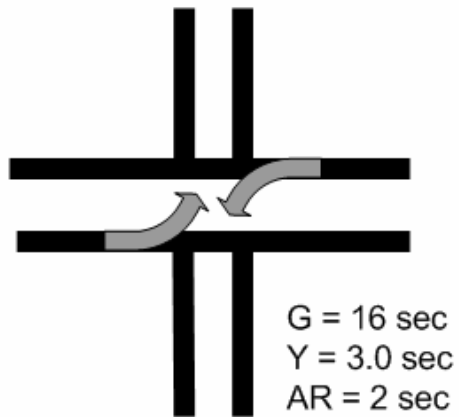


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AR = 2 sec

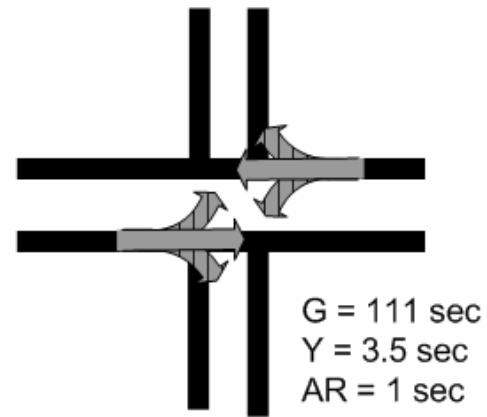


## NW 69<sup>th</sup> Terrace (Intersection 2)

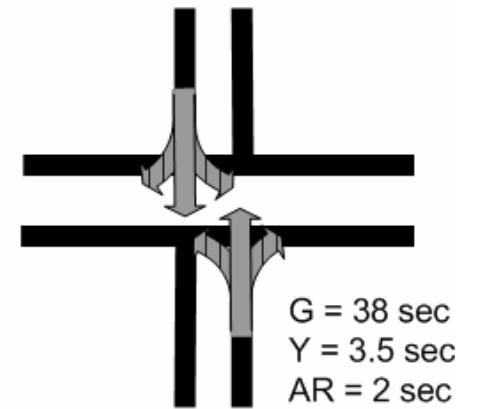
Phase 1



Phase 2



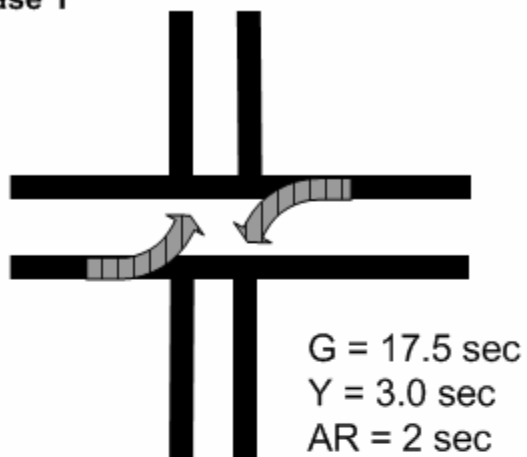
Phase 3



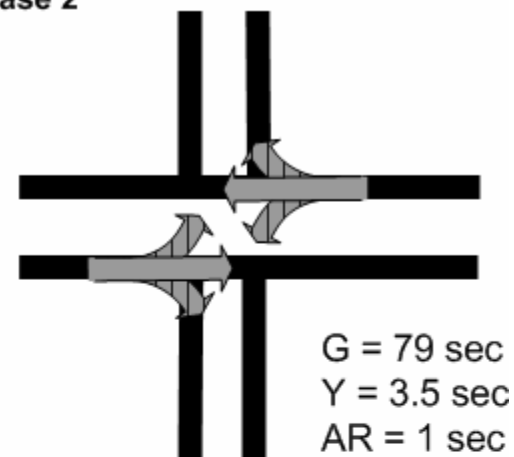


Oaks Mall West  
(Intersection 3)

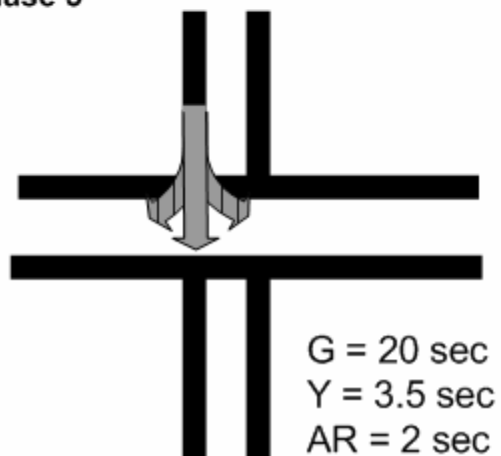
Phase 1



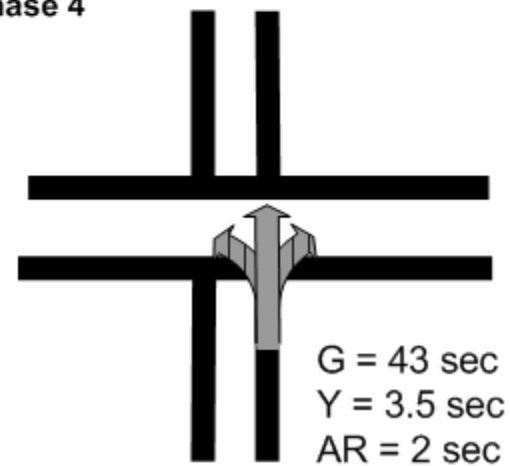
Phase 2



Phase 3



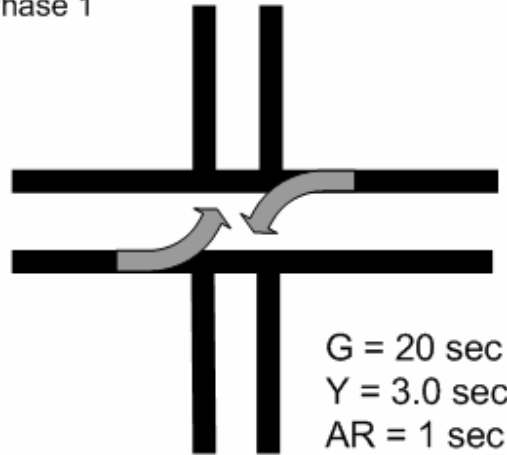
Phase 4





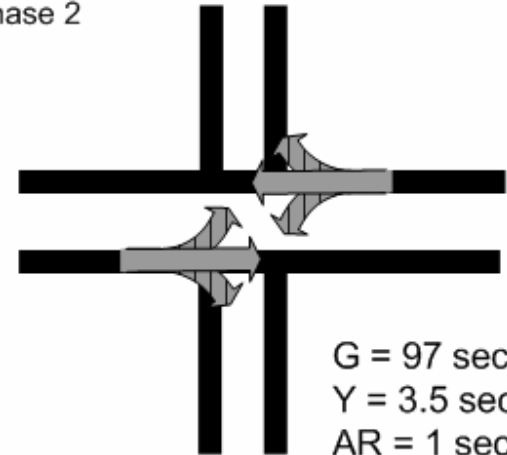
**NW 66<sup>th</sup> Street  
(Intersection 4)**

Phase 1



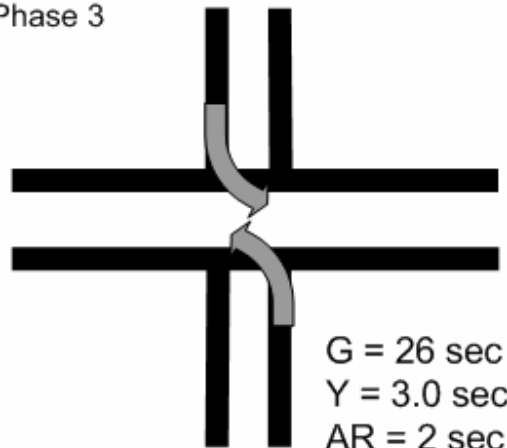
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Phase 2



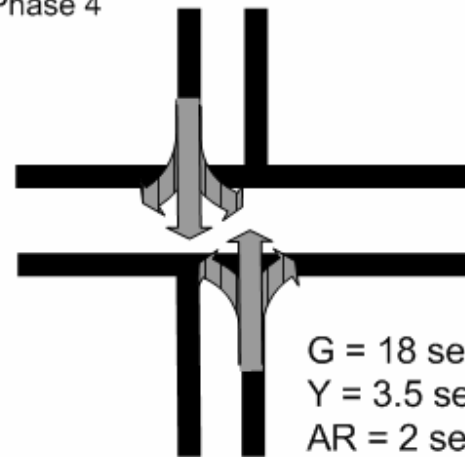
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Phase 3



G = 26 sec  
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Phase 4

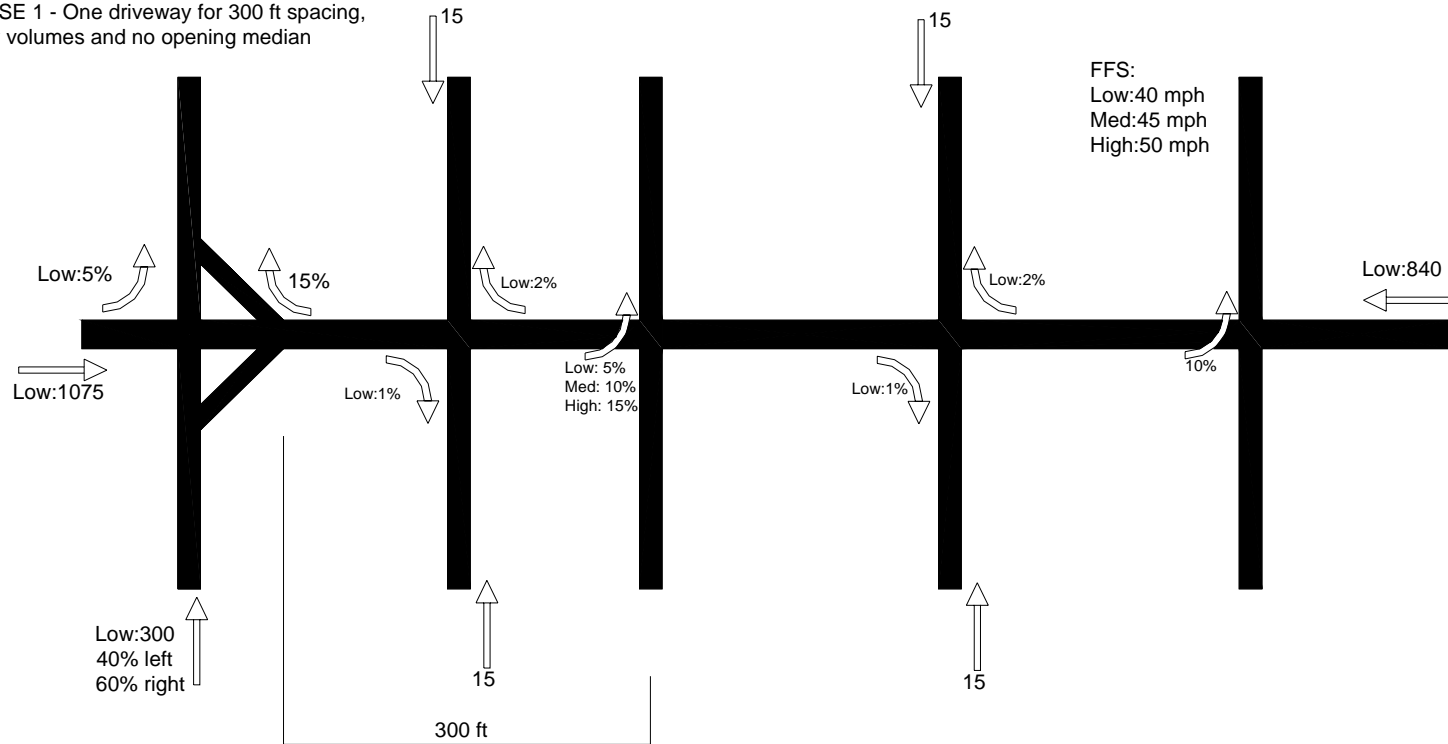


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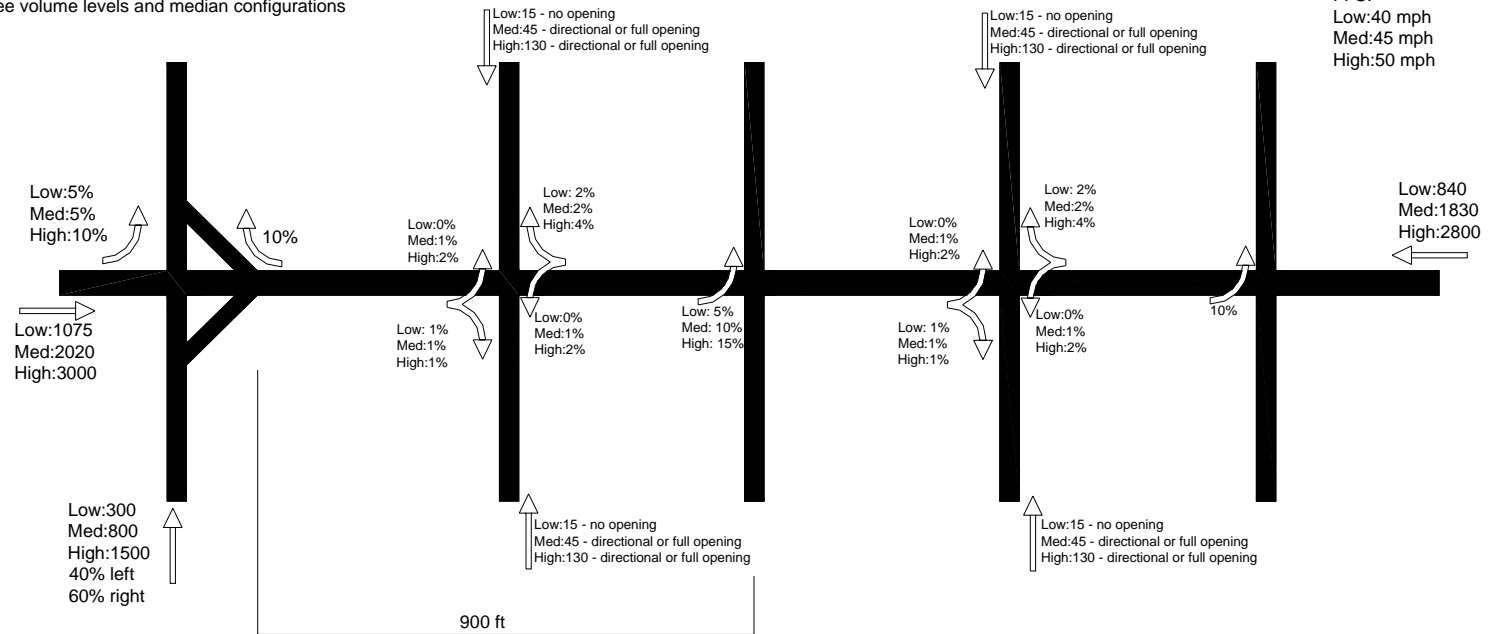
## **Appendix C: Simulation Experimental Design Network Configurations**



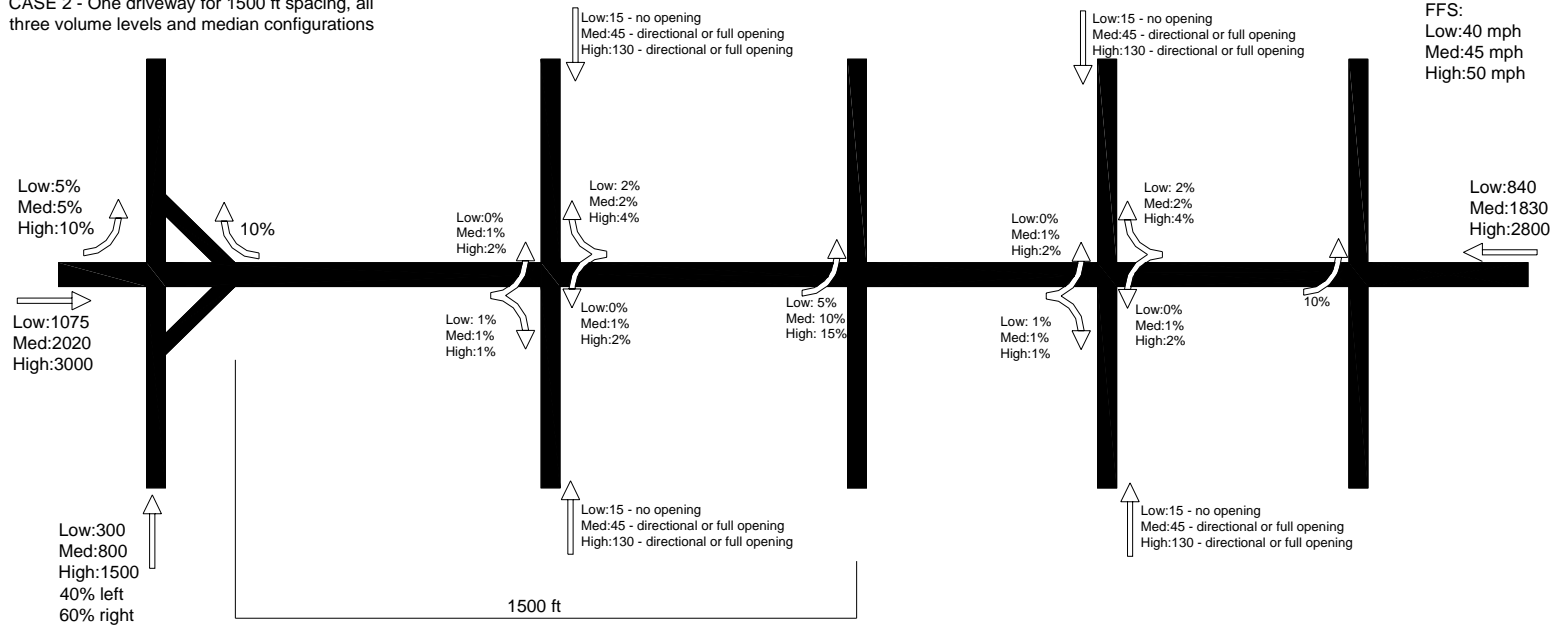
CASE 1 - One driveway for 300 ft spacing,  
low volumes and no opening median



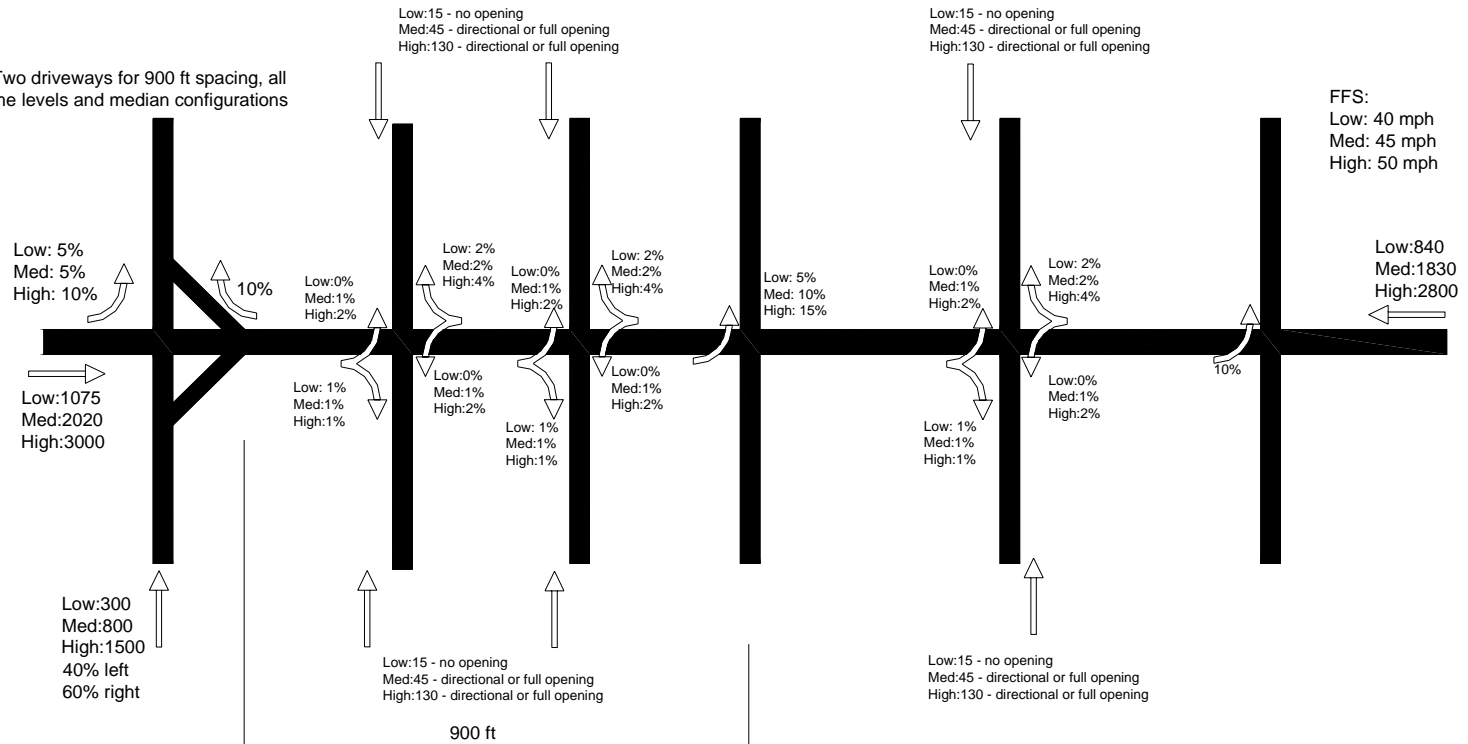
CASE 2 - One driveway for 900 ft spacing, all three volume levels and median configurations



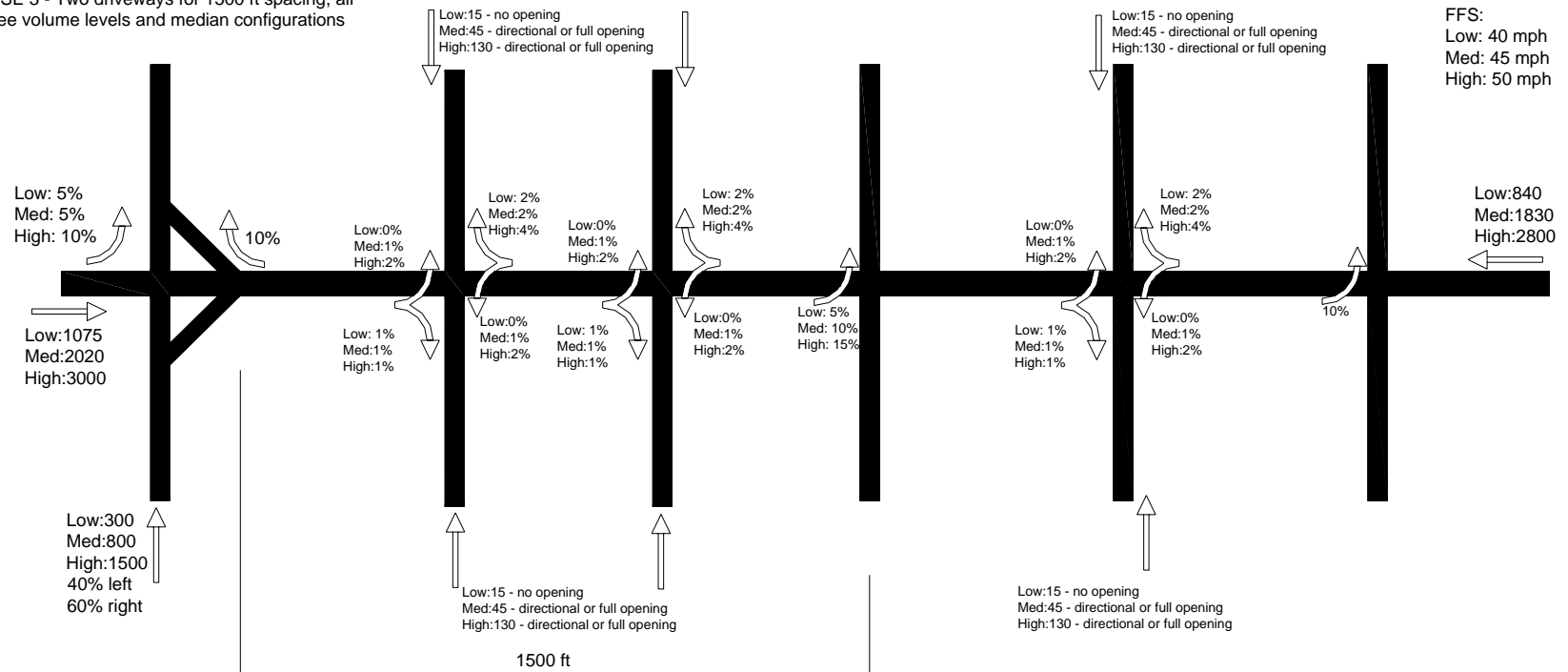
CASE 2 - One driveway for 1500 ft spacing, all three volume levels and median configurations



CASE 3 - Two driveways for 900 ft spacing, all three volume levels and median configurations

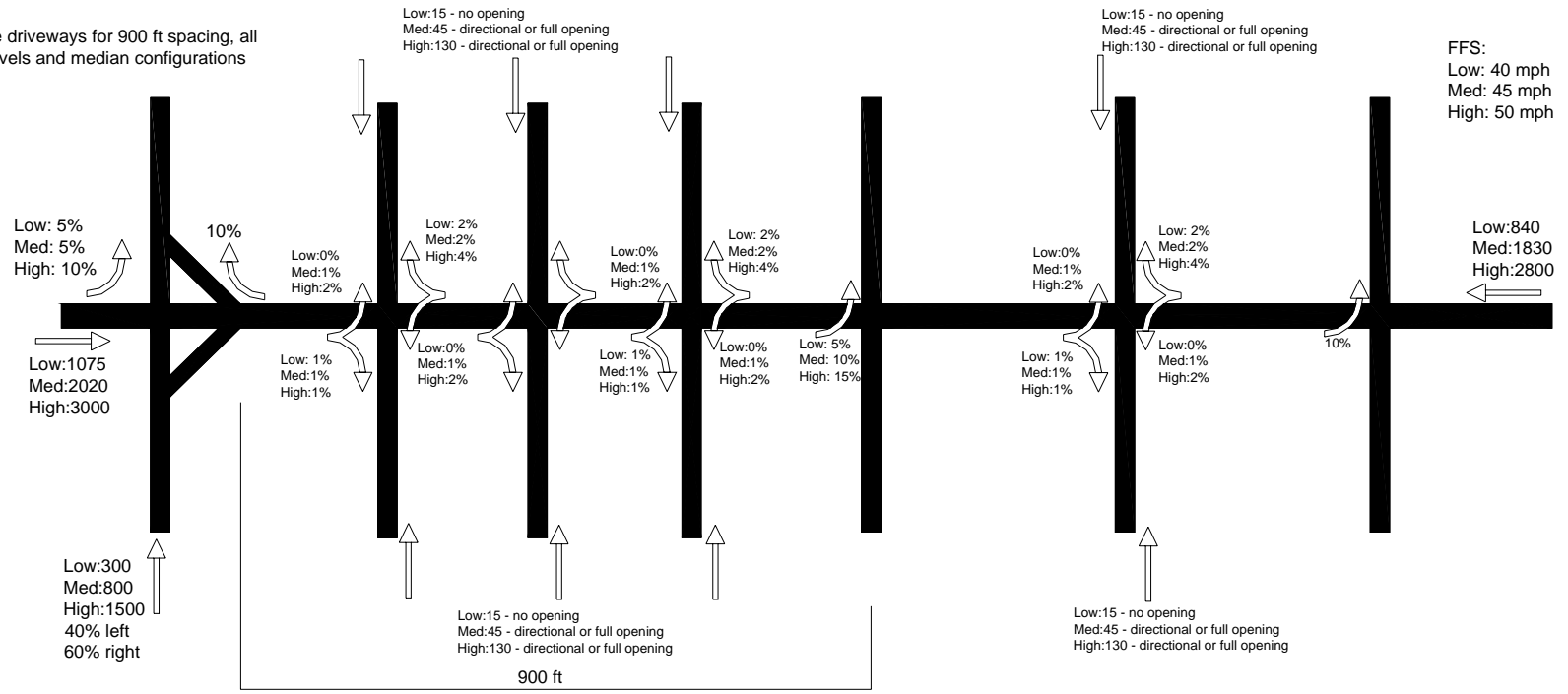


CASE 3 - Two driveways for 1500 ft spacing, all three volume levels and median configurations

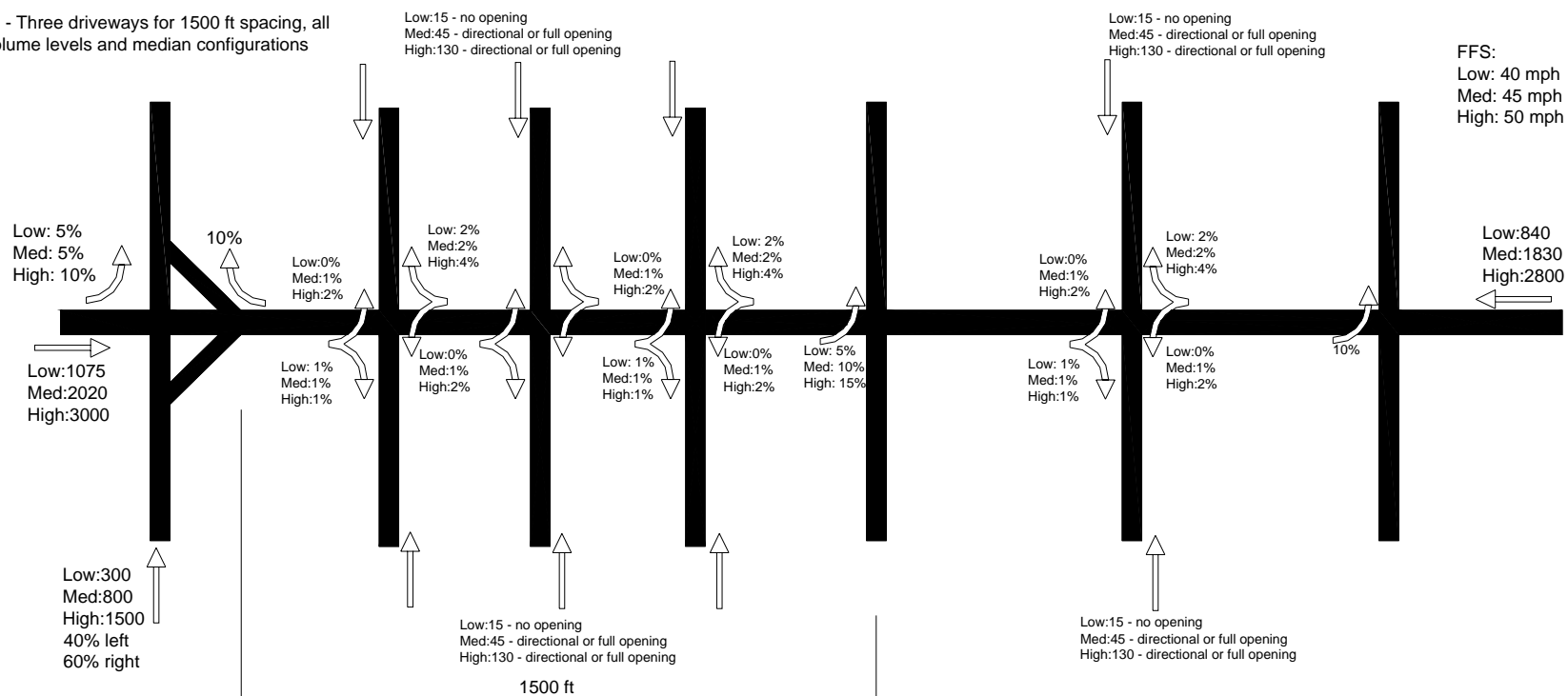


FFS:  
Low: 40 mph  
Med: 45 mph  
High: 50 mph

CASE 4 - Three driveways for 900 ft spacing, all three volume levels and median configurations



CASE 4 - Three driveways for 1500 ft spacing, all three volume levels and median configurations



## **Appendix D: Description of Software Tool for Guidance on Signal Spacing**



## Average Speed Estimation

The ‘Speed Estimator’ page of this software tool allows you to estimate the average speed on the arterial link between the interchange off-ramp and the first downstream signalized intersection (based on equation 10 and the other assumptions). The other feature on this page allows the analyst to get an estimate of the required distance between the interchange off-ramp and the first downstream signalized intersection, given a user input upper threshold speed and a design percent reduction in average speed. A picture of the user interface for this tool is shown in Figure 43.

**Interchange Signal Spacing Guidance**

Speed Estimator | Progression Index Calculator

**Output Options**

- ☐ Average Speed Estimate
- ☒ Signal Distance Recommendation
- ☒ % Speed Reduction vs Signal Distance Graph

**Estimation Parameters**

Upper Threshold Speed (default = FFS - 10)

Lower Threshold Speed

Design % reduction in average speed

**Input Parameters**

Total Directional Arterial Volume (veh/h)

Number of Driveways

Average Driveway Inbound Volume (veh/h)

Average Driveway Outbound Volume (veh/h)

% Left Turns at first downstream signal

Distance to First Downstream Signal (ft)

Free Flow Speed (mi/h)

**Results**

Average Speed (mi/h)

Recommended Signal Distance (ft)

**Figure 43. User Interface for Software Tool (Page 1)**

The upper threshold speed is the speed the analyst considers to be a realistic maximum upper speed for the given roadway and control conditions (i.e., under negligible traffic demands). By default, this speed is set to the free-flow speed (FFS) minus 10 mi/h, but it can be overridden by the analyst. Note that the free-flow speed is usually taken to be posted speed limit plus 5 mi/h, unless field data indicate otherwise.

The percent reduction in speed is a value between 0-100%. This value is used in combination with the upper threshold speed to define the average speed upon which to base the downstream signal location on. For the data shown in the user interface, the target design speed is:

$$35 \times (1 - 0.20) = 28 \text{ mi/h}$$

Equation 10 is rearranged and solved for the signal distance using this target speed, as follows.

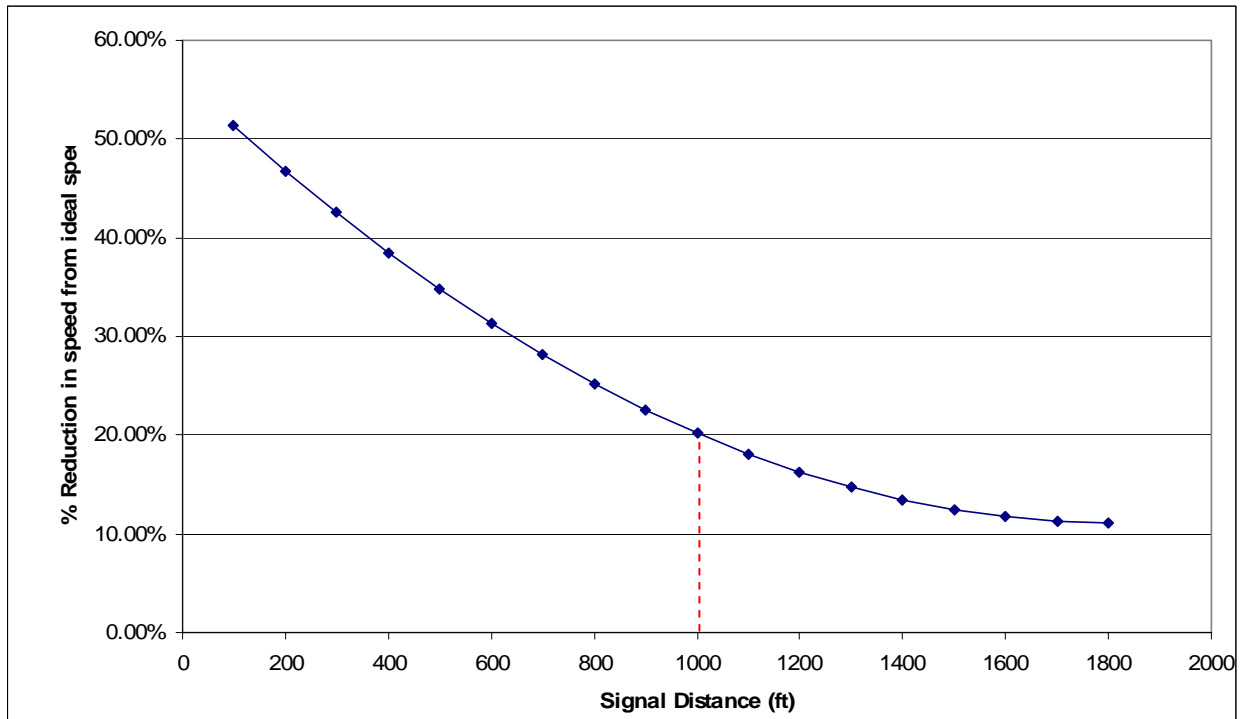
$$28 = 11.57 - 0.4572 (2400/100) - 0.0099 (2 \times 75) - 0.0117 (2 \times 75) + 0.8307 (2) + 1.7377 (SigDist/100) - 0.0479 (SigDist/100)^2 - 19.29 (10/100) + 0.405 (45)$$

$$1.7377 (SigDist/100) - 0.0479 (SigDist/100)^2 = -11.57 + 10.973 + 1.491 + 1.758 - 1.661 + 1.929 - 18.245 + 28$$

$$-0.0479(SigDist/100)^2 + 1.7377(SigDist / 100) - 12.675 = 0$$

Given the quadratic form of the equation, there are two possible solutions. The two solutions to this equation are 10.11 and 26.16 (i.e., 1011 and 2616 ft). The shorter distance of the two is the answer, provided it is not negative.

A graph of the relationship between signal location and percent reduction in speed can also be displayed by pressing the 'View Graph' button. Note that this feature is enabled by selecting the check box for '% Speed Reduction vs Signal Distance Graph' under the 'Signal Distance Recommendation' option. For the above example, the corresponding graph is shown below, with a dashed line overlaid to indicate the result.



**Figure 44. Graph of Percent Speed Reduction versus Signal Distance**

### Progression Quality Indicator

The second page of this software tool implements the spacing index calculations described in Chapter 5. Entering values for cycle length, travel speed, and signal spacing and then pressing the ‘Calculate Results’ button will provide the spacing index value, which again is an indication of how ideal or non-ideal these conditions are for accommodating two-way progression. This is shown in Figure 45. Note that the analyst can change the units of speed or spacing by pressing the units buttons to the right of the input boxes.

**Figure 45. User Interface for Software Tool (Page 2)**

Another feature with this tool allows an analyst to calculate the ideal signal spacing for a given cycle length and travel speed. This is done according to the following formula.

$$\text{Ideal Signal Spacing} = \frac{\text{Cycle Length}}{2} \times \text{Travel Speed} \quad [11]$$

Where Cycle Length is in units of seconds, Travel Speed in units of ft/sec, and Ideal Signal Spacing in units of feet. For example, the ideal signal spacing for a cycle length of 90 seconds and a travel speed of 40 mi/h (58.68 ft/s) is,

$$\text{Signal Spacing} = \frac{90 \text{ s}}{2} \times 58.68 \text{ ft/s} = 2640 \text{ ft} \quad [12]$$

These results are shown in Figure 46.

**Interchange Signal Spacing Guidance**

Speed Estimator | Progression Index Calculator

Output Options

☐ Spacing Index

☒ Optimal Spacing

Input Parameters

Cycle Length (sec)

Travel Speed  mi/h

Signal Spacing  ft

Results

Travel Time (sec)

Spacing Index

Recommended Signal Distance (ft)

Calculate Results

**Figure 46. Software Tool Display for Ideal Signal Spacing Calculation Results**