

1. Report No. FHWA/TX-04/1752-14		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle A MODEL FOR EVALUATING INTEGRATION STRATEGIES FOR OPERATING DIAMOND INTERCHANGE AND RAMP METERING				5. Report Date September 2003	
				6. Performing Organization Code	
7. Author(s) Zong Tian and Kevin Balke				8. Performing Organization Report No. Report 1752-14	
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. Project No. 0-1752	
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080				13. Type of Report and Period Covered Research: September 2002 – August 2003	
				14. Sponsoring Agency Code	
15. Supplementary Notes Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. Research Project Title: A Model for Evaluating Integration Strategies for Operating Diamond Interchange and Ramp Metering					
16. Abstract Diamond interchanges and their associated ramps are where the surface street arterial system and the freeway system interface. Historically, these two elements of the system have been operated with little or no coordination between the two. One danger of operating these two systems in isolation is that traffic from the ramp, particularly if it is metered, can spill back into the diamond interchange, causing it to become congested. The aim of this research was to develop integrated operational strategies for managing the diamond interchange and ramp metering operations for the purpose of improving system performances. Modeling methodologies were developed for analyzing an integrated diamond interchange – ramp metering system (IDIRMS). A computer model named DRIVE was developed, which is classified as a mesoscopic simulation model. The model was validated against the VISSIM microscopic simulation model, and researchers found general agreement between the two models. Operational characteristics were also investigated using DRIVE to gain better understanding of the system features. Integrated operational strategies were developed and evaluated under various traffic flow conditions. The analysis results indicate that with integrated operations through an adaptive signal control system, the onset of freeway congestion and breakdown is effectively postponed.					
17. Key Words Diamond Interchange, Ramp Metering, Integration, DRIVE, Simulation			18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161		
19. Security Classif.(of this report) Unclassified		20. Security Classif.(of this page) Unclassified		21. No. of Pages 80	22. Price

A MODEL FOR EVALUATING INTEGRATION STRATEGIES FOR OPERATING DIAMOND INTERCHANGE AND RAMP METERING

by

Zong Tian
Associate Transportation Researcher
Texas Transportation Institute

and

Kevin Balke, Ph.D., P.E.
Research Engineer
TransLink[®] Research Center Director
Texas Transportation Institute

Report 1752-14
Project Number 0-1752
Research Project Title: A Model for Evaluating Integration Strategies
for Operating Diamond Interchange and Ramp Metering

Sponsored by the
Texas Department of Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

September 2003

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. The engineer in charge was Kevin Balke, P.E. (Texas, #66529).

ACKNOWLEDGMENTS

This project was conducted as part of the TransLink[®] research program and was performed in corporation with the Texas Department of Transportation and the Federal Highway Administration. The project team recognizes the following TransLink[®] partners for their generous support of the TransLink[®] research program:

- U.S. Department of Transportation, Federal Highway Administration;
- Texas Department of Transportation;
- Metropolitan Transit Authority of Harris County;
- Texas Transportation Institute; and
- Rockwell International.

The project team would also like to recognize the following individuals for their support of this specific project:

- David Gibson, Federal Highway Administration;
- Mark Olson, Federal Highway Administration;
- Al Kosik, Traffic Operations Division, Texas Department of Transportation;
- Richard Reeves, Traffic Operations Division, Texas Department of Transportation;
- Sally Wegmann, Houston District, Texas Department of Transportation;
- Terry Sams, Dallas District, Texas Department of Transportation;
- Wallace Ewell, Fort Worth District, Texas Department of Transportation
- Pat Irwin, San Antonio District, Texas Department of Transportation

The authors would also like to recognize Dr. Carroll Messer, Dr. Nadeem Chaudhary, and Mr. Roelof Engelbrecht for the technical contributions and insights to this project.

TABLE OF CONTENTS

	Page
List of Figures.....	viii
List of Tables	ix
CHAPTER 1: INTRODUCTION.....	1
Problem Statement	2
Objectives of the Research	3
Scope of Research.....	4
Organization of Report	5
CHAPTER 2: SYSTEM MODELING METHODOLOGIES	7
IDIRMS Numbering Scheme.....	7
Diamond Interchange.....	10
Freeway and Ramp Metering.....	14
Ramp Flow Profiles	17
Ramp Queue Spillback	22
CHAPTER 3: DEVELOPMENT OF THE MODELING SOFTWARE.....	29
CHAPTER 4: MODEL VALIDATION.....	33
Site Description.....	33
VISSIM Model Development.....	33
Model Calibration and Validation Results.....	35
CHAPTER 5: ANALYSES ON SYSTEM OPERATIONAL CHARACTERISTICS	41
Effects of Diamond Signal Phasing on Ramp queues	41
Effects of Diamond Cycle Length on Ramp Queues.....	43
Effects of Ramp Metering Operations on Freeway Performance.....	45
CHAPTER 6: INTEGRATION STRATEGIES	49
Resource Management Philosophy.....	49
The System Operating Objectives	50
Integration Strategies	51
Evaluation of Operational Strategies under Recurring Congestion.....	61
A Framework for Selecting Operational Strategies.....	64
CHAPTER 7: SUMMARY AND CONCLUSIONS	67
REFERENCES.....	69

LIST OF FIGURES

	Page
Figure 1. Queue Spillback at a Diamond Interchange with Ramp Metering.....	2
Figure 2. Proposed Integrated System and Its Boundaries.....	5
Figure 3. System Elements and Numbering Schemes.	8
Figure 4. Delays and Queues for the Case with an Initial Queue and a Residual Queue.....	12
Figure 5. Ramp Arrival Flow Profile with Basic Three-Phase.....	18
Figure 6. Ramp Arrival Flow Profile with TTI Four-Phase.	19
Figure 7. Ramp Arrival Flow Profile without Arterial Right-Turn Control.....	21
Figure 8. Ramp Arrival Flow Profile with Arterial Right-Turn Control.	22
Figure 9. Feeding Movement Service Sequences and Ramp Flow Profiles.....	27
Figure 10. DRIVE Modules and Functions.	30
Figure 11. DRIVE Workflow Chart.....	32
Figure 12. Ramp and Diamond Interchange Delays with Basic Three-Phase and Queue Flush (Average of 5 Runs).....	36
Figure 13. Freeway Mainline Delay with Queue Flush.....	37
Figure 14. Ramp and Diamond Interchang Delays with Basic Three-Phase without Queue Flush (Average of 5 Runs).....	38
Figure 15. Freeway Mainline Delay without Queue Flush.....	39
Figure 16. Oversaturated Case with Queue Spillback: Basic Three-Phase.....	39
Figure 17. Oversaturated Case with Queue Spillback: TTI Four-Phase.....	40
Figure 18. Queue Length Comparison with Basic Three-Phase and TTI Four-Phase.	42
Figure 19. Traffic Demand Evolution with Queue Spillback - Basic Three-Phase.....	43
Figure 20. Traffic Demand Evolution with Queue Spillback - TTI Four-Phase.	43
Figure 21. Effect of Cycle Length on Ramp Queue Length.	44
Figure 22. Freeway Throughput with Responsive Metering Rate.....	46
Figure 23. Freeway Throughput with Fixed Metering Rate and Queue Flush.	46
Figure 24. Freeway Throughput with Fixed Metering Rate and without Ramp Queue Flush.	47
Figure 25. Additional Detector Layouts for the Proposed Adaptive Control System.....	54
Figure 26. Hold Internal Phases with Basic Three-Phase.....	56
Figure 27. Hold Arterial Phases with Basic Three-Phase.....	56
Figure 28. Hold Frontage Road Phase with Basic Three-Phase and Conditional Service.	57
Figure 29. Hold Frontage Road Phase (ϕ_8) to Control Left-Side Ramp Entry with TTI Four- Phase.....	58
Figure 30. Hold Arterial Phase (ϕ_2) to Control Left-Side Ramp Entry with TTI Four-Phase.....	59
Figure 31. Incident within Interchange.....	60
Figure 32. Incident Downstream of Interchange.	61
Figure 33. Queue Flush Duration with/without Integration – Low Demand.....	62
Figure 34. Queue Flush Duration with/without Integration – High Demand.....	63
Figure 35. Delay Evolution on Freeway with/without Integration.....	63

LIST OF TABLES

	Page
Table 1. Diamond Interchange Movements and Freeway On-Ramp Demands.	9
Table 2. Proportion of Traffic Feeding Freeway On-Ramps.....	10
Table 3. Determination of $Q_M(t)$ Values.....	24
Table 4. Input/Output Information for DRIVE Software.	31

CHAPTER 1: INTRODUCTION

Freeway interchanges establish interconnections between freeway systems and surface street arterials and provide the backbone of transportation networks. One of the most commonly used interchange types in Texas is the tight urban diamond interchange (TUDI), where two traffic signals are installed on the arterial street to control the interchanging traffic (1,2). Diamond interchanges are often characterized by unique traffic flow patterns, especially high turning movements and limited spacing between the signals that make managing their operations difficult. To complicate matters, the majority of freeway ramp meters installed in Texas are located in the vicinity of diamond interchanges. As a result, diamond interchange locations are often sources of operational bottlenecks for both surface street arterials and freeways.

One operational issues existing today is that the diamond interchange and ramp metering are primarily treated as independent elements. Traffic engineers and planners typically do not consider the interactions between these two elements, nor do they consider the potential benefits that can be derived from coordinating their operations (3). The lack of system integration or coordination between the diamond signals and the ramp metering signal often creates major operational concerns, among which queue spillback from the metered ramp is the most obvious one. This situation is illustrated in Figure 1.

During typical rush hours, high traffic demands on the freeway often require restricted entry of traffic from the metered ramp, thus resulting in long queues on the ramp. The fact that the traffic released from the upstream diamond signal arrives in platoons also exacerbates the queue spillback effect, where limited storage spacing on the ramp cannot accommodate the short-term surge of large platoons. Unless the signal controller at the upstream diamond interchange has some way to sense the queue buildup, traffic would continue to flow to the ramp, until the queue spills back to the surface street (e.g., frontage road or the diamond signal location). Such a queue spillback would interfere with the surface street operation and may create safety concerns. Suggested strategies to control queue spillback generally involve some queue override policies to flush the ramp queues by either increasing the metering rate or terminating metering operations (4). However, such an operation may lead to a freeway *breakdown*; a phenomenon indicated by a sudden speed drop and perhaps a drop in the flow. A

breakdown of the freeway affects the efficiencies of the entire system. The purpose of this project is to begin the process of investigating whether providing integrated operations between a diamond interchange and a ramp meter would reduce the deficiencies (i.e., to minimize queue spillback and queue flush at the ramp meter) of the current independent operations by developing appropriate integrated operational strategies on diamond control and ramp metering.

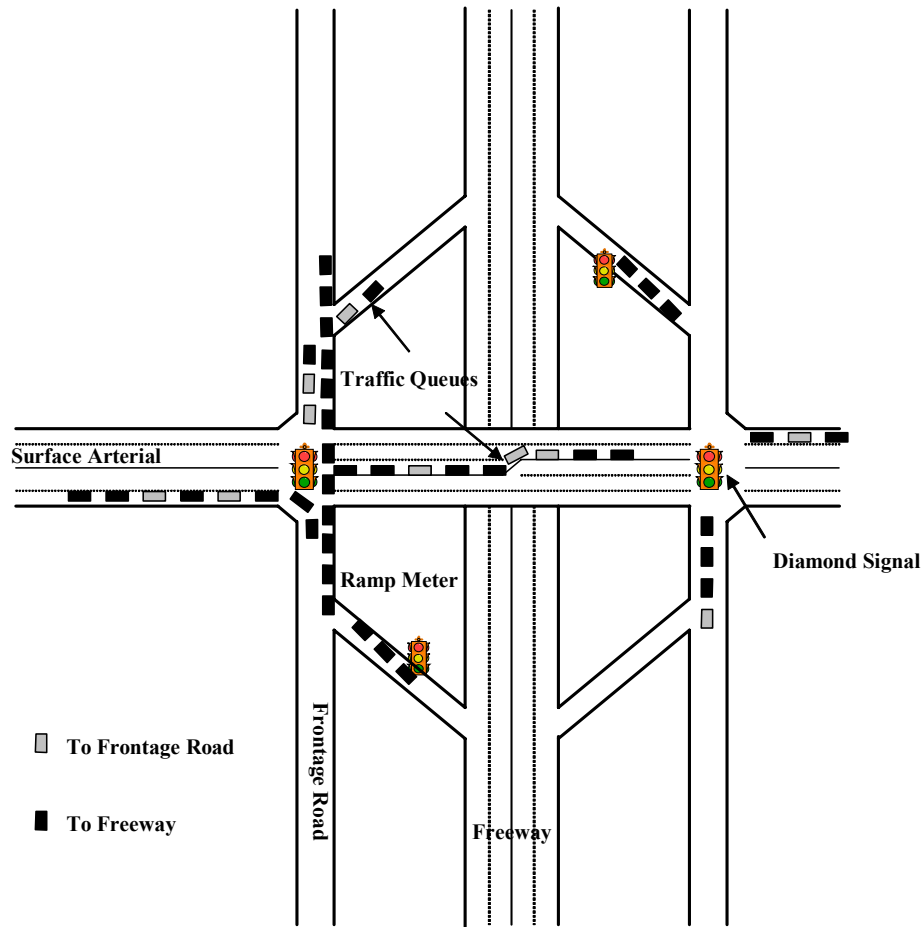


Figure 1. Queue Spillback at a Diamond Interchange with Ramp Metering.

PROBLEM STATEMENT

Current operations at a diamond interchange and a ramp meter lack system coordination between the two components. The lack of system coordination is reflected by the fact that little consideration is currently given to diamond operational strategies that minimize or eliminate ramp queue spillback when ramp metering is in operation. Existing diamond interchange

strategies focus on serving traffic demands from external approaches, which are monitored by various traffic sensors. Appropriate signal phasing and timing are then developed to best serve the traffic demands (5,6). However, existing diamond operations completely ignore the constraints imposed by the downstream ramp meter. Excessive and non-controlled release of traffic from the diamond often results in queue spillback at the ramp meter. On the other hand, the effectiveness of the ramp metering (both the algorithm design and the operational strategy) also plays an important role in controlling queue spillback (7,8,9).

Queue spillback resulting from the lack of coordination between the ramp meter and diamond interchange creates serious operational concerns on the diamond interchange and the surface street arterial. Although queue override policies currently being used at ramp meters can eliminate queue spillback, frequent queue flush can lead to freeway breakdown and diminish the main purpose of ramp metering. Therefore, a need exists to address the diamond interchange, ramp metering, and freeway components in an integrated and coordinated manner to eliminate the deficiencies of the current operations. Integrated operational strategies need to be developed to minimize queue spillback occurrences at the ramp meter while maintaining optimal system throughput.

OBJECTIVES OF THE RESEARCH

The goals of this research are:

- (a) to develop a methodology for analyzing the operations of an integrated diamond interchange/ramp-metering system (IDIRMS); and
- (b) to investigate strategies for operating diamond interchanges and ramp-metering systems in an integrated fashion that would reduce the deficiencies of the current operations.

Specific objectives of this research include the following:

- Develop analytical procedures for estimating various performance measures (e.g., ramp queue length and system delays) for the integrated diamond interchange/ramp-metering system, given a set of system variables and parameters (e.g., ramp-metering rates, traffic demand profile on both freeway and diamond interchange approaches, diamond signal timing and geometric information such as spacing between diamond and ramp meter).

The overall methodology can be applied for system operations analysis, development and

evaluation of integrated operational strategies aimed at minimizing queue spillback occurrences on the metered ramp.

- Use VISSIM (10), a well-calibrated microscopic simulation model, to validate the analytical procedures by comparing the performance measures produced from both the analytical procedures and microscopic simulation model.
- Establish a framework and identify viable integrated operational strategies for IDIRMS based on a set of established objectives and priorities. One example of an operational strategy is to manipulate the signal timing at the diamond interchange so that queue spillback can be minimized. The integrated strategies should take into account the close interactions between the diamond interchange signals and the ramp meter.
- Evaluate the framework and demonstrate its applicability that would allow traffic engineers and decision makers to evaluate specific operational strategies and provide tradeoff assessments on each strategy.

SCOPE OF RESEARCH

This project was intended to be the first steps into exploring the basic relationship and potential benefits that might be derived from operating a diamond interchange and the ramps immediately adjacent to that diamond interchange in a coordinated fashion. This research was focused on identifying the basic relationships and developing analytical tools that could be used in the future to assess operating diamond interchanges and ramp meters as a system. While the research project did examine strategies that could be used to achieve integrated operations, it was never intended to develop a system architecture, control logic, data flows, algorithms, or technologies that could be used in an actual operation.

For the purposes of this research project, we defined the boundaries of the system to be that shown in [Figure 2](#). This is a type of diamond interchange with one-way frontage roads, typically seen in urban Texas highways. The project also assumes that U-turn lanes are provided for both directions. The system includes a segment of freeway mainline, ramp meters, and a signalized diamond interchange.

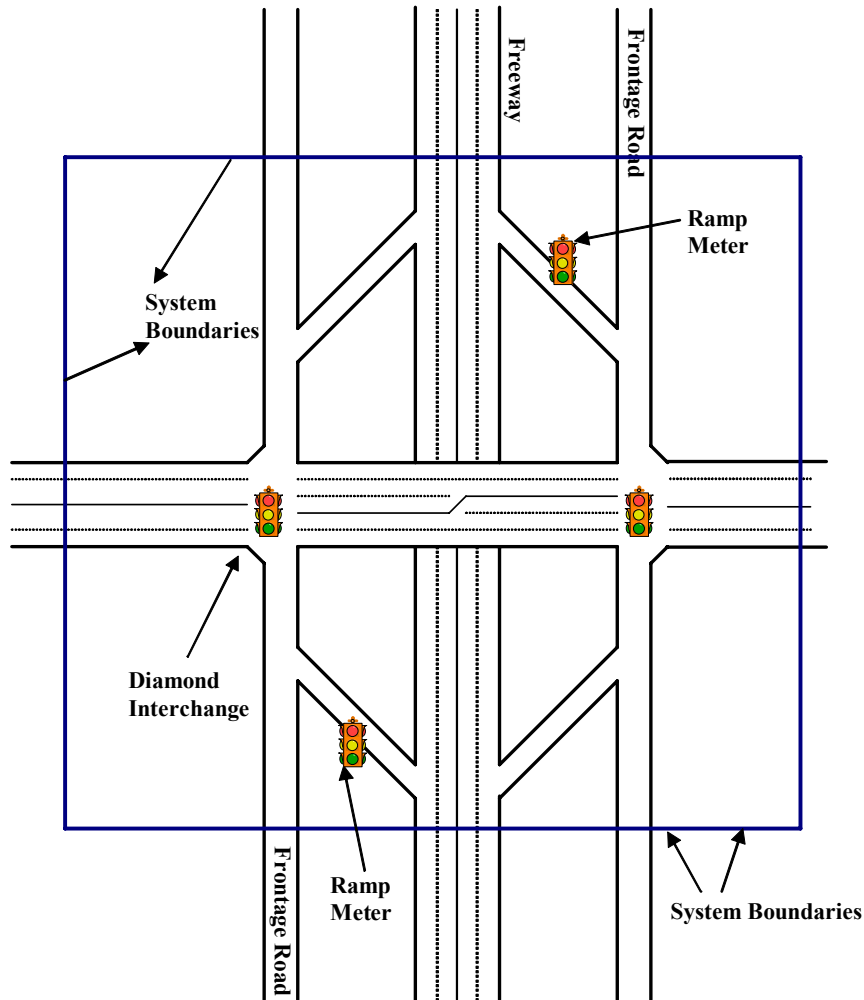


Figure 2. Proposed Integrated System and Its Boundaries

ORGANIZATION OF REPORT

This report includes a total of seven chapters, including this introductory chapter. In [Chapter 2](#) of this report, the researchers provide mathematical descriptions on the modeling process of an integrated diamond interchange/ramp-metering system. [Chapter 3](#) describes the development of computer software that implements the major modeling processes described in [Chapter 2](#). [Chapter 4](#) provides model validation results against both VISSIM microscopic simulation and PASSER III software. [Chapter 5](#) provides some analyses on the system operational characteristics using the software developed in this research. The purpose of these

analyses is to gain better understanding of the system features to facilitate the development of operational strategies. [Chapter 6](#) documents the development of operational strategies to achieve integrated operations between ramp metering and a diamond interchange. The operations are demonstrated through simulation and their effectiveness is evaluated. [Chapter 7](#) provides a summary and major conclusions of the research.

CHAPTER 2: SYSTEM MODELING METHODOLOGIES

This chapter documents the major modeling methodologies for the IDIRMS. The system and its boundaries are defined. A numbering scheme is proposed for the major origin-destination flows as well as major traffic movement flows in the system. The modeling processes for diamond interchange, freeway mainline, and ramp metering operations are described in mathematical equations. Special modeling efforts are conducted on the traffic arrival flow profiles at the ramp meters and on diamond interchange operations considering ramp queue spillback.

IDIRMS NUMBERING SCHEME

Figure 3 shows the numbering scheme for the major traffic flows within IDIRMS. The figure at the top includes the numbered origins (O_i) and destinations (D_j); turning traffic movements at the diamond interchange; and freeway mainline and on-ramp flows. These traffic flows provide necessary information on the traffic demand side for performing analyses on system performances. The figure at the bottom is the standard phasing scheme used in PASSER III. Once the origin-destination (OD) matrix is available, all other traffic flows can be derived, including 14 turning movements at the diamond interchange, two on-ramp flows, and two mainline flows. Table 1 and Table 2 summarize the relationships among various traffic movement flows and OD flows. The variables listed in the tables are defined below:

$o, d =$ index for origin and destination, $o \in 1 \sim 6, d \in 1 \sim 6$

$r =$ index for on-ramp and freeway, $r \in 1 \sim 2$

$v_{o,d} =$ traffic flow demand from origin o and destination d , vph

$V_m =$ traffic movement m at the diamond interchange, vph, $m \in 1 \sim 14$

$\tau_{ra} =$ the portion of diversion to ramp r during incident conditions

$\tau_{rb} =$ the portion of diversion to the downstream interchange during incident conditions

$R_r =$ traffic demand at ramp r , vph

$P_{m,r} =$ proportion of movement m that feeds ramp r

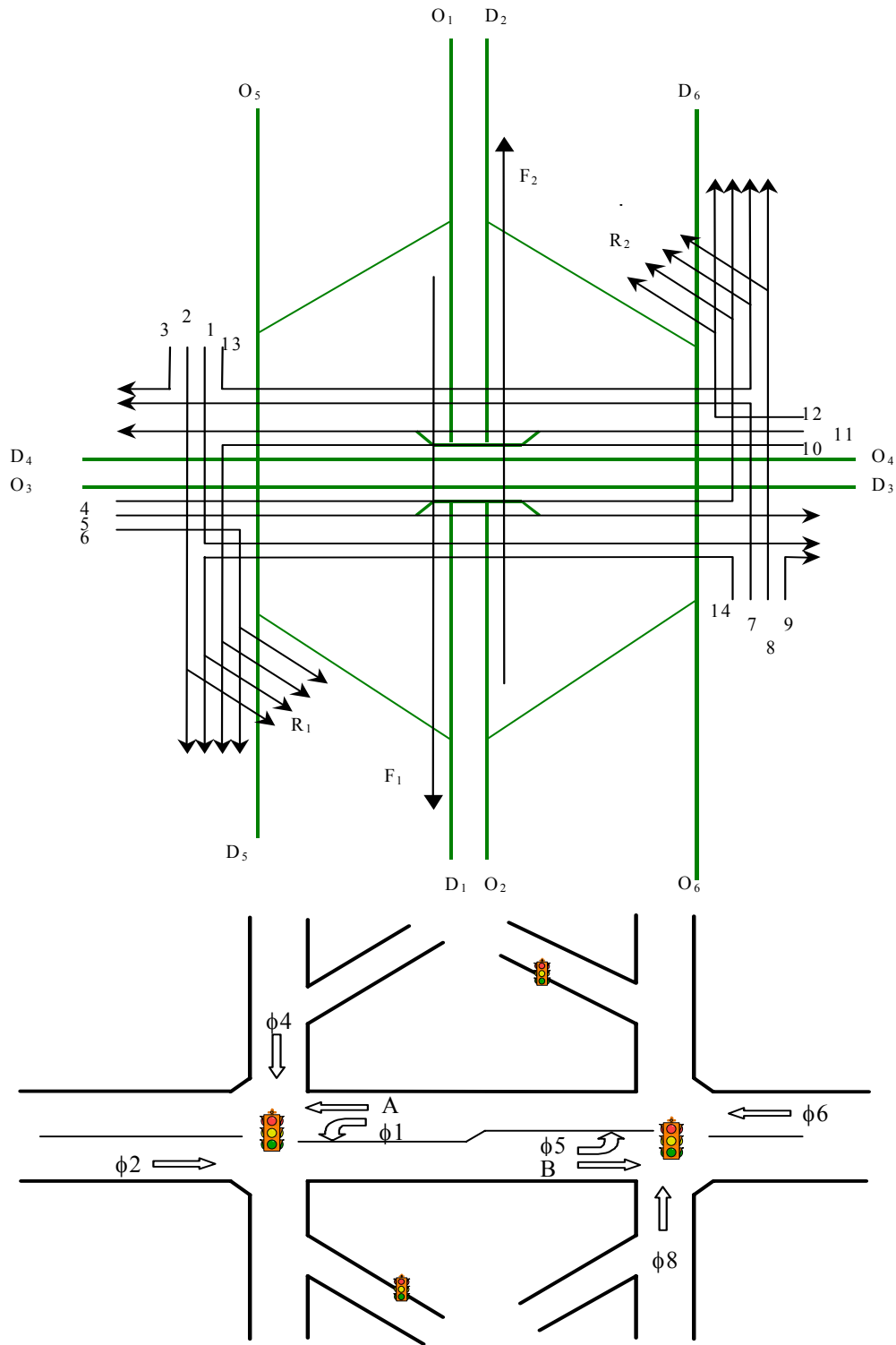


Figure 3. System Elements and Numbering Schemes.

Table 1. Diamond Interchange Movements and Freeway On-Ramp Demands.

Diamond/Ramp	Location	Movement	OD Source
<i>Diamond Interchange</i>	Frontage Road 1	LT	$V_1 = v_{1,3} + v_{5,3}$
		TH	$V_2 = v_{1,5} + v_{5,5} + v_{5,1} + (\tau_{1a} + \tau_{1b})v_{1,1}$
		RT	$V_3 = v_{1,4} + v_{5,4}$
		U	$V_{13} = v_{3,6} + v_{5,2} + v_{1,6}$
	Arterial A-Direction	LT	$V_4 = v_{3,2} + v_{3,6}$
		TH	$V_5 = v_{3,3}$
		RT	$V_6 = v_{3,1} + v_{3,5}$
	Frontage Road 2	LT	$V_7 = v_{6,4} + v_{2,4}$
		TH	$V_8 = v_{6,6} + v_{6,2} + v_{2,6} + (\tau_{2a} + \tau_{2b})v_{2,2}$
		RT	$V_9 = v_{6,3} + v_{2,3}$
		U	$V_{14} = v_{6,5} + v_{6,1} + v_{2,5}$
	Arterial B-Direction	LT	$V_{10} = v_{4,1} + v_{4,5}$
		TH	$V_{11} = v_{4,4}$
		RT	$V_{12} = v_{4,6} + v_{4,2}$
<i>Ramp</i>	Ramp 1	-	$R_1 = v_{3,1} + v_{4,1} + v_{5,1} + v_{6,1} + \tau_{1a}v_{1,1}$
	Ramp 2	-	$R_2 = v_{3,2} + v_{4,2} + v_{5,2} + v_{5,1} + v_{6,2} + \tau_{2a}v_{2,2}$
<i>Freeway</i>	Direction 1	-	$F_1 = v_{1,1}$
	Direction 2	-	$F_2 = v_{2,2}$

Table 2. Proportion of Traffic Feeding Freeway On-Ramps.

Feeding Ramp	Movement, m	Proportion, $p_{m,r}$
R ₁	V2	$p_{2,1} = \frac{v_{5,1} + \tau_{1a}v_{1,1}}{v_{5,1} + v_{5,5} + v_{1,5} + (\tau_{1a} + \tau_{1b})v_{1,1}}$
	V6	$p_{6,1} = \frac{v_{3,1}}{v_{3,1} + v_{3,5}}$
	V10	$p_{10,1} = \frac{v_{4,1}}{v_{4,1} + v_{4,5}}$
	V14	$p_{14,1} = \frac{v_{6,1}}{v_{6,1} + v_{6,5} + v_{2,5}}$
R ₂	V8	$p_{8,2} = \frac{v_{6,2} + \tau_{2a}v_{2,2}}{v_{6,2} + v_{6,6} + v_{2,6} + (\tau_{2a} + \tau_{2b})v_{2,2}}$
	V12	$p_{12,2} = \frac{v_{4,2}}{v_{4,2} + v_{4,6}}$
	V4	$p_{4,2} = \frac{v_{3,2}}{v_{3,2} + v_{3,6}}$
	V13	$p_{13,2} = \frac{v_{5,2}}{v_{5,2} + v_{5,6} + v_{1,6}}$

One critical element for analyzing the IDIRMS is the estimation of the OD flows. OD flows can be either obtained from an actual OD survey or estimated based on link and turning movement counts at specific locations. OD estimation is a subject that has attracted significant research interests and efforts (11,12,13) and is not addressed in this report. Readers can refer to these studies for further details.

DIAMOND INTERCHANGE

The methodology of modeling diamond interchange operations consists of analysis over multiple cycles, with consideration of stochastic traffic demands for each cycle. The methodologies for calculating capacities, delays, and queues at the diamond interchange follow similar methodologies as used in PASSER III (14); however, special considerations are given in modeling the effect of ramp queue spillback on diamond interchange operations.

Calculations on delays and queues use the standard arrival-departure queue polygon method, which is essentially calculating the uniform control delay and the initial queue delay

(i.e., the first term and the third term of delay equations) as documented in PASSER III and in the Highway Capacity Manual (15). The second-term delay, the random and over-saturation delay, is not necessary because the calculation is precisely for the duration of one cycle length, and randomness is accounted for in the stochastic flows generated each cycle. Figure 4 illustrates a general case where both an initial queue, N_m^{j-1} , and a residual queue, N_m^j , exist for a traffic movement m during cycle j . Other symbols in the figure are defined below:

$A_1, A_2, A_3 =$ total area (also total delay in veh-sec) during a portion of the cycle

$V_m^j =$ arrival rate for movement m during cycle j

$S_m =$ saturation flow rate for movement m , vph

$S_m' =$ departure flow rate when impeded by the ramp queue, vph

$N_m' =$ queue length at the start of green, veh

$N_m'' =$ queue length at the time when the ramp queue spills back and impedes the discharge of movement m , veh

$NR_m^j =$ the residual queue due to ramp queue spillback, veh

$NC_m^j =$ the residual queue due to movement m itself reaching over-saturation, veh

$r_m^j =$ effective red time for movement m in cycle j , sec

$t_{u,m}^j =$ portion of the green interval when movement m can discharge freely without impedance, sec

$C =$ cycle length, sec

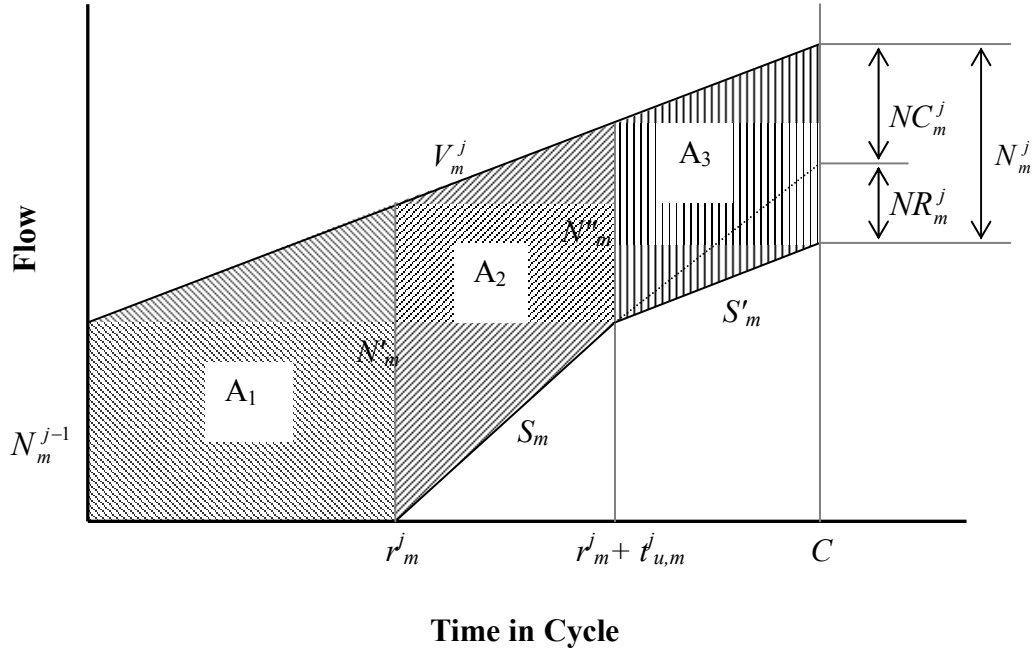


Figure 4. Delays and Queues for the Case with an Initial Queue and a Residual Queue.

NC_m^j can be determined based on Equation (1):

$$NC_m^j = \frac{(V_m^j - c_m^j)}{3600} C + N_m^{j-1} \quad (1)$$

where

c_m^j = the unimpeded capacity of movement m calculated based on Equation (2)

$$c_m^j = \frac{C - r_m^j}{C} s_m \quad (2)$$

NR_m^j in Figure 4 is a portion of the total residual queue that is contributed by the ramp queue spillback. Discussions on the calculation of NR_m^j are provided later in this chapter.

The total shaded areas of the queue polygon represent the total delays experienced by the vehicles arriving in the current cycle j and are calculated based on the following equations:

$$A_1 = (N_m^{j-1} + N_m') \frac{r_m^j}{2} = (2N_m^{j-1} + \frac{r_m^j}{3600} V_m^j) \frac{r_m^j}{2} \quad (3)$$

$$A_2 = (N_m' + N_m'') \frac{t_{u,m}^j}{2} = [2N_m^{j-1} + \frac{2r_m^j V_m^j - t_{u,m}^j (s_m - V_m^j)}{3600}] \frac{t_{u,m}^j}{2} \quad (4)$$

$$\begin{aligned} A_3 &= (N_m'' + N_m^j)(C - r_m^j - t_{u,m}^j) \\ &= [N_m^{j-1} + N_m^j + \frac{r_m^j V_m^j - t_{u,m}^j (s_m - V_m^j)}{3600}](C - r_m^j - t_{u,m}^j) \end{aligned} \quad (5)$$

The average delay for movement m during cycle j is:

$$d_m^j = \frac{A_1 + A_2 + A_3}{C \frac{V_m^j}{3600}} = 3600 \frac{A_1 + A_2 + A_3}{C V_m^j} \quad (6)$$

The queue length is represented by the vertical distance in the queue polygon in [Figure 4](#). The maximum queue is likely to occur at the start of the green interval.

Delays and queues for the internal movements are modeled similarly to PASSER III based on the delay-offset methodology ([16,17](#)). However, the effect of ramp queue spillback is specifically modeled in our model (e.g., ramp queue spillback to internal left-turn movement), which has not been addressed in PASSER III. The methodology consists of an analysis procedure on a second-by-second basis. The analysis takes into consideration the unique arrival/departure flow profiles, which are associated with the phasing and timing of the diamond interchange signal.

FREEWAY AND RAMP METERING

Modeling freeway and ramp metering operations is also based on the cumulative arrival/departure queue polygon method, but the analysis is carried out on a second-by-second basis. The following equations provide mathematical descriptions on the modeling process.

Equations (7) through (9) describe the freeway mainline flow arriving immediately upstream of the on-ramp at time interval t . The initial randomly generated demand, $F_r(t)$, is capped by a factor γ times the free-flow capacity, C_{Fr} , reflecting the maximum flow rate that can get to the point immediately upstream of the ramp merge. $F_r''(t)$ is the average flow at time step t during the ramp metering interval, n . $F_r''(t)$ is used to determine the ramp metering rate in Equation (11), so that the same ramp metering rate would result for a metering interval.

$$F_r'(t) = \begin{cases} \gamma C_{Fr}, & F_r(t) > C_{Fr}\gamma \\ \text{Min}[F_r(t) + \Delta F_r(t-1), \gamma C_{Fr}], & \text{Otherwise} \end{cases} \quad (7)$$

$$\Delta F_r(t) = \text{Max}(0, \Delta F_r(t-1) + F_r(t) - F_r'(t)) \quad (8)$$

$$F_r''(t) = \frac{1}{n} \sum_{i=\text{int}(\frac{t}{n})n+1}^{i+n-1} F_r'(i) \quad (9)$$

$$A_{Rr}(t) = \sum_{i=1}^t \frac{R_r(i)}{3600} \quad (10)$$

$$M_r(t) = \begin{cases} M_{r,\min}, & q_{Fr}[\text{int}(\frac{t}{n})n] \geq \frac{C_{Fr}(\eta-1)}{3600} \\ M_{r,\min}, & F_r''(t) + M_{r,\min} > C_{Fr} \\ \text{Min}[C_F - F_r''(t), M_{r,\max}], & \text{Otherwise} \end{cases} \quad (11)$$

$$q_{Rr}(t) = \text{Max}[0, q_{Rr}(t-1) + \frac{R_r(t) - M_r(t)}{3600}] \quad (12)$$

$$D_{Rr}(t) = A_{Rr}(t) - q_{Rr}(t) \quad (13)$$

$$O_{Rr}(t) = 3600[D_{Rr}(t) - D_{Rr}(t-1)] \quad (14)$$

$$C_{Fr}(t) = \left\{ \begin{array}{ll} F^{-1}(RND, C_{Qr}, \sigma_{Qr}), & 3600q_{Fr}(t-1) + F''(t) + O_{Rr}(t) > \eta C_{Fr} \\ F^{-1}(RND, C_{Fr}, \sigma_{Fr}), & \text{Otherwise} \end{array} \right\} \quad (15)$$

η is the breakdown factor (1.5 is assumed in the model) to reflect that freeway breakdown will occur once the bottleneck demand is 1.5 times or higher than the average free-flow capacity, C_{Fr} . Equation (15) determines the freeway mainline capacity at time t, which has the two-capacity nature with random variations, as given by the random variable generation function, F^{-1} .

$$A_{Fr}(t) = \sum_{i=1}^t \frac{[F_j''(i) + O_{Rr}(i)]}{3600} \quad (16)$$

$$q_{Fr}(t) = \text{Max}[0, q_{Fr}(t-1) + \frac{F_r''(t) + O_{Rr}(t) - C_{Fr}(t)}{3600}] \quad (17)$$

$$D_{Fr}(t) = A_{Fr}(t) - q_{Fr}(t) \quad (18)$$

$$O_{Fr}(t) = 3600[D_{Fr}(t) - D_{Fr}(t-1)] \quad (19)$$

$$TD_{Rr} = \sum_{i=1}^T \frac{q_{Rr}(i)}{3600} \quad (20)$$

$$TD_{Fr} = \sum_{i=1}^T \frac{q_{Fr}(i)}{3600} \quad (21)$$

Where

$F_r(t)$ = randomly generated freeway mainline demand for direction r and time interval t ,
vph

$F'_r(t)$ = capped freeway mainline arrival flow rate at the point of ramp merge location,
vph

$F''_r(t)$ = average mainline arrival flow rate during ramp metering interval, vph

$\Delta F_r(t)$ = mainline residual demand at time interval t , vph

$R_r(t)$ = traffic arrival rate at time interval t at ramp r , vph

$M_r(t)$ = ramp metering rate at time interval t at ramp r , vph

$A_{Rr}(t)$ = cumulative number of arrivals at time interval t and ramp r , veh

$D_{Rr}(t)$ = cumulative number of departures at time interval t and ramp r , veh

$O_{Rr}(t)$ = throughput at time interval t and ramp r , vph

$A_{Fr}(t)$ = cumulative number of arrivals at time interval t and freeway direction r , veh

$D_{Fr}(t)$ = cumulative number of departures at time interval t and freeway direction r , veh

$O_{Fr}(t)$ = mainline throughput at time interval t and freeway direction r , vph

$C_{Fr}(t)$ = freeway capacity at time interval t and direction r , vph

C_{Fr} = free-flow capacity of mainline direction r , vph

C_{Qr} = queue-discharge capacity of mainline direction r , vph

- σ_{F_r} = standard deviation of free-flow capacity for direction r , veh
- σ_{Q_r} = standard deviation of queue-discharge capacity for direction r , veh
- $M_{r,min}$ = minimum metering rate for ramp r , vph
- $M_{r,max}$ = maximum metering rate for ramp r , vph
- $q_{F_r}(t)$ = freeway mainline queue length at time interval t and direction r , veh
- $q_{R_r}(t)$ = queue length at time interval t and ramp r , veh
- TD_{F_r} = total freeway mainline delay for direction r , veh-hr
- TD_{R_r} = total delay for ramp r , veh-hr
- η = breakdown factor, 1.5
- γ = flow cap factor, 1.2

RAMP FLOW PROFILES

Modeling ramp metering and freeway operations requires adequate description of the ramp arrival flow profile, $R_r(t)$, which is uniquely determined based on the diamond phasing and timing. This section provides detailed descriptions on modeling the arrival flow profiles at the ramp meter. The project focuses on the two commonly used diamond phasing schemes: basic three-phase and TTI four-phase (6).

With the existence of an upstream signalized diamond interchange, vehicles arrive at the ramp meter with unique flow structures. Figure 5 and Figure 6 illustrate the arrival flow profiles at ramp 1 (R1) with basic three-phase and TTI four-phase phasing schemes, respectively. The profiles shown in both figures assume that the arterial right-turn movement (M6) and the U-turn movement (M14) are uncontrolled and would arrive at the ramp randomly. It is also assumed that platoons released from the diamond interchange do not disperse while traveling to the ramp-metering location.

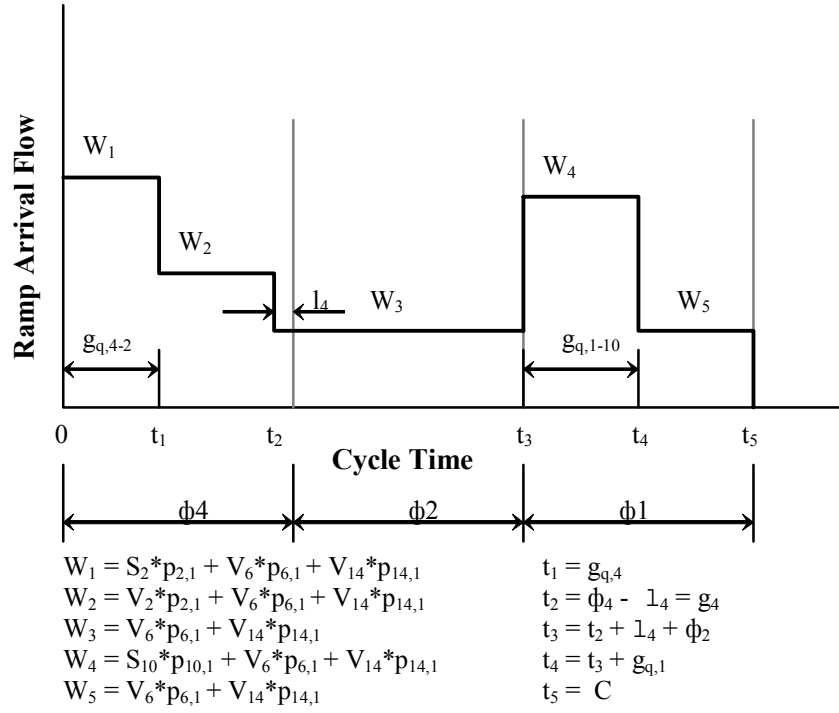
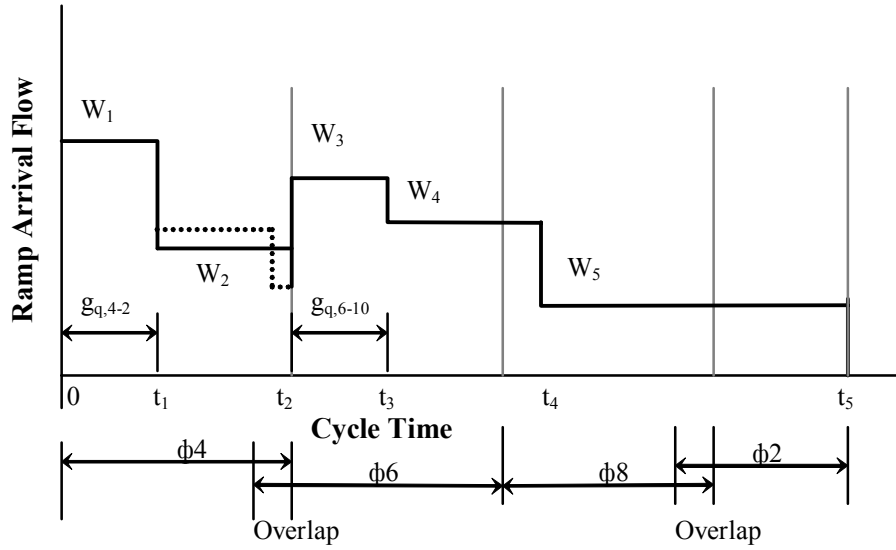


Figure 5. Ramp Arrival Flow Profile with Basic Three-Phase.



$$\begin{aligned}
 W_1 &= S_2 * p_{2,1} + V_6 * p_{6,1} + V_{14} * p_{14,1} & t_1 &= g_{q,4} \\
 W_2 &= V'_2 * p_{2,1} + V_6 * p_{6,1} + V_{14} * p_{14,1} & t_2 &= \phi_4 \\
 W_3 &= \mu * p_{10,1} + V_6 * p_{6,1} + V_{14} * p_{14,1} & t_3 &= t_2 + g_{q,6} \\
 W_4 &= V_{10} * p_{10,1} + V_6 * p_{6,1} + V_{14} * p_{14,1} & t_4 &= t_2 + \phi_6 \\
 W_5 &= V_6 * p_{6,1} + V_{14} * p_{14,1} & t_5 &= C
 \end{aligned}$$

** V'_2 is the average flow rate of the unsaturated green portion, including the lost time interval

Figure 6. Ramp Arrival Flow Profile with TTI Four-Phase.

The symbols shown in both figures are described below:

W_t = arrival flow rate during time period t_{i-1} and t_i , vph

$g_{q,\phi-m}$ = queue discharge portion of the green interval for signal phase ϕ and movement m , sec

l_ϕ = lost time for phase ϕ

Equations (22) through (24) show the calculations on $g_{q,\phi-m}$:

$$g_{q,4-2} = \begin{cases} \frac{3600\Delta N_2 + V_2 r_4}{S_2 - V_2}, & x_2 < 1 \\ g_4, & x_2 \geq 1 \end{cases} \quad (22)$$

$$g_{q,1-10} = \begin{cases} g_1 x_{10} = \frac{3600 \Delta N_{10} + V_{10} C}{S_{10}}, & q_{10} < Q_{10} \\ \frac{Q_{10}}{S_{10}} 3600, & q_{10} \geq Q_{10} \end{cases} \quad (23)$$

$$g_{q,6-10} = \begin{cases} \frac{3600 (\Delta N_{10} + \Delta N_{11}) + (V_{10} + V_{11}) r_6}{S_{10+11} - (V_{10} + V_{11})}, & x_{10+11} < 1 \\ g_6, & x_{10+11} \geq 1 \end{cases} \quad (24)$$

where

ΔN_m = the residual queue from the previous cycle for movement m , veh

q_{10} = the maximum number of vehicles that would occur for movement 10 (residual plus arrival) during the current cycle, veh

Q_{10} = internal queue storage space for movement 10, veh

x_m = v/c ratio for movement m

S_{10+11}, x_{10+11} are for the external approach on the arterial related to M10 and M11.

The individual flows in the flow profiles should satisfy Equation (25):

$$R_r = \frac{1}{C} \sum_{t=1}^5 W_t (t_t - t_{t-1}) \quad (25)$$

When the arterial right-turn movement is not controlled by the signal, such as the case of a channelized right-turn movement, both basic three-phase and TTI four-phase can be represented by five different flow regions in a profile (see Figure 5 and Figure 6). When the right-turn movement is controlled by the signal, more than five flow regions are necessary to represent the flow profiles. Figure 7 and Figure 8 illustrate some ramp arrival flow profiles for a duration of six cycles with stochastic traffic demands in each cycle. Figure 7 shows the case without arterial right-turn control, and Figure 8 shows the case with arterial right-turn control.

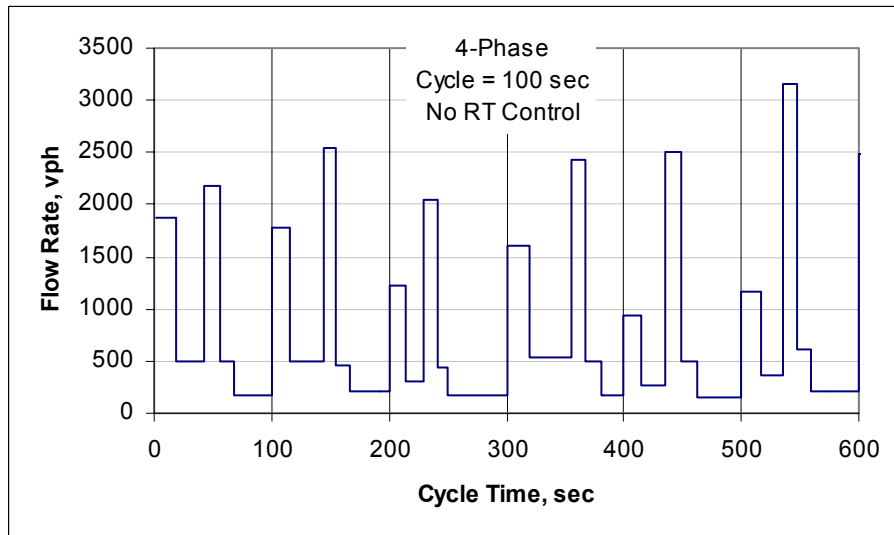
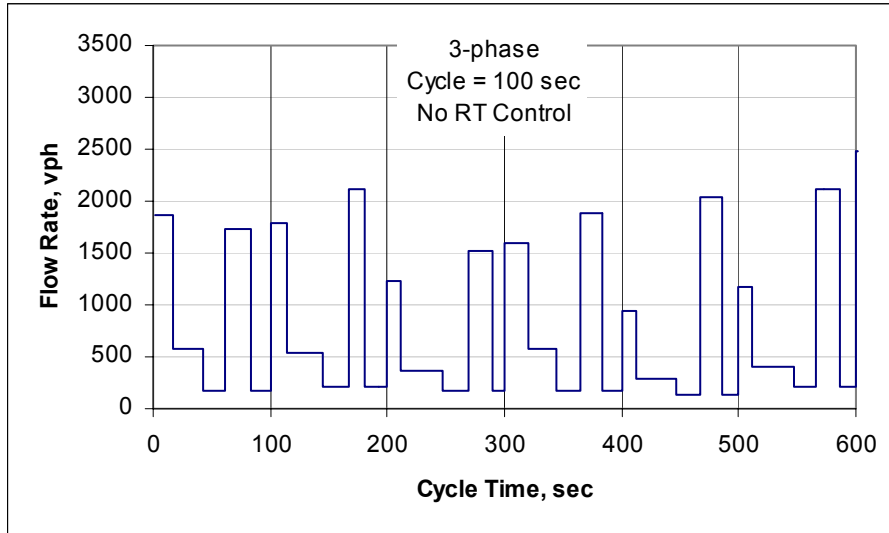


Figure 7. Ramp Arrival Flow Profile without Arterial Right-Turn Control.

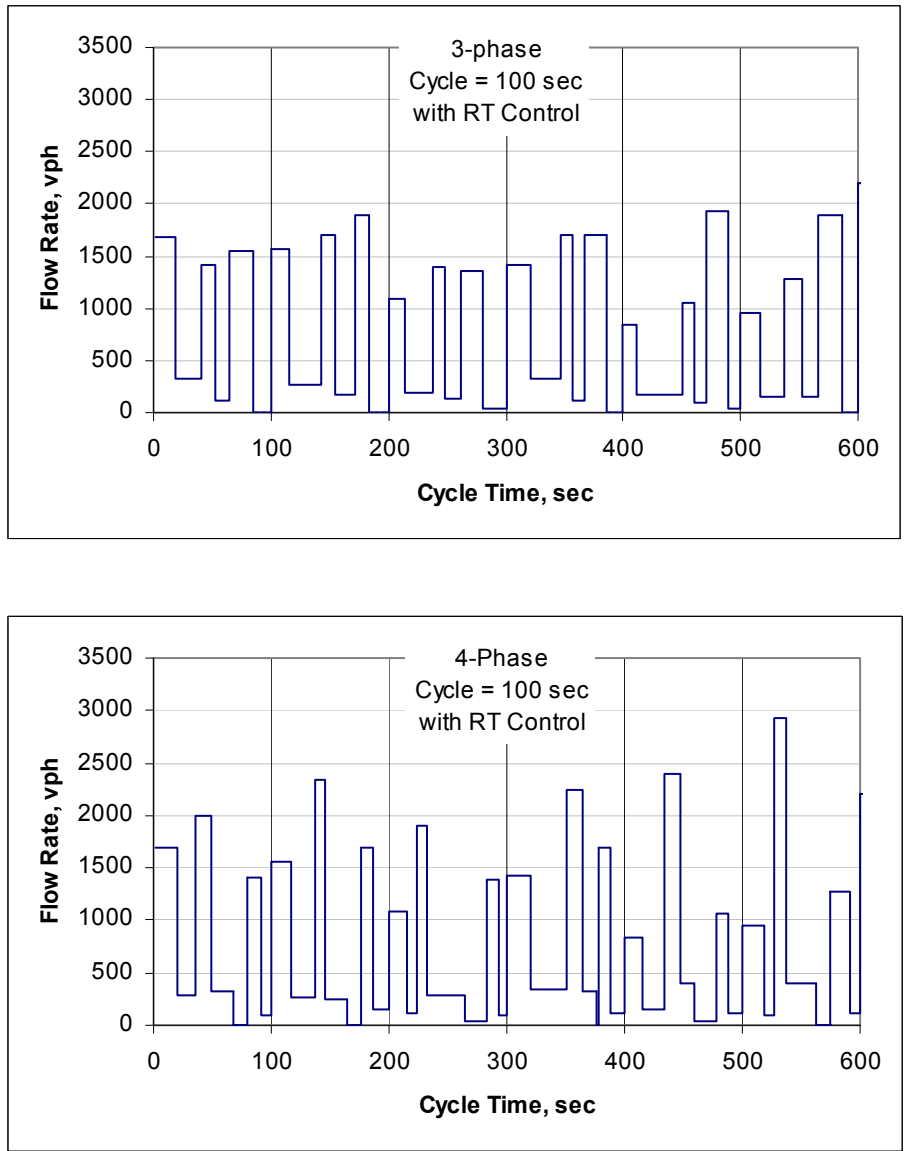


Figure 8. Ramp Arrival Flow Profile with Arterial Right-Turn Control.

RAMP QUEUE SPILLBACK

When sufficient storage between the ramp meter and the diamond signal exists to store the vehicle queues, the diamond interchange signal can discharge the vehicles according to the traffic flow profiles depicted in [Figure 7](#) and [Figure 8](#) without incurring any impedance. However, when the storage space is filled with queued vehicles due to either limited spacing or simply over-saturation, the ramp queues would impede traffic flows discharged from the diamond signals, resulting in reduced capacity and increased delay for the affected traffic movements. Previous studies on modeling queue spillback at signalized intersections are

generally based on two approaches. One approach is to reduce the saturation flow rate (1,18). For example, Messer and Bonneson (1) proposed using a simple factor to adjust the saturation flow rate based on the queue length of the downstream link. The other approach is to reduce the effective green time (19), considering the queue block effect as equivalent to the loss of green time. Both approaches would reach similar conclusions.

Because modeling of ramp operations and ramp queues occur on a second-by-second basis, it is possible to have detailed modeling on the impact of spillback on diamond interchange operations. The following discussions describe the modeling methodology, which uses the approach of adjusting discharging flows.

The basic principle to model vehicle discharge with potential queue spillback is based on the fact that the vehicle discharging rate is governed by the minimum of three flows, as shown in Equation (26):

$$Q_R(t) = \text{Min}[Q_W(t), Q_M(t), Q_B(t)] \quad (26)$$

where

- $Q_R(t)$ = discharging flow rate from diamond signal at time step t , vph
- $Q_W(t)$ = unimpeded demand flow at time slice t , vph
- $Q_M(t)$ = the maximum possible discharging flow rate at time step t , vph
- $Q_B(t)$ = discharging flow rate that would result in queue spillback at time step t , vph

All these flows represent the traffic that would arrive at the ramp meter. Because not all the traffic discharged from the diamond interchange would arrive at the ramp meter, the actual discharging flow rate at the diamond signal is normally higher (e.g., traffic going to the frontage road) than what is shown in Equation (26); therefore, the flow rate should be adjusted accordingly based on the proportion of each movement that goes to the ramp meter. $Q_W(t)$ is the demand flow when there is no queue spillback to the diamond signal, so that vehicles can discharge from the signal unimpeded according to traffic flow profiles described in Figure 5 through Figure 8. $Q_M(t)$ is the maximum possible flow rate that can be discharged from the diamond signal and would arrive at the ramp meter. $Q_B(t)$ is the equivalent portion of the ramp

arrival flow when the diamond signal is discharging at the saturation flow rate during a particular phase. $Q_M(t)$ varies depending on the diamond phasing scheme, and its values are determined based on [Table 3](#).

Table 3. Determination of $Q_M(t)$ Values.

Phasing Code	Phase Sequence		Time and Entering Flows
Basic Three-Phase	Ramp 1	Φ4	t_{1-2}, W_{1-1}
		Φ2	t_{1-3}, W_{1-3}
		Φ1	t_{1-5}, W_{1-4}
	Ramp 2	Φ8	t_{2-2}, W_{2-1}
		Φ6	t_{2-3}, W_{2-3}
		Φ5	t_{2-5}, W_{2-4}
TTI Four-Phase	Ramp 1	Φ4	t_{1-2}, W_{1-1}
		Φ6	$t_{1-5} - \Phi 2, W_{1-3}$
		Φ8,2	t_{1-5}, W_{1-5}
	Ramp 2	Φ8	t_{2-2}, W_{2-1}
		Φ2	$t_{2-5} - \Phi 2, W_{2-3}$
		Φ4,6	t_{2-5}, W_{2-5}

Note: Refer to Figures 5 and 6 for referencing t_{1-i} and W_{1-i} values for Ramp 1

Using ramp 1 as an example, when basic three-phase is used, the phasing sequence at the left-side signal is $\phi 4$, $\phi 2$, and $\phi 1$. W_{1-1} is the flow rate at the ramp meter that is equivalent to when M2 discharges at its saturation flow rate. This flow can last up to t_{1-2} , the end of green of $\phi 4$, as long as there is sufficient demand for M2. Similarly, W_{1-4} is the ramp arrival flow rate that is equivalent to when M1 discharges at its saturation flow rate, and it can last up to the point at t_{1-5} , the end of the green of $\phi 1$, as long as there is a sufficient demand.

$Q_B(t)$ is the flow that would result in queue spillback to block the diamond signal, which can be determined based on [Equation \(27\)](#):

$$Q_B(t) = 3600[Q_r - q_{Rr}(t-1)] + M_r(t) \quad (27)$$

where

Q_r = storage space of ramp r between the ramp metering signal and the diamond interchange signal, veh

With $Q_R(t)$ determined based on Equation (26) and the ramp demand flow profile such as shown in Figure 5 or Figure 6, any vehicles that cannot be discharged freely are considered as part of the residual queues for the current cycle, as denoted by NR_m^j earlier. The following describes the methodology to estimate NR_m^j , using ramp 1 and with basic three-phase as an example.

During the current cycle j , the traffic demands feeding ramp 1 from the four feeding movements (M2, M6, M10, M14) are given by Equations (28) through (31):

$$V_{2-1}^j = p_{2,1}^j V_2^j \quad (28)$$

$$V_{10-1}^j = p_{10,1}^j V_{10}^j \quad (29)$$

$$V_{6-1}^j = p_{6,1}^j V_6^j \quad (30)$$

$$V_{14-1}^j = p_{14,1}^j V_{14}^j \quad (31)$$

The phasing sequence at the diamond signal is ϕ_4, ϕ_2, ϕ_1 . The residual ramp queues denoted as $N_{\phi_4}^j, N_{\phi_2}^j, N_{\phi_1}^j$ during each phase are recorded during the process of modeling ramp operations. Please note that M6 and M14 are uncontrolled and arrive at the ramp uniformly during the cycle. Therefore, $N_{\phi_4}^j$ is contributed by three movements: M2, M6, and M14; $N_{\phi_2}^j$ is contributed by two movements: M6 and M14; and $N_{\phi_1}^j$ is contributed by three movements: M10, M6, and M14.

The residual queue for M2, NR_2^j , is determined by:

$$NR_2^j = \frac{\left(\frac{V_{2-1}^j}{V_{2-1}^j + V_{6-1}^j + V_{14-1}^j} N_{\phi 4}^j \right)}{p_{2,1}^j} \quad (32)$$

The residual queue for M10, NR_{10}^j is determined by:

$$NR_{10}^j = \frac{\left(\frac{V_{10-1}^j}{V_{6-1}^j + V_{10-1}^j + V_{14-1}^j} N_{\phi 1}^j \right)}{p_{10,1}^j} \quad (33)$$

The residual queue for M6, NR_6^j , is determined by:

$$NR_6^j = \frac{\left(\frac{V_{6-1}^j}{V_{2-1}^j + V_{6-1}^j + V_{14-1}^j} N_{\phi 4}^j \right)}{p_{6,1}^j} + \frac{\left(\frac{V_{6-1}^j}{V_{6-1}^j + V_{14-1}^j} N_{\phi 2}^j \right)}{p_{6,1}^j} + \frac{\left(\frac{V_{6-1}^j}{V_{6-1}^j + V_{10-1}^j + V_{14-1}^j} N_{\phi 1}^j \right)}{p_{6,1}^j} \quad (34)$$

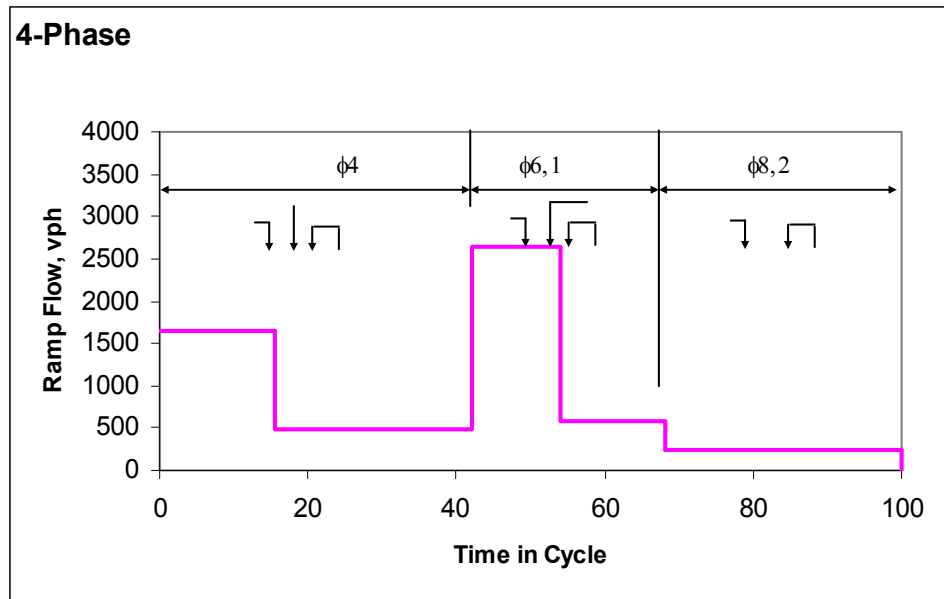
The residual queue for M14, NR_{14}^j , is determined by:

$$NR_{14}^j = \frac{\left(\frac{V_{14-1}^j}{V_{2-1}^j + V_{6-1}^j + V_{14-1}^j} N_{\phi 4}^j \right)}{p_{14,1}^j} + \frac{\left(\frac{V_{14-1}^j}{V_{6-1}^j + V_{14-1}^j} N_{\phi 2}^j \right)}{p_{14,1}^j} + \frac{\left(\frac{V_{14-1}^j}{V_{6-1}^j + V_{10-1}^j + V_{14-1}^j} N_{\phi 1}^j \right)}{p_{14,1}^j} \quad (35)$$

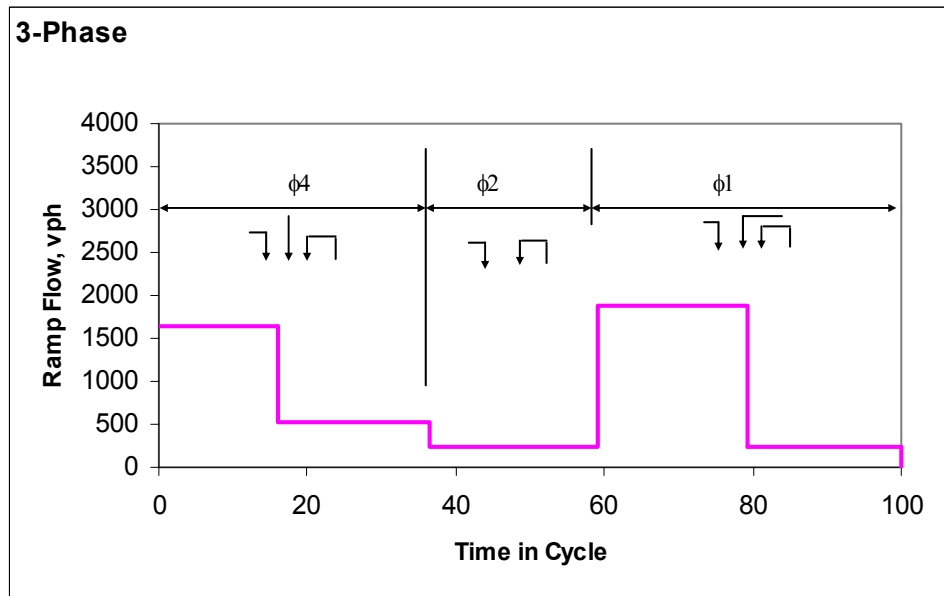
The methodology described above for estimating residual queues due to ramp spillback has an underlying assumption that once the ramp queue blocks the diamond signal, the entire feeding movement will be blocked, including traffic heading for the frontage road. However, spillback has no impact on those traffic movements that do not feed the on-ramps, such as the right-turn movement on the frontage approach (M3), the arterial left-turn movement (M1), and the arterial through movement (M4, M5).

While at the present stage it is not known whether basic three-phase and TTI four-phase would result in significant differences in ramp delays and queues, the two phasing schemes do present different impacts on diamond interchange operations under queue spillback conditions. The two phasing schemes result in the feeding traffic movements being serviced in different sequences, as illustrated in [Figure 9](#) for ramp 1. It can be seen that the frontage road movement (M2) is serviced following the arterial left-turn movement (M10) with basic three-phase, while

M2 is serviced prior to M10 with TTI four-phase. Because M2 and M10 are the major ramp feeding movements with higher flow rates and platoons, M2 is more likely to face queue spillback with basic three-phase than with TTI four-phase. This operational feature is further verified later when model validation is discussed.



a) Basic Three-Phase



b) TTI Four-Phase

Figure 9. Feeding Movement Service Sequences and Ramp Flow Profiles.

CHAPTER 3: DEVELOPMENT OF THE MODELING SOFTWARE

A computer software named DRIVE (Diamond Interchange/Ramp Metering Integration Via Evaluation) was developed, which employed the modeling methodologies documented in [Chapter 2](#). DRIVE can be used to perform system analysis and evaluation of operational strategies for IDIRMS.

[Figure 10](#) depicts the main modules and functions of DRIVE. DRIVE is designed to perform modeling and analysis for the IDIRMS, which includes a diamond interchange, freeway on-ramps with ramp metering, and freeway mainline operations.

DRIVE is classified as a mesoscopic simulation model and was developed using VisualBasic in Excel. VisualBasic in Excel takes advantage of both VisualBasic's programming features and Excel's spreadsheet functions. Excel also serves as the simple user interface for processing input and output information. Unlike deterministic models such as PASSER III, TRANSYT-7F, and HCM, DRIVE is designed to perform analysis over multiple cycles (currently designed for an analysis period of 100 cycles), considering stochastic traffic demand variations. Therefore, DRIVE presents significant enhancements over deterministic models by providing more realistic traffic demand modeling and taking into consideration the impact of over-saturation on system performances. [Table 4](#) summarizes the major input and output information for DRIVE. The modeling workflow chart is shown in [Figure 11](#).

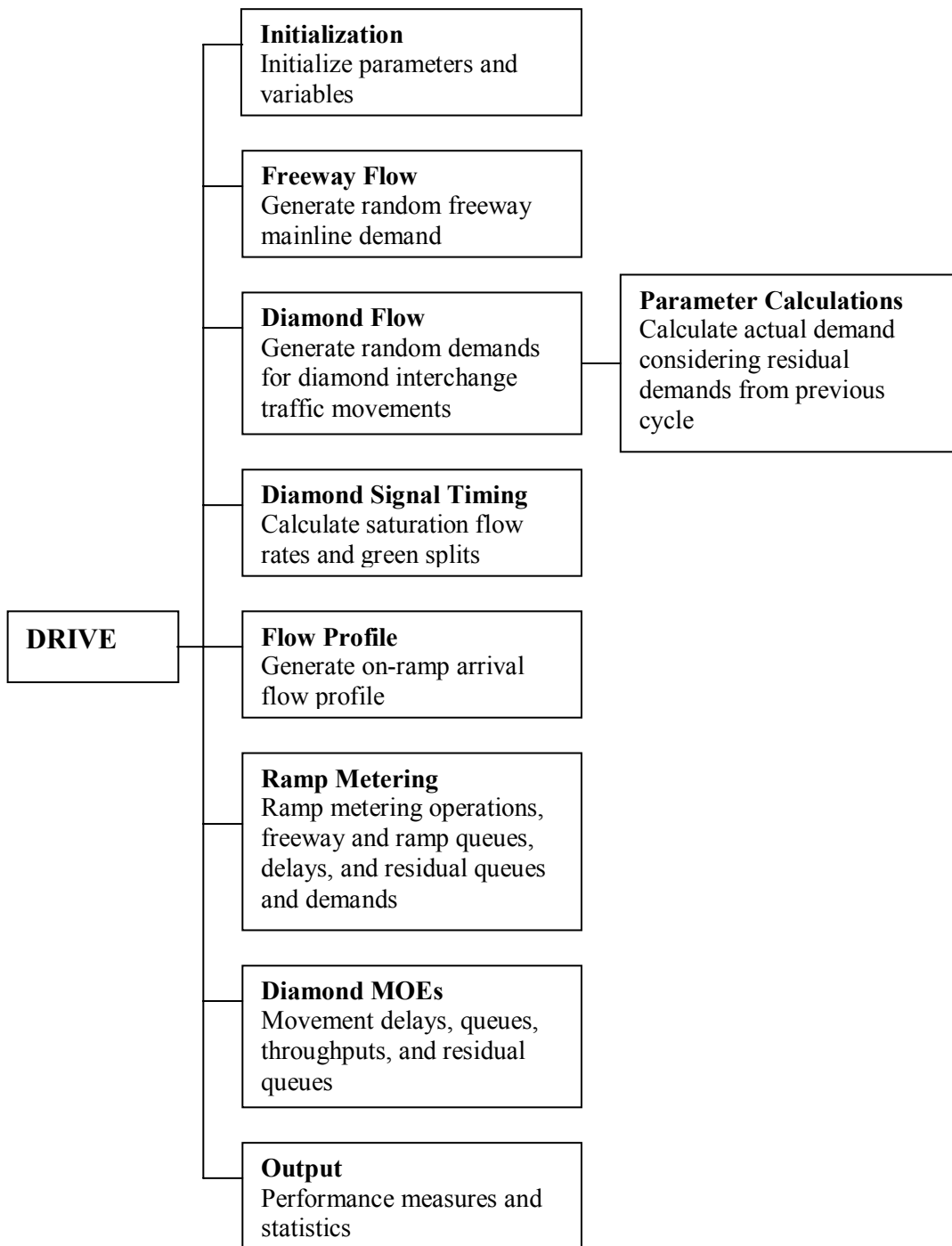


Figure 10. DRIVE Modules and Functions.

Table 4. Input/Output Information for DRIVE Software.

Input/Output	Information and Parameters	
<i>Input</i>	Traffic Demand	OD matrix
	Geometry and Traffic Flow Parameters	<i>Diamond</i> : Internal storage space, lane configuration, diamond spacing, arterial speed <i>On-Ramp/Frontage Road</i> : Ramp queue storage, queue block storage, ramp metering rates, queue flush rate <i>Freeway</i> : Average free-flow capacity and its standard deviation, average queue-discharge capacity and its standard deviation, freeway breakdown factor
	Signal Timing	Cycle length, phasing type, phase lost time, metering rate interval
	Other	Right-turn control type, meter flush mode, signal operations mode, fixed or stochastic demand option, random number seed
<i>Output</i>	Diamond Interchange	Total delay, average delay, maximum queue, 95-percentile queue, average queue, residual queue
	Ramp	Throughput, maximum queue, 95-percentile queue, average queue, total delay, average delay, number of queue flush, time duration of queue flush, ramp meter attainability, % time ramp queue spillback, % time queue blockage to diamond signal
	Freeway	Throughput, total delay
	Summary Profiles	For every time step: Queue length on both freeway mainline and on-ramp, ramp metering rates, freeway mainline and on-ramp throughputs

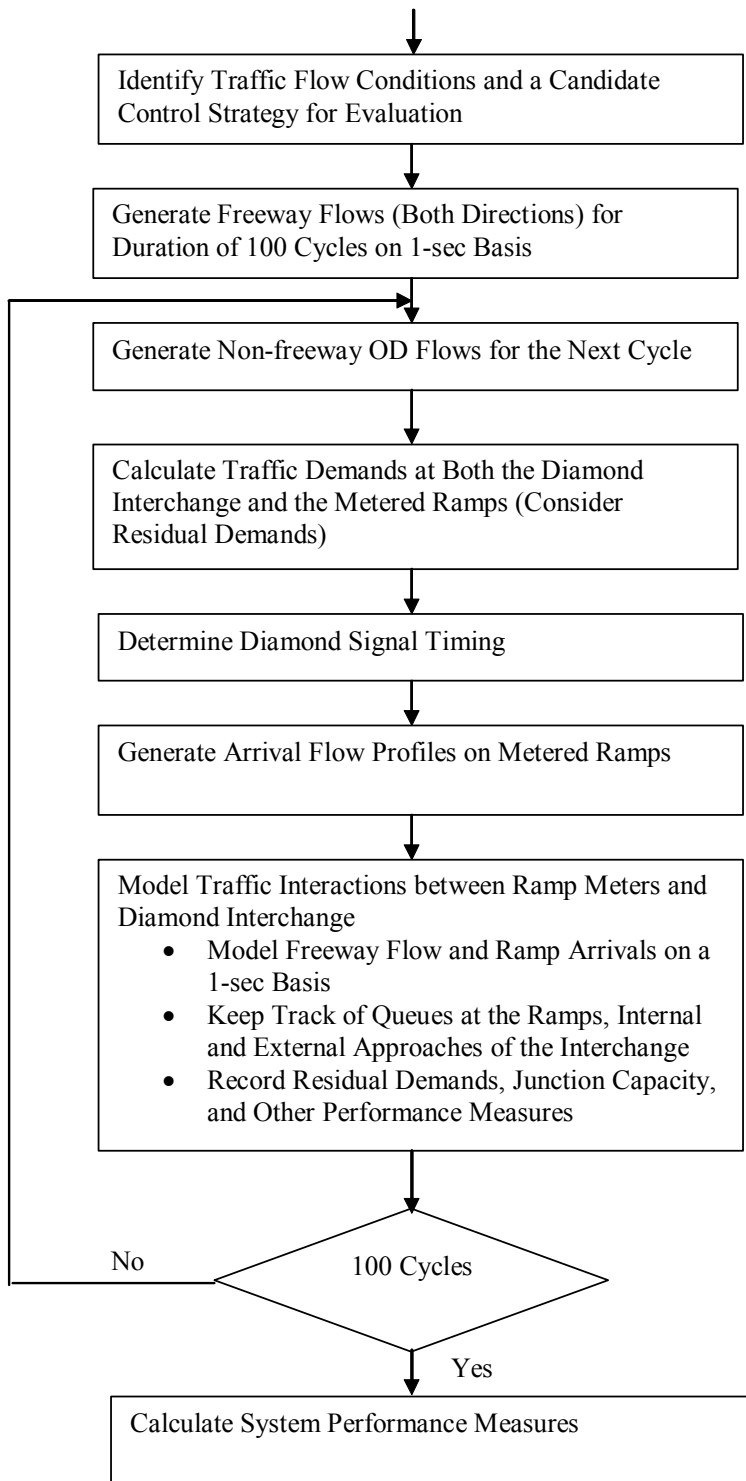


Figure 11. DRIVE Workflow Chart.

CHAPTER 4: MODEL VALIDATION

Validation on the DRIVE model was conducted using VISSIM (10), a microscopic traffic simulation model. Traffic demands and geometric data from a real interchange location were used to code the VISSIM model. Validation of DRIVE was primarily based on comparing the delays between DRIVE and VISSIM for the major traffic flows in the system. This chapter presents the model validation process and results.

SITE DESCRIPTION

Traffic volumes and network geometry data were collected at the Mayfield Road/SH 360 interchange located in Arlington, Texas, along the SH 360 corridor. A ramp-metering system consisting of five diamond interchanges was in operation for the northbound direction during the a.m. peak period between 6:00 a.m. and 9:00 a.m. The Mayfield/SH 360 diamond interchange signal was operating with basic three-phase, but was not in coordination with other signals in the Mayfield Road arterial. The ramp metering was operating at a fixed rate of 900 vph, the maximum metering rate for a typical single-lane ramp meter. Excessive queue detectors were installed to trigger ramp meter queue flush, a policy adopted in Texas as well in many other states (4) for ramp metering operations. Therefore, ramp queues never spilled back to the diamond interchange in the field. The average ramp demand was 860 vph during the peak hour. Due to traffic demand fluctuation and its stochastic nature, ramp traffic demands exceeded the 900 vph metering capacity during certain cycles of the peak period, and queue flushes were occasionally observed in the field. The diamond interchange itself has sufficient capacity to handle the traffic demands at the interchange.

VISSIM MODEL DEVELOPMENT

The primary reason for using VISSIM traffic simulation for model validation was to duplicate the conditions of ramp queue spillback, which could not be obtained from the field. The validation process involved first calibrating the VISSIM model and DRIVE for under-

saturated conditions based on PASSER III, where queue spillback did not occur. The model was then modified to reflect the conditions with ramp queue spillback. Validation of DRIVE was carried out by comparing the delays between DRIVE and VISSIM.

The VISSIM model was established based on the traffic flow, geometry, and signal timing information collected in the field at the Mayfield/SH 360 interchange, but with the following modifications:

- Since no truck traffic information was available, all the vehicles were coded as passenger cars.
- For consistency purposes, fixed signal timing was used for the diamond interchange with the green splits determined based on the methodology of equal volume-to-capacity ratio, which was originally developed by Webster (20) and used in PASSER III.
- Ramp metering was coded for both directions for the purpose of obtaining additional data points for model validation, even though the southbound direction does not currently have a ramp meter installed.

Once the VISSIM model was established, a basic calibration process was conducted to achieve the following objectives:

- The traffic volumes obtained from simulation should be checked to ensure correct coding on the traffic demands.
- The maximum ramp metering throughput should match its designed metering capacity of 900 vph.

It is worth mentioning about the second point that in VISSIM, there is no special logic designed for ramp metering. Drivers react to a ramp meter in a similar manner to a traffic signal. To achieve the desired metering rate, a solution is to code the ramp meter using VISSIM's VAP function (21). One critical element is to have a demand detector coded at the metering signal, similarly to the demand detector used in the field. The ramp-metering signal would remain red unless there is a call at the demand detector, which is consistent with the current traffic-responsive operations in Texas.

MODEL CALIBRATION AND VALIDATION RESULTS

Average delay was selected as the primary *measure of effectiveness* (MOE) for comparison between DRIVE and VISSIM. Queue length would have been another candidate MOE; however, the queue length in each model is measured differently. VISSIM reports the backup queues in distance measured from a specified location, while DRIVE reports the queues in terms of the number of vehicles. While the backup queue could be converted into the number of vehicles, the average occupancy space by a vehicle dynamically changes in VISSIM depending on the ramp metering rate. On the other hand, backup queue includes the shockwave effect and would be in general at higher values than what is calculated in DRIVE.

The average delay was selected as the primary MOE for comparison because delays in both models are measured in a more consistent manner. However, it should be noted that the arrival/departure method used in DRIVE does not include the delays associated with deceleration and acceleration. A minimum delay would always occur in VISSIM due to the existence of ramp metering. To make it consistent between the delays calculated in both models, the minimum delay due to ramp metering was estimated in VISSIM by giving a very low ramp demand. This minimum delay, found to be about 7.0 seconds, was added in DRIVE to the delays based on the queue polygon method.

Considered as part of the model calibration process, the under-saturated conditions, i.e., no existence of ramp queue spillback, were analyzed using PASSER III, VISSIM, and DRIVE. The two cases analyzed included: a) ramp meter with queue flush, the current field operations; and b) ramp meter without queue flush. [Figure 12](#) and [Figure 13](#) are the results when the queue flush option was used, which was consistent with the existing ramp-metering policy. [Figure 12](#) compares the delays for the traffic movements at the diamond interchange and the on-ramps, while [Figure 13](#) compares the delays for freeway mainlines. All the results were based on the average of five runs for both VISSIM and DRIVE with identical simulation time (100 cycles or 10,000 seconds for this case). For diamond interchange movements, the results from PASSER III are also shown. With the queue flush policy, the diamond interchange and the on-ramps operate under capacity. As can be seen, both VISSIM and DRIVE produced delays that matched well with the results from PASSER III. As expected, the variations from each run are low for under-saturated situations. As for the freeway mainline, DRIVE produced significantly higher delays for the peak direction (northbound) with very high variations. The off-peak direction remains

under capacity for the entire analysis period; therefore, DRIVE reported zero delays, although VISSIM reported a 1.5 sec/veh delay. The higher delays and variations on the freeway mainline as reported by DRIVE were due to the modeling process of freeway breakdown. It was noted that VISSIM seemed to recover from breakdown much faster than what has been observed in field operations; therefore, further research is necessary to validate both models on modeling the freeway breakdown phenomenon.

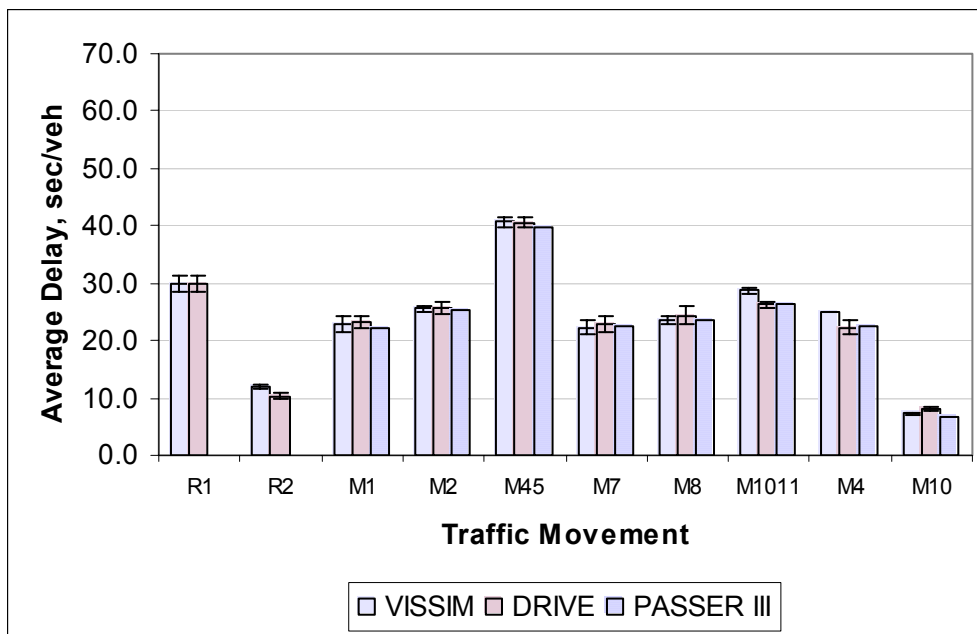


Figure 12. Ramp and Diamond Interchange Delays with Basic Three-Phase and Queue Flush (Average of Five Runs)

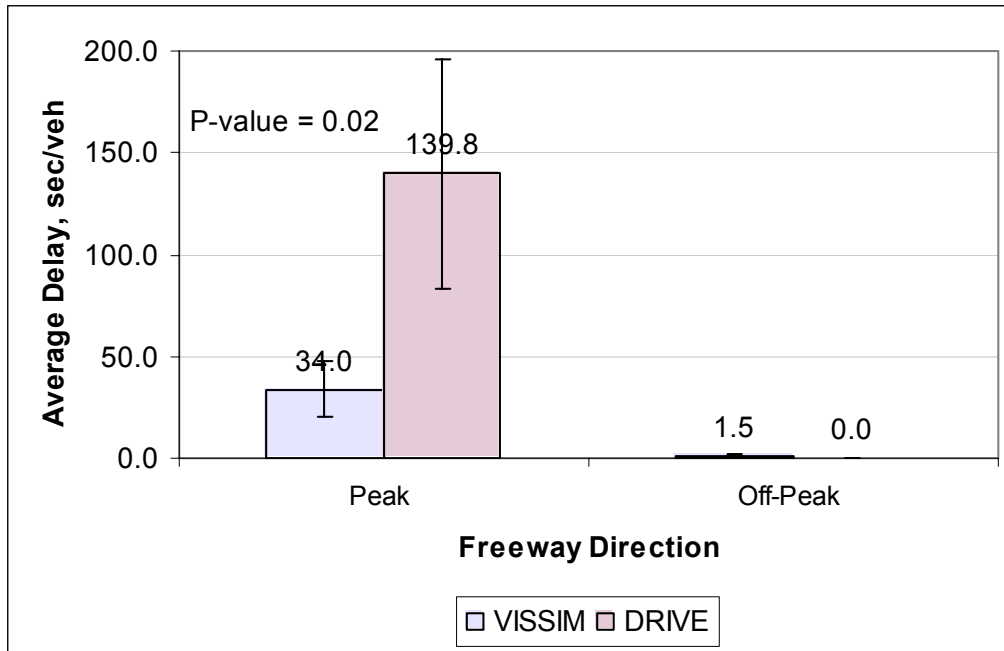


Figure 13. Freeway Mainline Delay with Queue Flush.

Figure 14 and Figure 15 show the results when queue flush was not used. As can be seen, delays as well as the variations in the peak direction on-ramp (R1) significantly increased, while the delays for the traffic movements at the diamond interchange remained relatively unchanged. This indicates that R1 experienced short-term overload (demand > capacity), but the queue never reached the diamond interchange signal to impact the traffic movements at the interchange. As for the delays on the mainline, DRIVE produced statistically identical delay results to that from VISSIM (P-value is 0.76), although the variation was still high due to the nature of freeway breakdown and its modeling complexity, as discussed earlier.

Figure 16 and Figure 17 illustrate the cases when the ramp was over-saturated and with queue spillback. The ramp demand was increased to approximately 940 vph at ramp 1, and to about 540 vph at ramp 2. Ramp-metering rates were maintained at 900 vph. Ramp 1 therefore was at over-saturation. As can be seen, the delays significantly increased for some traffic movements, especially for those feeding ramp 1. There were basically no impacts on those movements feeding ramp 2, since ramp 2 was still under-saturated, except for some minor increases in delay due to increased demand. Delays for R1 and M10 matched well between VISSIM and DRIVE and were capped at a certain level because there are limited queue storage spaces for these movements. Any queues exceeding the storage spaces were accumulated outside

the diamond interchange. It could also be seen that DRIVE reported no change in delay for M1, since an assumption was made in DRIVE that M1 is not affected by queue spillback. In VISSIM, however, M1 was eventually blocked by the queues on the frontage road approach; therefore showing an increase in delay. Similarly, DRIVE does not model the blockage to M11, while VISSIM showed occasional blockage to M11 when a M10 vehicle stayed in the right lane and blocked some M11 vehicles. Both VISSIM and DRIVE showed significant variations in the results for those over-saturated movements, which is the nature of traffic flow during over-saturation. Figure 16 and Figure 17 also illustrate that basic three-phase favors the arterial left-turn movement (M10), as indicated by the larger increase on delay for M2 with basic three-phase versus with TTI four-phase, which confirms an earlier assumption.

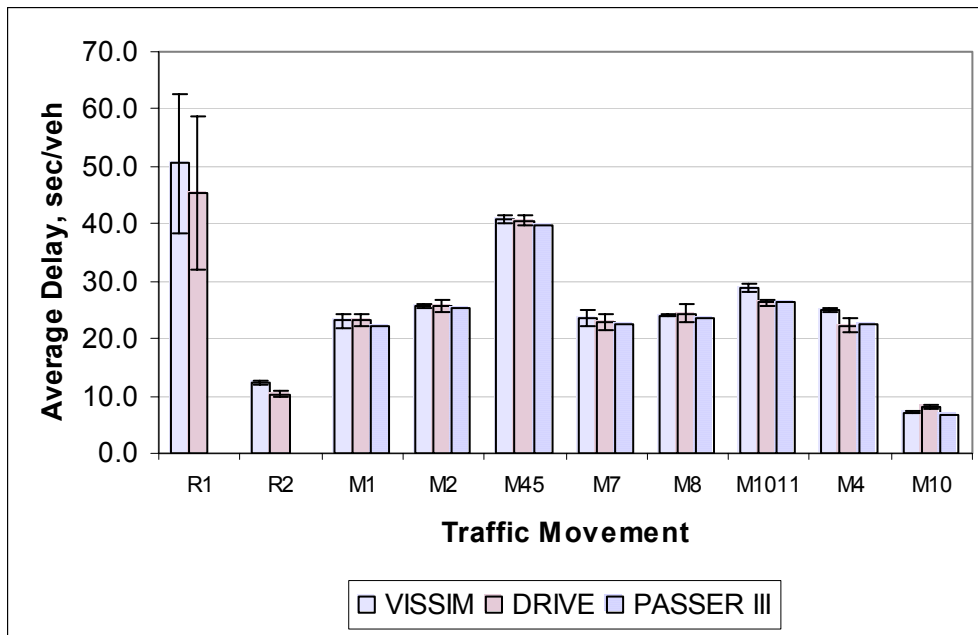


Figure 14. Ramp and Diamond Interchang Delays with Basic Three-Phase without Queue Flush (Average of Five Runs).

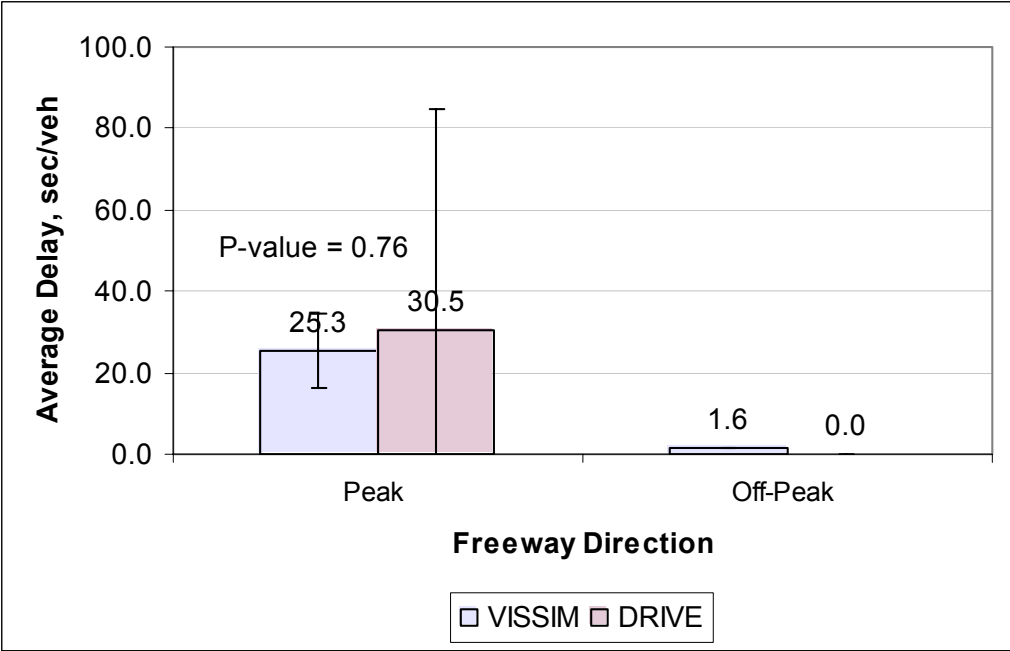


Figure 15. Freeway Mainline Delay without Queue Flush.

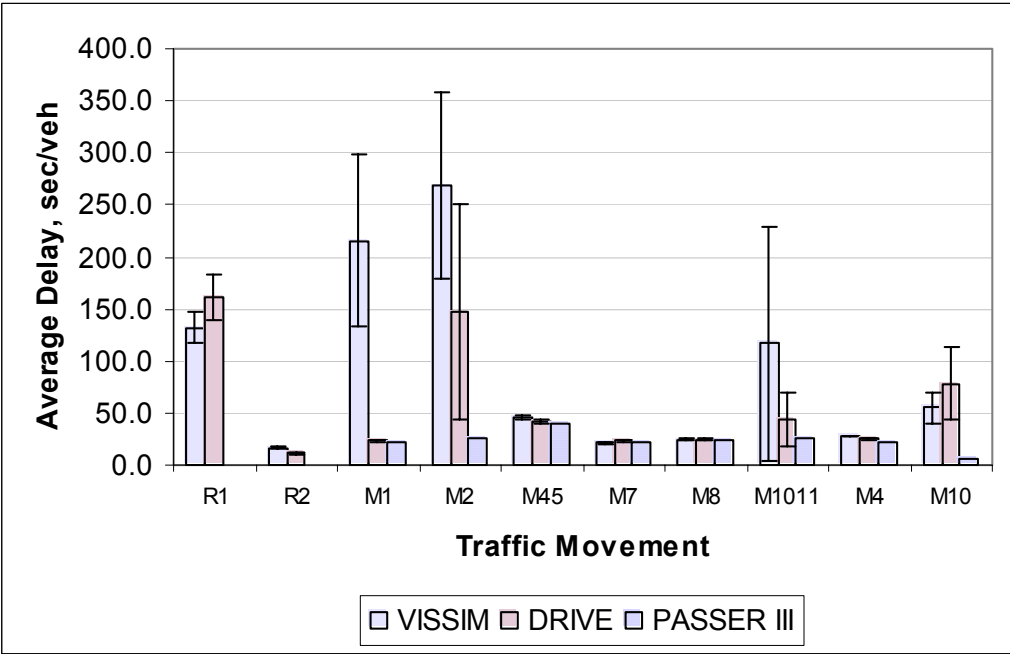


Figure 16. Over-saturated Case with Queue Spillback: Basic Three-Phase.

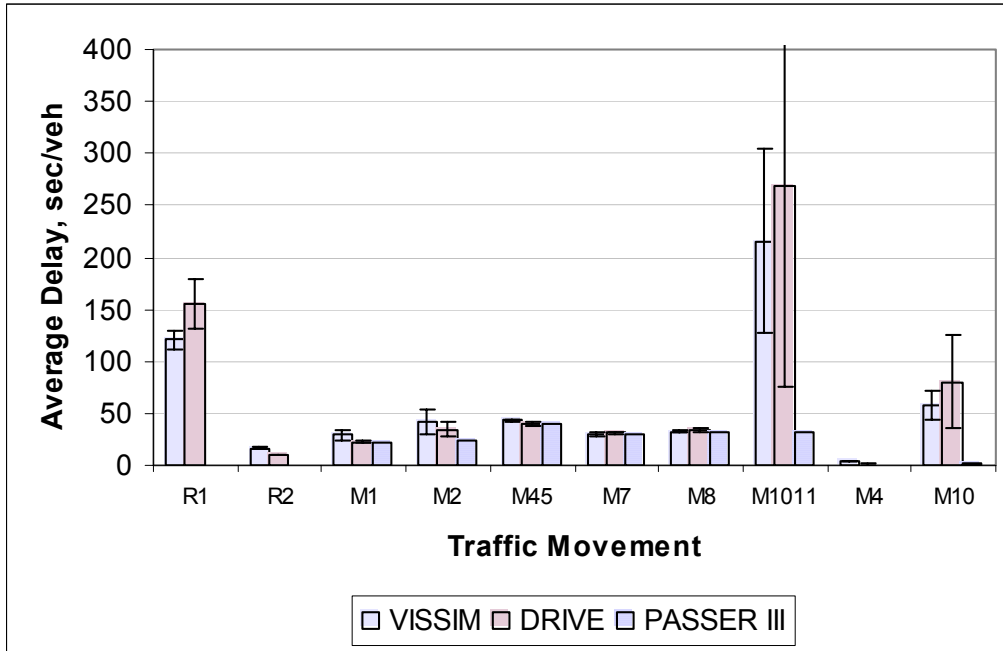


Figure 17. Over-saturated Case with Queue Spillback: TTI Four-Phase.

CHAPTER 5: ANALYSES OF SYSTEM OPERATIONAL CHARACTERISTICS

The goal of this research project was to develop strategies for integrating the operations of diamond interchanges and ramp metering operations; however, before we could develop operational strategies for integrating the operations of the diamond interchange and ramp, we first needed to explore the operational relationship between the signal timing used at the diamond interchange and the traffic patterns at the ramp. Using the DRIVE software, we examined the following questions related to diamond interchange and ramp operations:

- What effect does the type of phasing used at the diamond interchange have on the ramp queues?
- What effect does the type of phasing used at the diamond interchange have on the traffic movements feeding the ramps when ramp queue spillback occurs?
- What effect does the cycle length used at the diamond interchange have on the traffic queues at the ramp?
- What effect do the ramp metering operations (e.g., fixed metering vs. responsive metering, queue flush vs. no queue flush) have on freeway operations?

EFFECTS OF DIAMOND SIGNAL PHASING ON RAMP QUEUES

One potential strategy that could be used to achieve integrated operation would be selecting an appropriate type of phasing at the diamond (i.e., from a four-phase operation to a three-phase operation or vice versa) when the queue lengths at the ramp become too long. Therefore, one issue we wanted to examine was what effect the phasing patterns used at the diamond interchange have on queue growth at the ramp.

[Figure 18](#) compares the number of vehicles queued at the ramp resulting from basic three-phase and TTI four-phase while the ramp is operating during under-saturated conditions, i.e., no queue spillback to the diamond interchange. A t-statistical test indicates that both phasing schemes result in identical ramp queue length as indicated by the P-value of 0.36. This results suggests that the diamond phasing scheme has little impact on the ramp queues during under-saturated conditions.

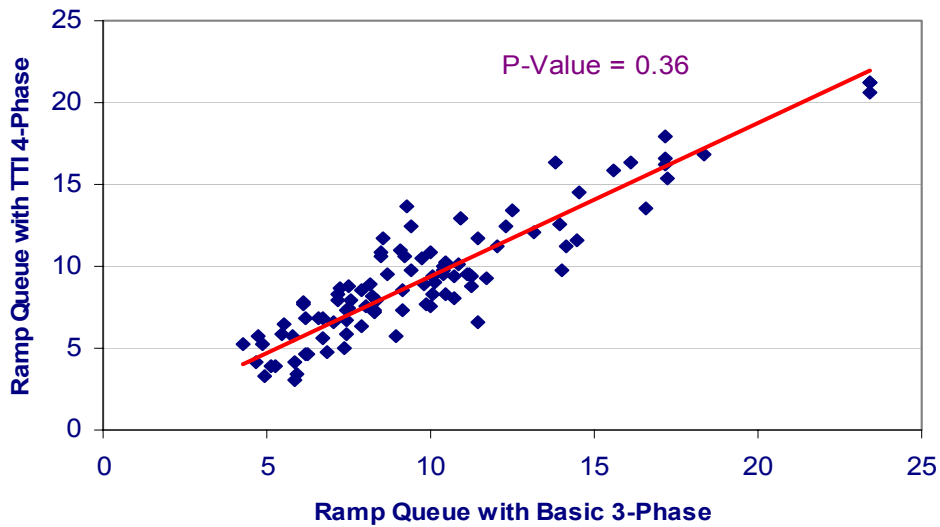


Figure 18. Queue Length Comparison with Basic Three-Phase and TTI Four-Phase.

Figure 19 and Figure 20 illustrate how traffic demand evolves, with basic three-phase and TTI four-phase at the diamond interchange, when the ramp demand exceeds its metering capacity, and queue spillback to the diamond interchange occurs. The traffic movements feeding Ramp 1 are shown in these figures (refer to Figure 3 for movement numbering schemes). For simplicity and demonstration purposes, constant traffic demands were assumed for the entire analysis period. Each line represents the traffic demand in vehicles per hour during the simulation period of 100 cycles. The leveled lines at the beginning indicate that ramp queues have not reached the diamond interchange yet; therefore, no queue spillback exists and no residual demand results. Once the ramp queue reaches the diamond interchange, queue spillback blocks these traffic movements and residual demands result. Such residual demands are shifted to the following cycles, resulting in a continued increase in traffic demands. Figure 19 shows the conditions with basic three-phase, and Figure 20 shows the conditions with TTI Four-Phase.

It is clearly seen that the demand increases faster for the arterial left-turn feeding movement with TTI four-phase than with basic three-phase, which confirms an earlier assumption that TTI four-phase favors the frontage road approach (M2 in this case) when queue spillback occurs.

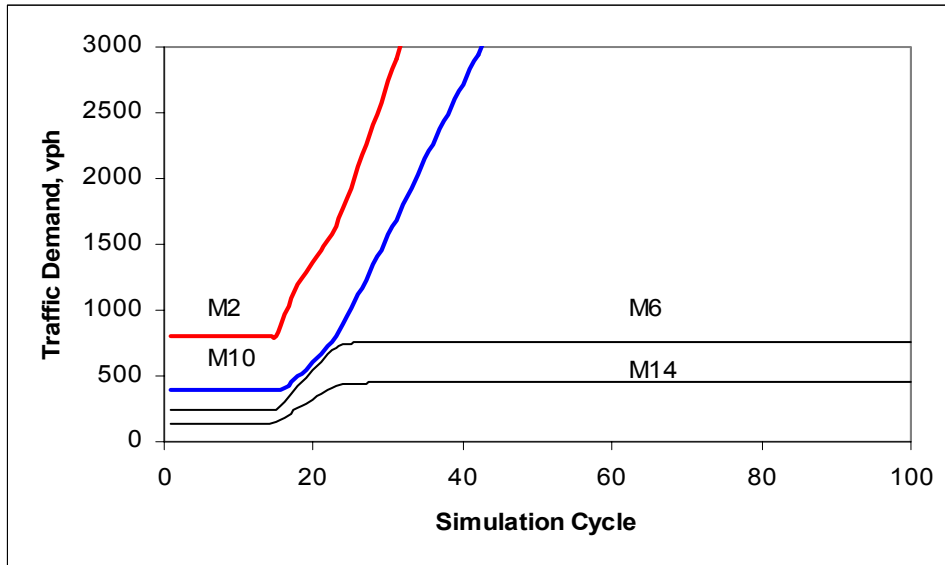


Figure 19. Traffic Demand Evolution with Queue Spillback - Basic Three-Phase.

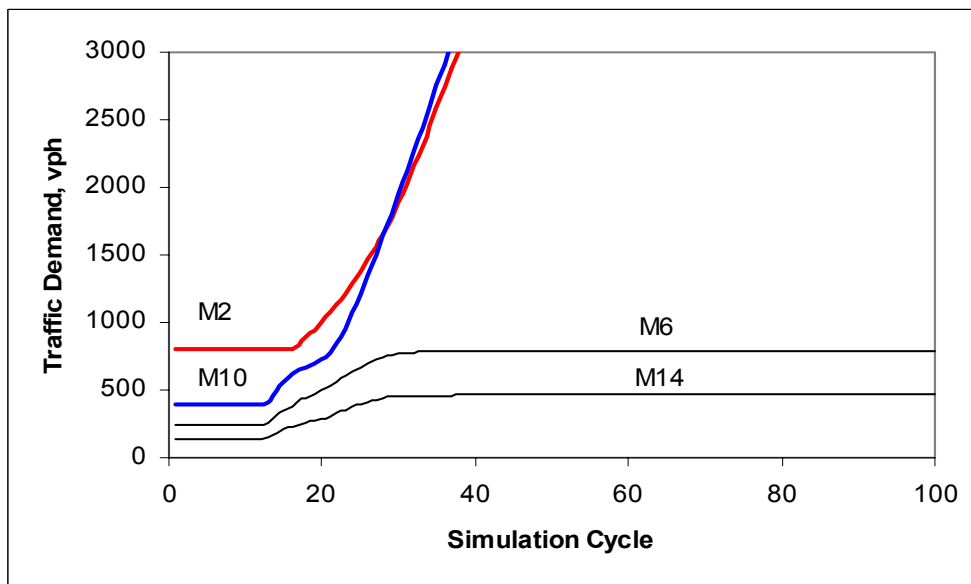


Figure 20. Traffic Demand Evolution with Queue Spillback - TTI Four-Phase.

Effects of Diamond Cycle Length on Ramp Queues

Another issue we wanted to explore was the effect of the cycle length used at the diamond on ramp queues. One potential strategy that could be used to manage traffic in the

diamond/ramp system would be to change the cycle length to minimize queue spillback. Experimental runs were conducted using the DRIVE software based on various cycle lengths at the diamond interchange and a fixed ramp metering rate. The ramp queues were recorded over the analysis period for different cycle lengths used. Figure 21 illustrates the estimated cumulative density functions for the ramp queues. These CDF curves provide any percentile queue length values that are counted over time. For example, with a cycle length of 60 seconds, the 80th percentile queue length is about three vehicles, i.e., during the entire analysis period, 80 percent of the time, the ramp queue length is less than three vehicles. It can be seen that with the increase of cycle length, the ramp queue length increases. This finding implies that under certain traffic conditions, such as low ramp demand level, the ramp queues may be effectively contained by simply using a shorter cycle length at the diamond interchange without having to get into more sophisticated operation and control systems. Of course, there will be limitations on how short the cycle length can be, either due to minimum phase constraints or capacity concerns at the diamond interchange.

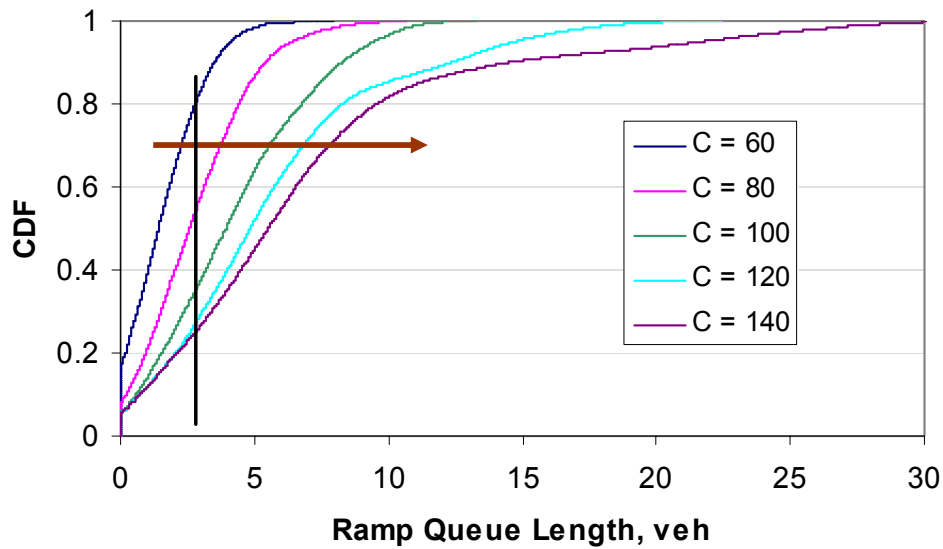


Figure 21. Effect of Cycle Length on Ramp Queue Length.

EFFECTS OF RAMP METERING OPERATIONS ON FREEWAY PERFORMANCE

The effects of ramp metering operations (e.g., responsive vs. fixed metering, metering with queue flush vs. without queue flush) on freeway operations were examined in this research. [Figure 22](#) and [Figure 23](#) illustrate the effects of operating traffic-responsive ramp metering and fixed ramp metering on freeway performances. These two figures show the freeway throughput during the entire 100 cycles (10,000 seconds) of simulation in DRIVE. As can be seen, with a responsive ramp metering operation, the freeway maintains a higher throughput and does not experience breakdown. With a fixed metering operation, however, breakdown occurs on the freeway about 1800 seconds after simulation, resulting in a reduced throughput. Ramp queue flush was assumed in both analyses once the queue exceeded the ramp storage space.

[Figure 24](#) shows the results when a fixed ramp-metering operation was assumed but without queue flush. It can be seen that freeway breakdown is delayed to about 5800 seconds after simulation, compared to 1800 seconds when the queue flush option was used. The result indicates that ramp metering with queue flush results in earlier freeway breakdown, and once breakdown occurs, it never recovers from it. Therefore, keeping ramp metering in operation for as long as possible is a preferred operating strategy, over metering with queue flush. It also suggests that once breakdown occurs and the same level mainline demand exists, ramp metering may no longer be effective in improving system operations.

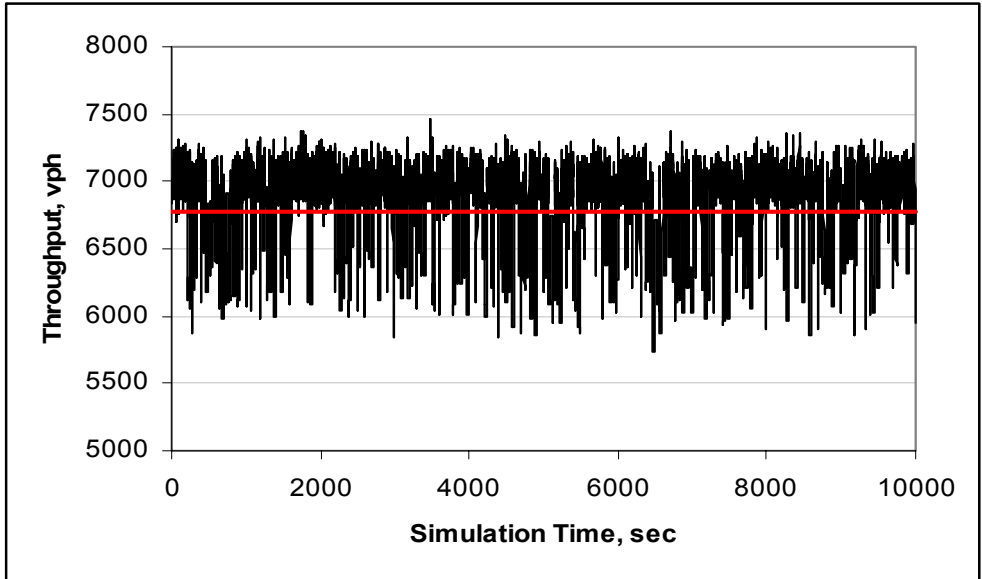


Figure 22. Freeway Throughput with Responsive Metering Rate.

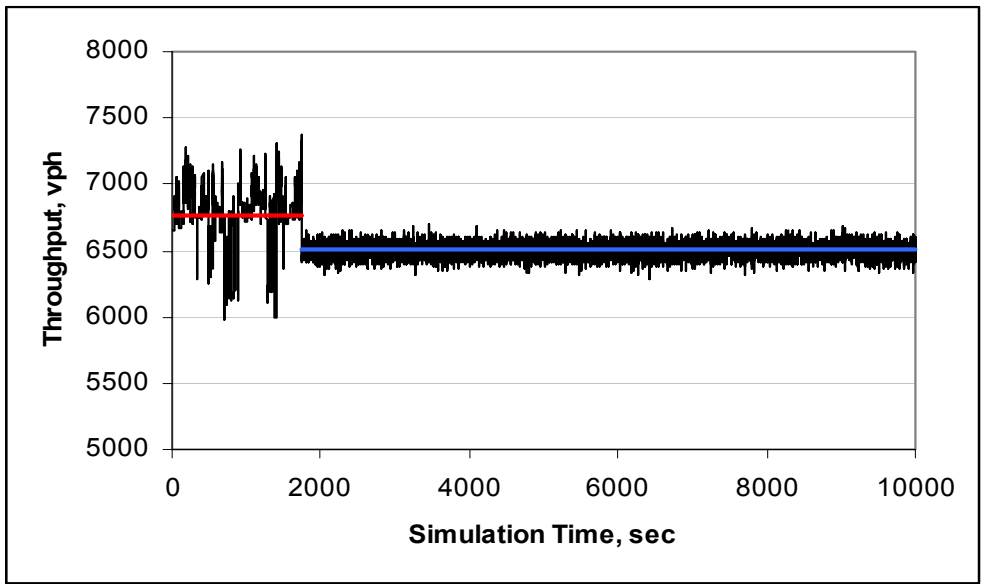


Figure 23. Freeway Throughput with Fixed Metering Rate and Queue Flush.

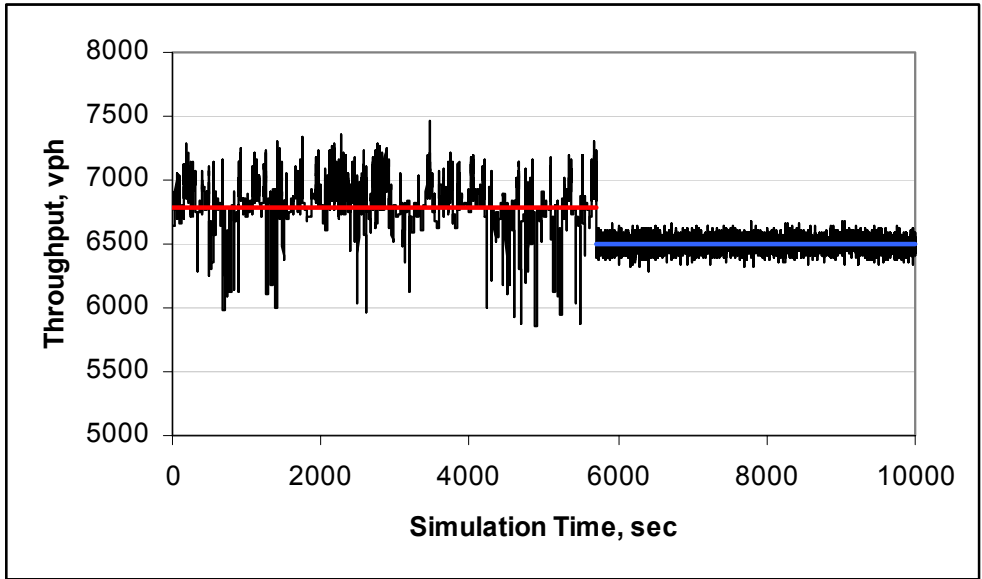


Figure 24. Freeway Throughput with Fixed Metering Rate and without Ramp Queue Flush.

CHAPTER 6: INTEGRATION STRATEGIES

Integrated operational strategies for IDIRMS are developed to achieve a primary system operational objective, i.e., to maintain ramp metering in operation through minimizing ramp queue spillback occurrences. This chapter documents the process of developing such integration strategies. Testing and evaluation of the strategies are conducted to demonstrate their applicability and effectiveness through a case study, and recommendations are provided based on study results.

RESOURCE MANAGEMENT PHILOSOPHY

The ultimate goal of integrated operational strategies is to achieve better management of available resources in IDIRMS under various traffic flow conditions. The strategies should be designed to respond to specific traffic conditions such as recurring and non-recurring traffic congestions. The strategies should be applicable in real-time traffic operations, where only the outcome could be measured but may not be well predicted. The key to a successful operational strategy relies on identification of all the critical elements and determining which elements can be managed and controlled. Operational strategies should address a broader range of impacts to the entire transportation system.

The resources within an IDIRMS include three major facilities: the freeway mainline, the ramp meters, and the diamond interchange. The properties of each facility that need to be managed include the capacities and the queue storage spaces of each facility. In general, we have little or no control over the freeway mainline demand (i.e., freeway-to-freeway demand). However, we could manage maintaining the freeway to operate at free-flow conditions through ramp metering so that the freeway would achieve the maximum throughput. Therefore, preventing or delaying the onset of freeway breakdown would be an essential step because freeway breakdown would result in a reduced throughput and would affect the entire system's operation. Both field operations and previous analyses indicate that ramp queue flush is one of the major causes of freeway breakdown. Flushing the queue at a ramp results in a sudden increase in freeway demands. The platoon suddenly entering the freeway as a result of flushing the queue also increases the likelihood of freeway breakdown. Therefore, an effective approach

to preventing freeway breakdown is to maintain ramp metering in operation without queue flush. However, ramp metering implies restricted ramp entry, thus increasing the likelihood of ramp queue spillback to the diamond interchange signal. Queue spillback to the diamond interchange can cause blockage to other traffic movements whose destinations are not the freeways via the on-ramps. Ramp queue spillback should be minimized through proactive control of the traffic movements feeding the on-ramps, which would only be achieved through adequate signal control at the diamond interchange. The best controlling location is therefore at the diamond interchange. Strategies to prevent queue spillback to the diamond interchange must then include managing the demand and storing the excessive queues outside the interchange, either on the arterial street approaches or on the frontage road approach. However, the most advantageous queue storage locations must be determined based on the analyses of potential impacts and operational trade-offs. Storing the queues on the arterial street approaches seems to be a preferred alternative because excessive queues on the frontage road approach may represent a more severe threat to system operations than queues on the arterial streets. Queues that spill back to the freeway mainline may interfere with the operations of the freeway mainline and result in reduced freeway throughput (22,23). Excessive queues on the frontage road approach would also cause blockage to the left-turn and right-turn movements, whose destinations are not the freeways. On the other hand, however, excessive queues on the arterial approaches may also interfere with other signalized intersections in the arterial. As a result, the available storage spaces on both locations have a limited range. The objective of the operational strategies, though, is to maximize the usage of these available queue storage spaces. When the last resort (i.e., all storage spaces) is used up, the excessive demands may eventually need to be released, such as terminating the ramp-metering operations. If ramp queue flush is considered as a failure event, the operational strategies should then delay its occurring but not necessarily completely avoid it.

THE SYSTEM OPERATING OBJECTIVES

As discussed in the [previous section](#), integrated operational strategies should be developed for achieving certain operating objectives. In this project, we focused on minimizing system delay as the ultimate goal although many other aspects of the operations, such as safety, may also play important roles. From the point of view of managing the entire system operations

and considering the trade-offs, the following operating objectives are identified for IDIRMS based on priority orders:

- 1) maintain freeway mainlines to operate at free-flow conditions and prevent or delay the onset of freeway breakdown;
- 2) prevent or minimize flushing ramp queues and maintain ramp metering in operation for as long as possible;
- 3) prevent or minimize ramp queue spillback to the diamond interchange signals;
- 4) control vehicle entries to the ramp meters through proactive signal operations and control at the diamond interchange;
- 5) store excessive demands and queues in the most advantageous locations, so that queue storage spaces can be efficiently used with minimum interference to freeway and adjacent arterial signal operations

INTEGRATION STRATEGIES

Integrated operational strategies are developed to achieve the identified operating objectives. The integration strategies are classified into three categories, with each strategy dealing with a specific traffic condition. The three strategies are categorized as: a) low level integration, b) recurring congestion, and c) non-recurring congestion. More discussions on each of these strategies are provided in this section.

Low Level Integration

This level of integration strategy is implemented simply through efficient management of the available resources without acquiring additional system equipment for operating more sophisticated detection and control systems. Examples of such strategies may include the following:

- adjusting the cycle length and splits at the diamond interchange,
- using a more efficient traffic-responsive ramp metering design, and
- setting back the location of the advance queue detector.

Adjustments to the diamond signal timing are limited to its cycle length and splits. For example, using a smaller cycle length can result in smaller cyclic queues. Increasing the splits for those movements feeding the ramp could also result in smaller cyclic queues. Adjustment to

ramp metering operations may involve changing the maximum and minimum metering rates to allow more flexibility of the ramp-metering flows to best utilize the freeway capacities. One alternative to increasing the metering rate is to allow bulk metering operations (i.e., multiple entry per green). The location of the advance queue detectors should be set back as far as possible to maximize ramp queue storage space. For example, the advance queue detectors may be better located on the frontage road instead of at the end of the on-ramp, as long as the ramp queues would not spill back to block the diamond interchange. These strategies are primarily fine tuning existing resources and therefore may be treated as mitigation measures under low traffic demand conditions. The DRIVE model would provide assessments of the traffic conditions and whether such operational strategies would provide satisfactory system performance measures. Traffic demand levels may be obtained either through historical field measurements or demand forecasts.

Recurring Congestion

Recurring congestion refers to the situation where demand regularly and repeatedly exceeds capacity. For the purposes of this research, two types of recurring congestion have been defined: short term and long term, which are all subjectively defined. Short-term recurring congestion refers to the situation where over-saturation lasts only for a short period of time (e.g., a period of 15 minutes or a few signal cycles at the diamond). Long-term recurring congestion refers to the situation where over-saturation may last for a prolonged period (e.g., at least 30 minutes or more). Both short-term and long-term congestions are encountered in daily operations and therefore are the focus of this research.

Under the conditions of short-term congestion where the ramp queues may spill back to the diamond interchange, significant adjustment to the normal signal operations may be necessary, including special timing plans or adaptive signal control capabilities. For example, in order to prevent queue spillback to the diamond interchange, the traffic movements feeding the ramps may be restricted of entry to the downstream frontage road link. Such a control measure could not be accomplished by some minor adjustments of the signal cycle lengths and splits. Special operations may be necessary to achieve restricted vehicle entry such as using all-red extensions and holding a particular signal phase. All-red extension implies displaying extended red signal indications as a means to stop traffic going through the interchange. Such an

operation, although it may serve the purpose of restricting vehicle flows, may not be a practical application. The operational strategies developed in this research are based on the principle of holding a particular signal phase to achieve the objective of restricted vehicle entry. When congestion lasts for a prolonged period, strategies should also focus on better managing the vehicle queues within the available resources, primarily on the arterial approaches and the frontage road/off-ramp approaches. Restricted ramp entry would result in excessive traffic demands and queues, which have to be stored outside the diamond interchange. Strategies should be developed to facilitate storing excessive vehicle queues in the most advantageous locations.

To prevent ramp queue spillback, the diamond interchange signal must be able to sense any ramp queue buildup and respond with adequate signal control and operations. Therefore, the system must have the capabilities of adaptive control features, which would require additional detection, communication, and signal control devices. In fact, such a system can be developed with the existing functions and features of most modern traffic signal controllers. Additional deployments of vehicle detectors are also necessary to achieve the adaptive operations. [Figure 25](#) is a proposed system design, where additional detectors are installed in addition to the detectors used for a standard diamond interchange and a traffic-responsive ramp-metering system. The additional detectors on the arterial approaches and the freeway off-ramps serve the purpose of detecting excessive vehicle queues so that the system can respond to traffic queues and prevent further spillback that would interfere with freeway mainline and adjacent signalized intersections. The queue detectors on the frontage road downstream of the diamond signal serve as the means of detecting ramp queue buildup so that special signal timing, such as phase holding, can be implemented at the diamond interchange signal.

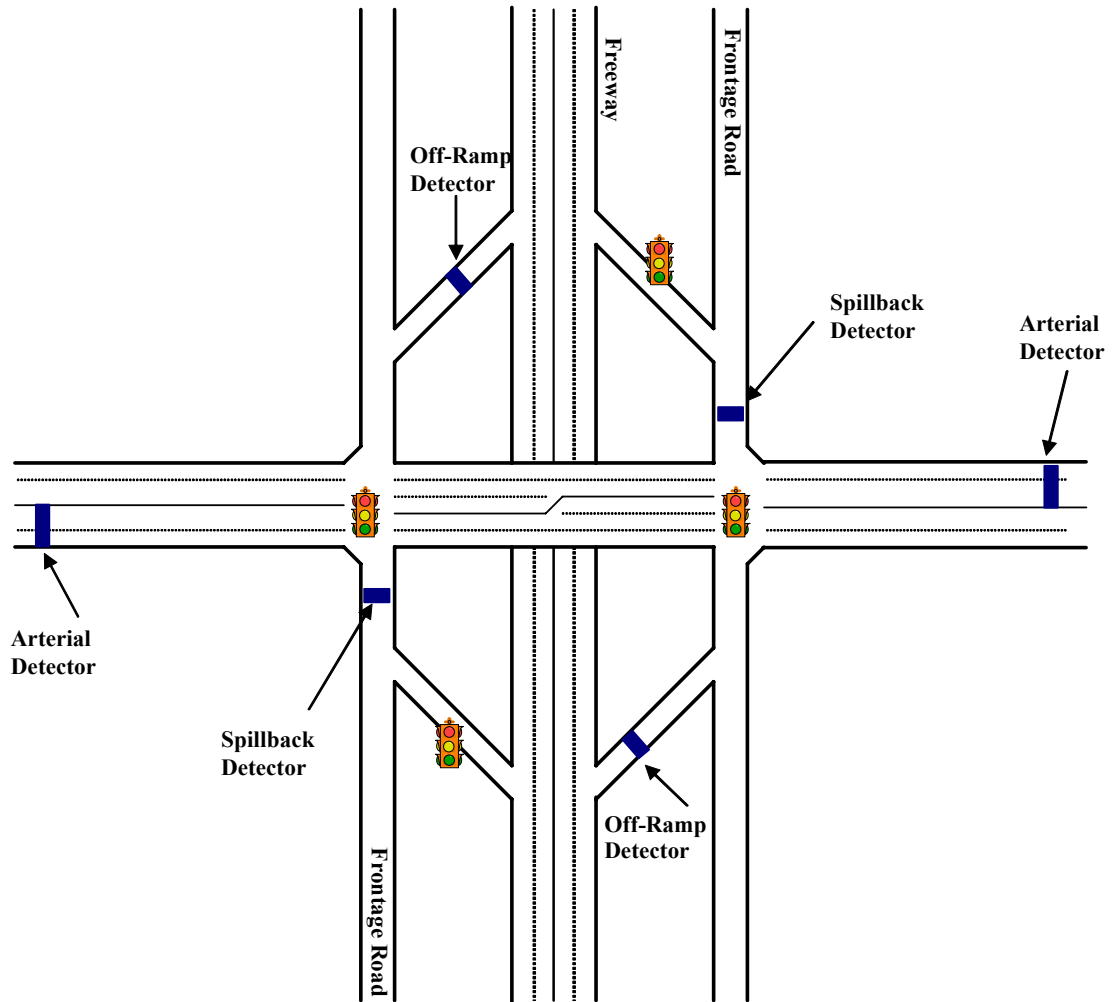


Figure 25. Additional Detector Layouts for the Proposed Adaptive Control System.

The principles of operation for the adaptive control system are described in this section. The diamond interchange signals would remain in normal operations if none of the queue detectors (i.e., arterial detectors, off-ramp detectors, and spillback detectors) detects traffic queues. In the VISSIM simulation model, the occupancy of a queue detector is sampled once every 20 seconds. A traffic queue is defined when the occupancy reaches 60 percent or higher. Whenever a ramp queue is detected by the spillback queue detector, the diamond signal would quickly transition to a particular signal phase to hold so that minimum vehicle entry to the ramp would result, and queue spillback to the diamond interchange signal can be prevented. The diamond signal would go back to normal operation once the ramp queue was dissipated. The location of the queue spillback detector should be some distance away from the diamond signal

to avoid queue spillback occurring during the transition periods between normal diamond signal operations. The signal phase for holding should be the one that would result in no vehicle entry to the ramp from those traffic movements feeding the ramp meter (e.g., the through movement on the frontage road approach and the left-turn movement on the internal arterial street approach). The green splits after the holding phase may be designed to facilitate clearing excessive queues resulting from the phase hold. Ramp metering would remain in operation until all the queue storage spaces are filled up, i.e., queues reach all the queue detector locations on both the arterial streets and freeway off-ramps.

Figure 26 through Figure 28 illustrate the proposed phases to hold with basic three-phase under the proposed adaptive control to prevent ramp queue spillback. Figure 26 shows the holding phases to be the internal left-turn phases ($\phi 1$ and $\phi 5$). By holding on these phases, no further vehicle entries to the metered ramps would result (except for uncontrolled arterial right-turn and U-turn traffic). Holding the internal left-turn phases would provide equal treatment to the two metered ramps; therefore, it would be suitable when the two ramp meters have similar traffic conditions. There is a disadvantage for holding the internal phases, i.e., arterial through traffic would be stopped and unnecessary delays would incur. Figure 27 shows the holding phases to be the arterial through phases ($\phi 2$ and $\phi 6$). Although control of vehicle entry to the ramps would also be achieved by holding these phases, it has the potential of vehicle queue spillover for the internal left-turn movements, which may cause blockage to the two diamond intersections. However, the advantage of holding the arterial through phases is to allow arterial through traffic to go through the interchange so that unnecessary delays to these vehicles can be avoided. Figure 28 shows the holding phases to be the frontage road phases and when the diamond interchange operates with a special feature called conditional service. With conditional service, an additional arterial left-turn phase ($\phi 1$ as shown in the figure) can be serviced while one of the frontage road phases is being serviced ($\phi 8$ as shown in the illustrated case). The use of conditional service would result in unequal treatment to the two ramp meters. As shown in this case, holding the frontage road phase ($\phi 8$) would restrict vehicle entry to the left-side ramp meter (R_1).

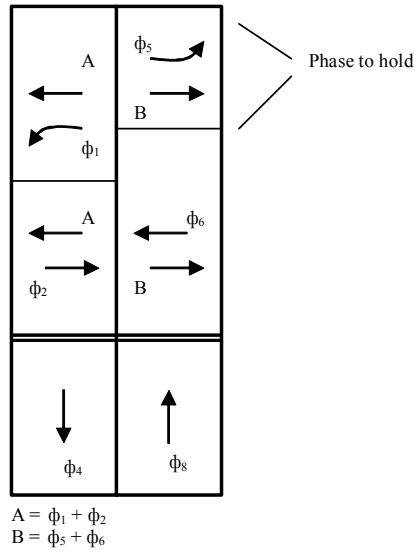


Figure 26. Hold Internal Phases with Basic Three-Phase.

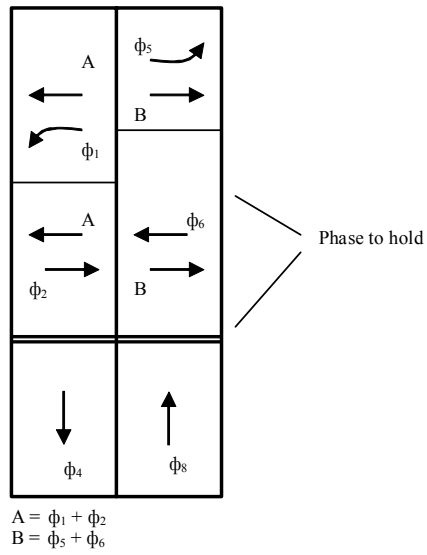


Figure 27. Hold Arterial Phases with Basic Three-Phase.

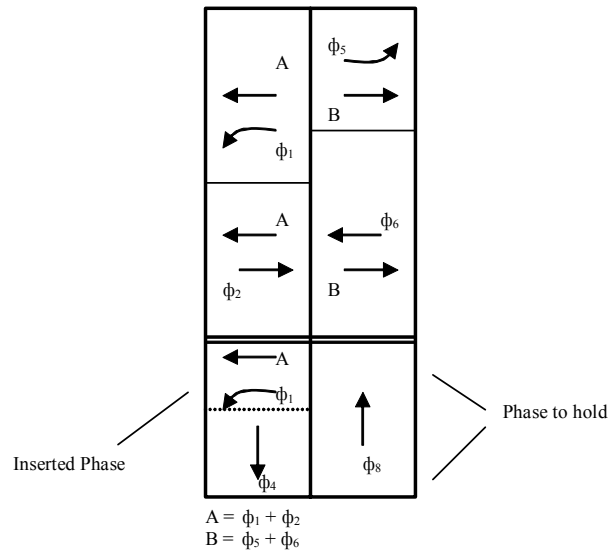


Figure 28. Hold Frontage Road Phase with Basic Three-Phase and Conditional Service.

Figure 29 and Figure 30 show the proposed holding phases when the diamond signals operate with TTI four-phase. These figures illustrate the holding phases, either the right-side frontage road phase (ϕ_8) or the arterial through phase (ϕ_2), to control vehicle entry to the left-side ramp (R_1). Similarly, ϕ_4 and ϕ_6 would be the holding phases if vehicle entry to the right-side ramp (R_2) needs to be controlled.

As can be seen, the majority of the operational strategies to control queue spillback with phase holding would normally provide control on one ramp, which would be appropriate when the ramps have different traffic conditions, such as one ramp having higher demands than the other. The only option for providing equal treatment on both ramps is to use basic three-phase with the holding phases being the internal left-turn phases.

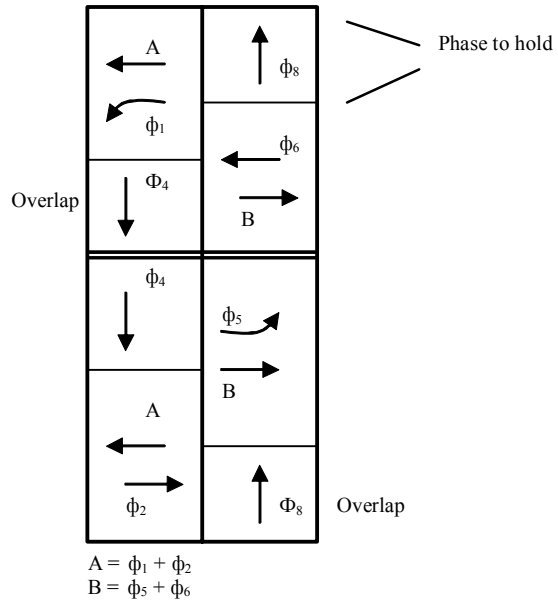


Figure 29. Hold Frontage Road Phase (ϕ_8) to Control Left-Side Ramp Entry with TTI Four-Phase.

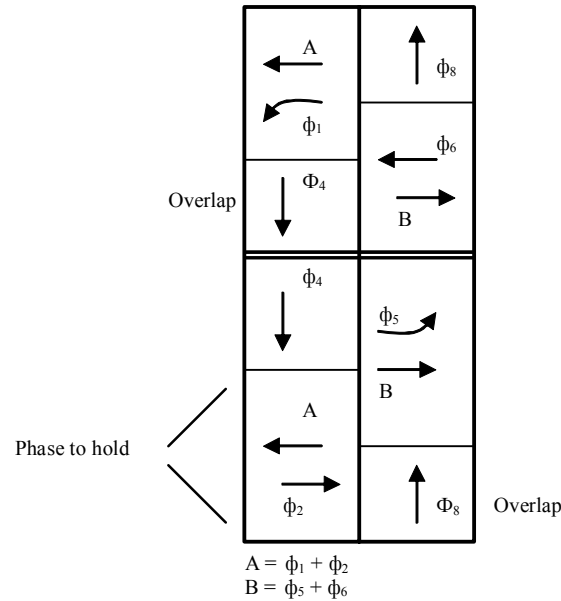


Figure 30. Hold Arterial Phase (ϕ_2) to Control Left-Side Ramp Entry with TTI Four-Phase.

Non-recurring Congestion

Non-recurring (also called incident) congestion is a result of incident conditions, where dramatic change in traffic flow patterns may result. Non-recurring congestion often creates significant turbulence to normal traffic operations. Incidents can block freeway lanes and distract drivers, and thus may significantly reduce freeway throughputs. Incidents could also result in travel pattern changes due to traffic diversion from routine travel paths. The travel pattern change and traffic diversion will depend on the nature of an incident, such as its location and its severity. The changes in traffic flow patterns would require special treatments on the diamond signal timing and ramp-metering operations. [Figure 31](#) and [Figure 32](#) illustrate the two types of incident locations and the possible traffic diversions. For example, an incident occurring within the interchange ([Figure 31](#)) may result in traffic diversion via the off-ramp, and the on-ramp demand would increase while the mainline demand at the ramp merge location would decrease. Strategies dealing with such an incident may involve increasing the ramp-metering rate or metering suspension, and increasing the green split for the frontage road/off-ramp approach ([21](#)).

On the other hand, incident occurring downstream of the on-ramp (Figure 32) would result in reduced freeway capacity. While traffic may divert through the off-ramp and attempt to enter the freeway at the on-ramp, the reduced downstream capacity may require restricted ramp entry. Therefore, the strategies to deal with such an incident may involve measures such as ramp closure. Ramp closure would result in traffic diversion to the downstream diamond interchange, where the potential impact on the downstream interchange should be further evaluated. This project does not go beyond the scope of an isolated diamond interchange location. Therefore, strategies dealing with incident conditions are not further addressed in this current scope.

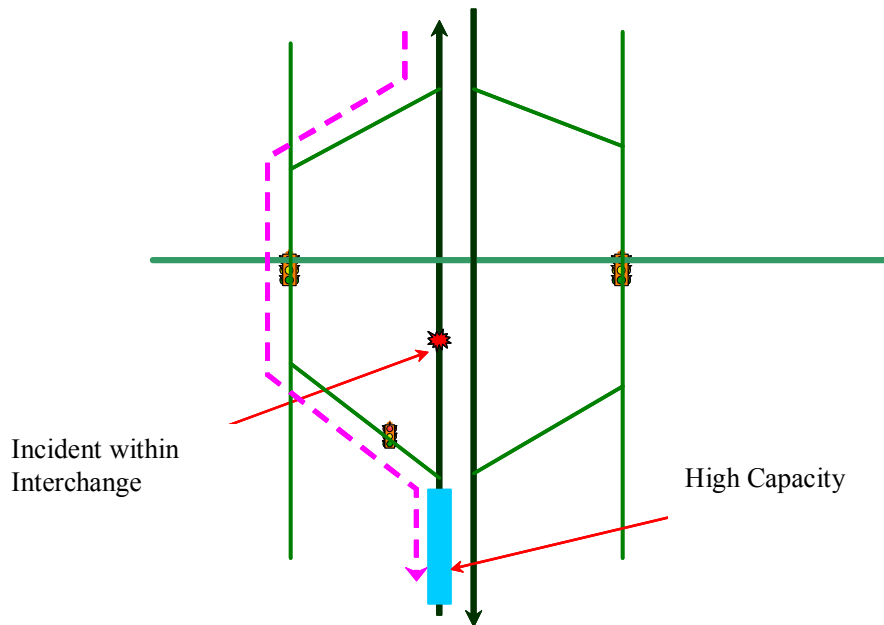


Figure 31. Incident within Interchange.

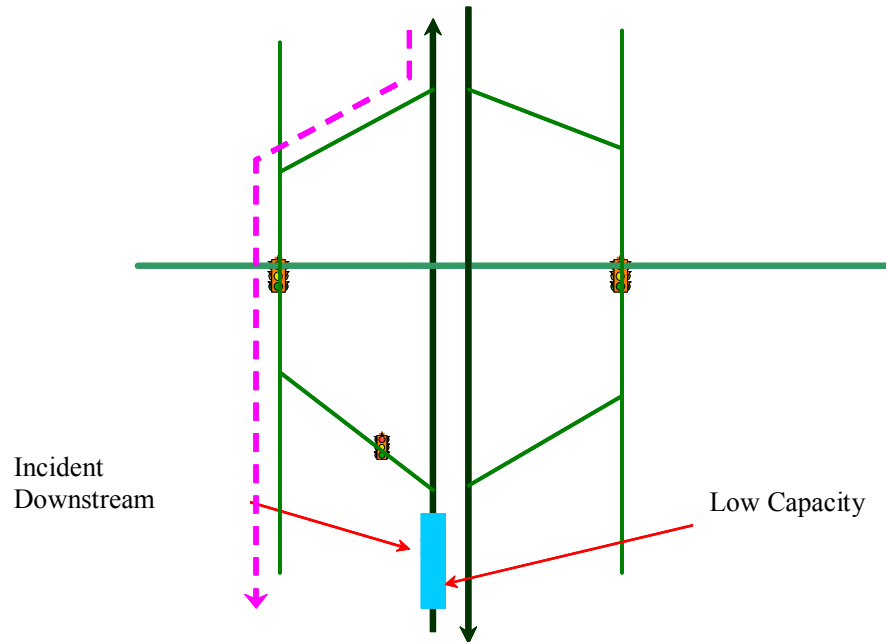


Figure 32. Incident Downstream of Interchange.

EVALUATION OF OPERATIONAL STRATEGIES UNDER RECURRING CONGESTION

The integrated operational strategies discussed in this chapter for ramp queue spillback control under recurring congestion were evaluated using the VISSIM simulation model. [Figure 33](#) and [Figure 34](#) compare the number of queue flushes and the total queue flush durations with and without the integrated operations. The cumulative queue flush duration is shown in the y-axis. The horizontal segment indicates the time during which no queue flush occurred; therefore, the number of queue flushes can be obtained by counting the number of horizontal segments. [Figure 33](#) illustrates a low demand situation, and [Figure 34](#) illustrates a high demand situation. Under a low traffic demand level, queue flush occurred only once (about an 80 sec duration) with integration compared to five times (about 180 seconds) without integration. The integration strategy effectively reduced the number of queue flushes and queue flush durations under a low demand level.

Under a higher demand level as shown in [Figure 34](#), the starting point of the queue flush was delayed by about 1800 seconds with integration; however, no significant difference is observed between the total queue flush durations, whether integrated operations were

implemented or not. This is a situation where freeway breakdown occurred after the first queue flush, but the freeway never recovered from breakdown. As a result, the freeway remained at the lower throughput level under breakdown. In this case, maintaining integrated control and ramp-metering operation may no longer be effective in terms of reducing congestion and delay.

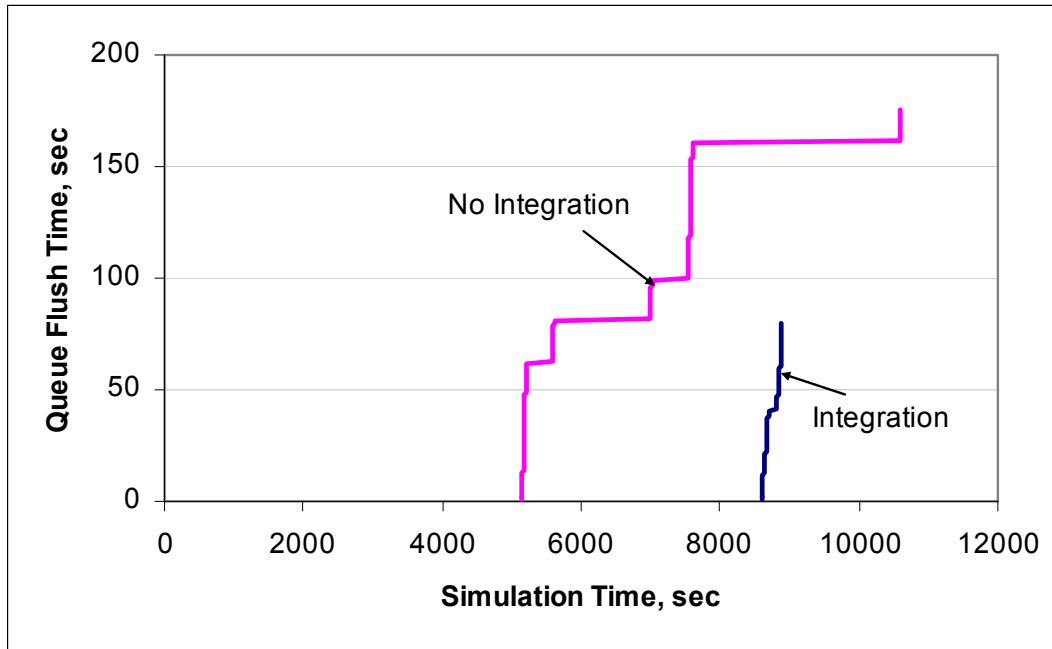


Figure 33. Queue Flush Duration with/without Integration – Low Demand.

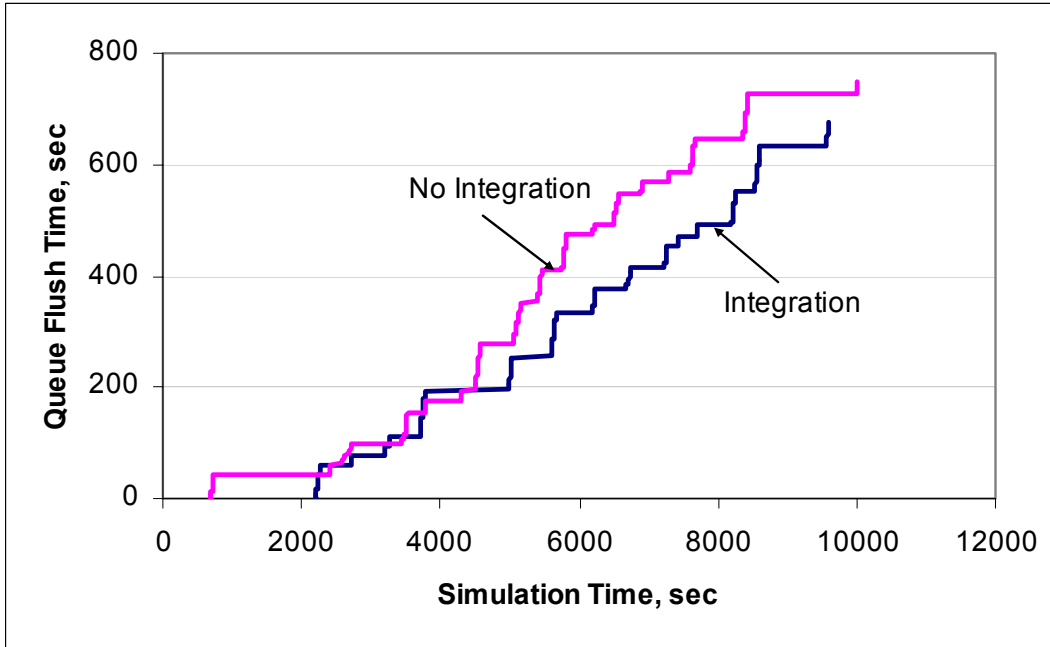


Figure 34. Queue Flush Duration with/without Integration – High Demand.

Figure 35 illustrates the delay evolutions on the freeway for cases with and without integrated operations. As can be seen, the onset of congestion on the freeway mainline was effectively postponed with the integrated operation. It can also be observed from the figure that once freeway breakdown and congestion occur, no significant difference is observed on freeway delays with or without integrated operations, which once again confirms that ramp metering may no longer be necessary once freeway breakdown occurs.

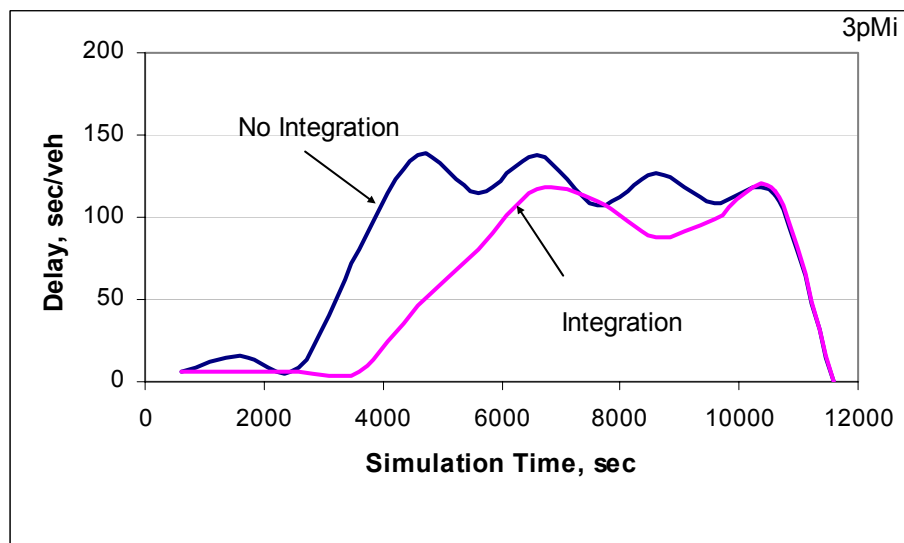


Figure 35. Delay Evolution on Freeway with/without Integration.

A FRAMEWORK FOR SELECTING OPERATIONAL STRATEGIES

In order to use the proposed integrated operational strategies for IDIRMS, traffic engineers at traffic management and control centers would need guidelines and procedures to assess traffic conditions and determine the best strategies. A framework is proposed in this section to serve such a purpose. The framework describes the necessary steps for traffic engineers to assess traffic situations and select appropriate integration and control strategies. The framework is described in the following steps.

Step 1: Assess Traffic Operations using DRIVE under Normal Traffic Conditions

With the system configuration, geometry, and projected traffic demand information, IDIRMS operations can be evaluated using DRIVE. Decisions can be made on what level of integration and operational strategies should be deployed to achieve the operating objectives. DRIVE would provide assessments on the following conditions:

- Does the freeway traffic meet the ramp-metering threshold?
- Can the ramp accommodate the maximum queues with normal signal timing and operations at the diamond interchange?
- Can the ramp-metering operations be modified (e.g., from fixed to traffic responsive) to accommodate the maximum queues?
- Can the advanced detectors be set further back (e.g., on the frontage road) to accommodate longer ramp queues without having to flush the queues?
- Is there queue spillback to the diamond interchange, and is the level of spillback at an acceptable level (e.g., what percent of the time would queue spillback occur)?

Step 2: Determine Integration and Operational Strategies

If the analysis in Step 1 indicates any of the following conditions, the low level integration strategies should be selected, which would not require additional system deployments beyond the existing system components:

- the ramp metering threshold is not met;
- ramp queues can be accommodated through modification of either diamond interchange signal timing or ramp metering operations; or

- the level of ramp queue spillback is acceptable to traffic management personnel (say, a very low probability with less than 5 percent)

If none of the above conditions exist, i.e., ramp queue and queue spillback cannot be contained with the low level integration strategies, recurring congestion strategies (under normal traffic conditions) or non-recurring congestion strategies (incident conditions) should be selected.

Step 3: Select Strategies under Recurring Congestion

When dealing with recurring congestion, the following procedure should follow:

- Assess the traffic flow conditions at the diamond interchange and the ramps to see whether directional flows are balanced or unbalanced.
- Determine the phasing strategies for the diamond interchange, either basic three-Phase or TTI four-phase.
- Select a phase hold strategy and determine adequate phase splits to achieve balanced service on traffic movements feeding the ramp meters.
- Continuously monitor freeway conditions for breakdown occurrences. Once the freeway experiences breakdown for a prolonged period (e.g., 10 minutes or more), turn the ramp metering off and go back to normal diamond interchange operations.

Step 4: Select Strategies under Non-recurring Congestion

When dealing with non-recurring congestion, the following procedure should follow:

- Identify incident locations, whether downstream of the interchange, upstream of the interchange, or within the interchange.
- Monitor any traffic pattern changes and possible diversions.
- Adjust diamond signal timing to accommodate diverted traffic flows.
- Select ramp control strategies, including ramp closure or turning ramp metering off.

CHAPTER 7: SUMMARY AND CONCLUSIONS

This research project aimed at developing operational strategies for better managing an integrated diamond interchange/ramp-metering system. Methodologies were developed to model the system performances with integrated operations. The methodologies were implemented into a computer model DRIVE. DRIVE is classified as a mesoscopic simulation model. Operational characteristics were investigated using DRIVE to gain better understanding of the integrated operations. DRIVE was validated against the VISSIM microscopic simulation model. Operational strategies were developed with the two types of commonly used diamond phasing control: basic three-phase and TTI four-phase. The researchers came to the following conclusions and recommendations:

- The proposed modeling procedure takes into account the close interactions between ramp operations and upstream diamond interchange operations. The impact of queue spillback from ramp meters was specifically modeled, which enhances the current state-of-the-art modeling techniques in the area of diamond interchange operations considering the impact of downstream ramp metering.
- DRIVE was validated against the VISSIM microscopic simulation model, and researchers found general agreement between the two models. However, significant variations were found for freeway delays, primarily attributed to the modeling methodologies of freeway breakdown. VISSIM indicated much quicker recovery from breakdown than what had been observed in field operations. Further studies of modeling freeway breakdown are necessary to confirm the validity of both models.
- Analyses of system operational characteristics revealed that basic three-phase and TTI four-phase do not result in significant difference in terms of ramp operations during under-saturated conditions. However, when ramp queue spillback does occur, TTI four-phase favors the frontage road movements.
- Recommended practice is to maintain ramp metering in operation for as long as possible and avoid ramp queue flush. Queue flush results in freeway breakdown and reduced system throughput. The most advantageous queue storage location is on the arterial approaches. Ramp queue spillback on the frontage road would result in blockage of other

movements, with particular concerns for interfering with freeway operations once queue spillback occurs on the freeway off-ramp.

- Operational strategies to control queue spillback rely on an adaptive system that would respond dynamically to ramp queue buildup. The adaptive system requires additional system detectors. The system would place a hold on a particular phase to control queue spillback at the ramp meter. Testing of the strategies indicated that the onset of freeway congestion can be effectively delayed. Research results also suggest that maintaining integrated operations and ramp metering may no longer be effective once the freeway experiences breakdown and with sustained high freeway traffic demands.
- A framework was developed for traffic management personnel to assess traffic conditions and select adequate operational strategies.
- Several research areas were identified for further research. Modeling freeway breakdown and its impact need further study using field evidence. Field implementation and testing are necessary steps to evaluate the effectiveness of the proposed integration strategies. The DRIVE model needs to be enhanced to model the advanced adaptive features of the system under queue spillback conditions.

REFERENCES

1. Messer, C.J., and Bonneson, J.A., *Capacity Analysis of Interchange Ramp Terminals*, Final Report, National Cooperative Highway Research Program 3-47, Transportation Research Board, Washington D.C., April 1997.
2. Kim, Y., and Messer, C.J., *Traffic Signal Timing Models for Oversaturated Signalized Interchanges*, Research Report 1148-2, Texas Transportation Institute, College Station, Texas, January 1992.
3. Turner-Fairbank Highway Research Center, *Coordinated Operation of Ramp Metering and Adjacent Traffic Signal Control Systems*, FHWA-RD-95-130, Washington, D.C., June 1996.
4. Tian, Z., *Implementation and Operations Guidelines on Coordinated Ramp Metering Systems*, Compendium of Papers, Advanced Transportation Institute, Texas A&M University, College Station, Texas, August 2002.
5. Messer, C.J., and Berry, D.J., *Effects of Design Alternatives on Quality of Service at Signalized Diamond Interchanges*, Transportation Research Record 538, January 1975, pp. 20-31.
6. Messer, C.J., Fambro, D.B., and Richards, S.H., *Optimization of Pretimed Signalized Diamond Interchanges*, Transportation Research Record 644, January 1977, pp. 78-84.
7. Gordon, R.L., *Algorithm for Controlling Spillback from Ramp Meters*, Transportation Research Record 1554, January 1996, pp. 162-171.
8. Papageorgiou, M., Hadj-Salem, H., and Middleham, F., *ALINEA: A Local Feedback Control Law for On-Ramp Metering*, Transportation Research Record 1320, January 1991, pp. 58-64.
9. Oh, H.U., and Sisiopiku, V.P., *A Modified ALINEA Ramp Metering Model*, Transportation Research Board Meeting, Preprint CD-ROM, Washington D.C., January 2001.
10. PTV, *VISSIM User Manual 3.60*, Stumpfstraße 1, D-76131 Karlsruhe, Germany, June, 2002.
11. Ploss, G., and Keller, H., *Dynamic Estimation of Origin and Destination Flows from Traffic Counts in Networks*, Proceedings of International Conference on Transportation Systems Studies, Tata McGraw-Hill, Delhi, India, July 1986, pp. 211-221.

-
12. Dixon, M.P., and Rilett, L.R., *Real-time OD Estimation Using Automatic Vehicle Identification and Traffic Count Data*, Computer-Aided Civil and Infrastructure Engineering, Vol. 17, Issue 1, 2002, pp. 7-21.
 13. Mountain, L.J. and Westwell, P., *The Accuracy of Estimation of Turning Flows from Automatic Counts*, Traffic Engineering and Control, No. 1, Vol. 24, 1983, pp. 3-7.
 14. Texas Transportation Institute, *PASSER III-98 User's Manual*, College Station, Texas, 1998.
 15. Transportation Research Board, *Highway Capacity Manual 2000*, Washington D.C., 2000.
 16. Wagner, F.A., Gerlough, D.L., and Barnes, F.C., *Improved Criteria for Traffic Signal Systems on Urban Arterials*, NCHRP Report 73, Transportation Research Board, Washington, D.C., January 1969.
 17. Wagner, F.A., Barnes, F.C., and Gerlough, D.L., *Improved Criteria for Traffic Signal Systems in Urban Networks*, NCHRP Report 124, Transportation Research Board, Washington, D.C., January 1971.
 18. Roupail, N.M. and Akcelik, R., *A Preliminary Model of Queue Interaction at Signalized Paired Intersections*, Proc. 16th ARRB Conference, pp. 325-345, Australian Road Research Board Ltd., Vermont South, Victoria, Australia.
 19. Prosser, N., and Dunne, M., *A Procedure for Estimating Movement Capacities at Signalized Paired Intersections*, Proceedings of the 2nd International Symposium on Highway Capacity, 1994, pp. 473-492.
 20. Webster, F.V., *Traffic Signal Settings*, Her Majesty's Stationery Office, Road Research Technical Paper 39, London, 1958.
 21. Tian, Z., Balke, K., Engelbrecht, R., and Rilett, L., *Integrated Control Strategies for Surface Street and Freeway Systems*, Transportation Research Record 1811, 2002, pp. 92-99.
 22. Munoz, J.C., and Daganzo, C.F., *The Bottleneck Mechanism of a Freeway Diverge*, Transportation Research A, Volume 36A, No. 6, July 2002, pp. 483-506.
 23. Cassidy, M.J., Anani, S.B., and Haigwood, J.M., *Study of Freeway Traffic near an Off-Ramp*, Transportation Research A, Volume 36A, No. 6, July 2002, pp. 563-572.