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for Project:

Interchange Design to Accommodate Ramp Metering System

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SI CONVERSION FACTORS

Approximate Conversions to SI Units				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
Length				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
Area				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
Volume				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
Mass				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
Temperature (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
Illumination				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
Force and Pressure or Stress				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

Approximate Conversions from SI Units				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
Length				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
Area				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
Volume				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
Mass				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
Temperature (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
Illumination				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
Force and Pressure or Stress				
N	newtons	02.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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16. Abstract

The majority of the existing research regarding ramp metering has largely been focused on ramp signal timing algorithms and/or the design of the on-ramp itself for accommodating ramp metering operations, irrespective of the particular overall interchange design. However, what is generally lacking is research that considers how ramp metering should factor into the selection of an overall interchange configuration.

The objective of this project was to develop guidance on interchange design, with consideration of ramp metering, that is appropriate for a planning and preliminary engineering level. In other words, guidance is provided to help understand the impact of ramp metering on interchange operations across a variety of interchange designs. Detailed operational questions (e.g., signal phasing sequences, signal coordination settings amongst the ramp terminal and adjacent intersections, the type of ramp-metering control algorithm, interactions between adjacent interchanges, and similar) are beyond the scope of the guidance developed from this project.

Specific topics addressed in this report include: ramp meter to freeway merge acceleration distance, multilane metering, performance measures, interchange configuration factors, and macroscopic queuing analysis. The conclusions and recommendations from this study are intended for incorporation into the FDOT Interchange Access Request User's Guide.

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EXECUTIVE SUMMARY

The Interchange Access Request User's Guide (FDOT, 2018) describes the process for requesting the modification of an existing interchange or constructing a new interchange on the interstate system. More specifically, the purpose of this document, as provided in the document is as follows: "The purpose of this User's Guide is to provide guidance on how to prepare documents that support requests for new or modified access to the Florida Interstate Highway System and non-interstate limited access facilities on the SHS. This User's Guide also provides information on the IAR [Interchange Access Request] process that shall consider the needs of the system at a regional level while maintaining the integrity of the highway network."

However, this document currently indicates that the addition of ramp metering is not subject to the review process for either an interchange modification request (IMR) or interchange justification request (IJR). Nonetheless, interchange configuration can significantly influence the level of effectiveness of ramp metering operations, in terms of balancing the competing objectives between the freeway mainline and the adjoining arterial roadway.

From an operations perspective, the purpose of ramp metering is to regulate the flow of traffic onto the freeway mainline in order to prevent breakdown. Such on-ramp restriction can often result in negative consequences for the connecting arterial roadway. For example, queue backup from the ramp meter can interfere with the operation of the arterial. Maximizing the efficiency of the arterial would generally preclude on-ramp restrictions. Achieving reasonable interchange traffic operational quality is more likely if the design of the interchange explicitly considers ramp metering operations.

The majority of the existing research regarding ramp metering has largely been focused on ramp signal timing algorithms and/or the design of the on-ramp itself for accommodating ramp metering operations, irrespective of the particular overall interchange design. However, what is generally lacking is research that considers how ramp metering should factor into the selection of an overall interchange configuration.

The objective of this project was to develop guidance on interchange design, with consideration of ramp metering, that is appropriate for a planning and preliminary engineering level. In other words, guidance is provided to help understand the impact of ramp metering on interchange operations across a variety of interchange designs. Advantages and disadvantages of different designs as well as guidance on how some specific geometric design elements might mitigate or exacerbate ramp-metering operations at the interchange is also provided. However, detailed operational questions (e.g., signal phasing sequences, signal coordination settings amongst the ramp terminal and adjacent intersections, the type of ramp-metering control algorithm, interactions between adjacent interchanges, and similar) are beyond the scope of the guidance developed from this project. Once a general interchange configuration and ramp metering strategy or algorithm are determined, detailed operational questions could be addressed through microscopic traffic simulation or comparable methods. Such issues could also be the subject of future research. Specific topics addressed in this report include:

- Ramp meter to freeway merge acceleration distance
- Multilane metering

- Performance measures
- Interchange configuration factors
- Macroscopic queuing analysis.

The following interchange forms were considered in this study:

- Diamond
- DDI (Diverging Diamond Interchange)
- SPUI (Single Point Urban Interchange)
- Partial cloverleaf (ParClo)
- Full cloverleaf (FullClo) with a collector-distributor (C-D)
- Full cloverleaf without a collector-distributor.

Several interchange performance measures were examined in this study: volume throughput, average travel speed, ramp meter queue discharge rate, ramp meter delay, and percent time spent in queue override mode. All of these measures are interrelated, but the percent time spent in queue override mode is the most intuitive in terms of assessing whether an interchange can avoid the very undesirable situation of on-ramp queues backing up into the adjoining arterial roadway. Not only can this create an operational problem for the arterial, but depending on the ramp terminal configuration, it may also pose a safety issue. Furthermore, increasing the metering rate to reduce the queue length is also especially undesirable from a freeway operations perspective, as it increases the likelihood of a mainline flow breakdown. Thus, the percentage of time that the meter is set to its maximum rate ('queue-flush' mode) is arguably the most significant performance measure for interchange operational quality, when ramp metering is present.

Of the interchange forms examined in this project, the DDI provides the best compromise between right-of-way footprint and ability to accommodate a wide variety of traffic characteristics with comparable or better performance measure results than the other forms. If extended arterial progression, particularly two-way, is a major issue, then the SPUI may be a preferred alternative. The ParClo design is also a reasonable alternative for ramp metering implementations, as long as the loop ramp ties into the direct ramp before merging with the freeway mainline (to create additional queue storage). Because of the independent lanes for left- and right-turning traffic, the performance of the ParClo varies more than the DDI with varying traffic conditions. Adding ramp metering to a FullClo without C-D design is not recommended due to more limited queue storage and reduced acceleration distance for the loop ramps. At a minimum, construction of a C-D lane is recommended. However, strong consideration should be given to converting the FullClo to a ParClo design (converting the exit loop ramps to direct ramps). Although not considered explicitly in the experimental design for this project, incorporating bypass lanes, such as for high-occupancy vehicles, is generally simpler with the non-loop ramp designs.

To assist with the assessment of on-ramp queue storage, a macroscopic queuing analysis tool was developed. This tool is open source and is available at https://github.com/swash17/RampMeterQueueing. This tool considers various traffic, control, and on-ramp roadway characteristics for a given ramp roadway-arterial intersection. It will provide estimates of queue length at every time step and the percentage of time that the advance queue override was activated. While a macroscopic queuing analysis cannot provide the same level of detail of on-ramp operations as a microscopic simulation tool, it can serve reasonably well as a 'first-cut' planning or preliminary engineering assessment.

Additional resources are also available on GitHub, including:

- Google Earth KMZ files for Florida ramp metering locations and a sample of ramp metering locations throughout the U.S. (https://github.com/swash17/FDOT_IRM).
- The SwashSim simulation project files for each of the interchange forms considered in this project (https://github.com/swash17/SwashSim/tree/master/Projects/Interchanges).
- Macro scripts/tools for processing the SwashSim output files to calculate the performance measures used in this project (https://github.com/swash17/SwashSim/tree/master/Utilities/PerformanceMeasureCalcs).

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1. INTRODUCTION

Interchanges, particularly those with ramp metering, represent a situation with competing objectives. From an operations perspective, the purpose of ramp metering is to restrict and/or regulate the flow of traffic onto the mainline in order to optimize mainline freeway flow. Such on-ramp restriction can often result in negative consequences for the connecting arterial roadway. For example, queue backup from the ramp meter can interfere with the operation of the arterial. Maximizing the efficiency of the arterial would generally preclude on-ramp restrictions. Some ramp metering algorithms are designed to "flush the queue" if it reaches an advance queue detector location. This is very undesirable from a freeway operations perspective, but is typically done as a compromise for the arterial operations.

Thus, it can be very difficult to achieve an operational condition in which both the arterial, at the interchange area, and the freeway, with ramp metering operational, are functioning at optimal levels for high traffic demands. This is even more commonly the case when ramp metering is installed at an interchange that was not originally designed with ramp metering in mind. Freeway interchanges that are designed without consideration for a future installation of ramp metering may encounter geometric and/or operational design challenges during a later design process to retrofit the interchange with ramp metering.

The majority of the existing research regarding ramp metering has largely been focused on ramp signal timing algorithms and/or the design of the on-ramp itself for accommodating ramp metering operations, irrespective of the particular overall interchange design. However, what is generally lacking is research that considers how ramp metering should factor into the selection of an overall interchange configuration.

The objective of this project was to develop guidance on interchange design, with consideration of ramp metering, that is appropriate for a planning and preliminary engineering level. In other words, guidance is provided to help understand the impact of ramp metering on interchange operations across a variety of interchange designs. Advantages and disadvantages of different designs, as well as guidance on how some specific geometric design elements might mitigate or exacerbate ramp-metering operations at the interchange is also provided. However, detailed operational questions (e.g., signal phasing sequences, signal coordination settings amongst the ramp terminal and adjacent intersections, the type of ramp-metering control algorithm, interactions between adjacent interchanges, and similar) are beyond the scope of the guidance developed from this project. Once a general interchange configuration and ramp metering strategy/algorithm are determined, detailed operational questions could be addressed through microscopic traffic simulation or comparable methods. Such issues/topics could also be the subject of future research. Specific topics addressed in this report include: ramp meter to freeway merge acceleration distance, multilane metering, performance measures, interchange configuration factors, and macroscopic queuing analysis. It is intended that the conclusions and recommendations developed from this study can be incorporated as guidance into the Interchange Access Request User's Guide (FDOT, 2018).

2. LITERATURE REVIEW

The literature is very sparse with respect to considerations for overall interchange design when ramp metering is installed. Thus, the review in this chapter largely covers ramp metering operational issues and on-ramp only geometric design issues.

Ramp-Metering Design Guides

MUTCD

Chapter 4I of the Manual on Uniform Traffic Control Devices (MUTCD) (FHWA, 2009) presents standards, guidance, options, and support provisions for application, design, and operation of ramp meters. However, an engineering study of traffic conditions and physical characteristics of the ramp shall be performed to determine whether installation of a ramp meter is justified. MUTCD references the FHWA's Ramp Management and Control Handbook (2006) as a support document for ramp metering.

FHWA

The Ramp Management and Control Handbook (FHWA, 2006) identifies ramp closure, ramp metering, special-use treatments, and ramp-terminal treatments as ramp-management strategies. Chapters 5, 6, 8, and 10 discuss ramp metering in detail. Chapter 5 provides operational (i.e., increase in throughput and average speed), safety (i.e., crash reduction around merge zone), and environmental (i.e., reduction in vehicle emissions) benefits of several freeway-to-freeway ramp-metering deployments. This chapter defines several operational factors associated with ramp metering including metering strategies, geographic extent, metering approaches, metering algorithms, queue management, and flow control. An effective ramp-metering strategy aims to optimize the freeway (generally increase in average speed) and the arterial (generally reduction in queue length and delay) based on a set of objectives. Ramp meters can be coordinated along a segment, corridor, or system, when traffic-related issues go beyond the extent of a single ramp. Ramp-metering control scheme can be local or system-wide, operating under pre-timed or traffic-responsive control methods (Table 2-1). Local metering is appropriate for isolated ramp meters, while system-wide metering is appropriate for coordinated ramp meters.

Metering at demand, also referred to as non-restrictive metering, is beneficial when ramp metering is first installed, and the motorists are not familiar with ramp metering. It may also be used at ramps, where traffic diversion is not acceptable, or there is not enough capacity to support restrictive metering. Non-restrictive metering has metering rates equal or greater than the ramp volume to increase safety on the freeway and to ensure there is no queue spillback onto the upstream arterial. Operator selection of metering rates is used to address special conditions such as incident or special events.

Table 2-1. Pre-timed metering vs. traffic-responsive metering

Pre-timed	**Traffic-responsive**
Simple hardware configuration	Complex hardware configuration
Pre-set metering rates based on historical conditions Fixed metering rates per time of day Activated based on pre-set schedules	Metering rates based on real-time traffic conditions
Does not need detection in the field or communication with a traffic management center (TMC)	Needs detection in the field
Able to operate in temporary lack of communication, detector malfunction, or construction	Needs communication with a TMC
Does not respond efficiently to non-recurring congestion	Responds efficiently to real-time traffic conditions
Requires frequent observations	Requires technical expertise for implementation
Higher operation costs	Higher capital and maintenance costs

Chapter 5 describes several metering algorithms including the Minnesota Department of Transportation (MnDOT) zone algorithm, the Washington State Department of Transportation (WSDOT) bottleneck algorithm, the WSDOT fuzzy-logic algorithm, the Denver helper algorithm, the Northern Virginia algorithm, and the SWARM algorithm. The Minnesota zone algorithm aims to group corridors to metering zones (typically, each zone has a free-flow upstream and a bottleneck downstream), and calculate metering rates as $[(B + X + S - A - \quad) \times \quad]/\quad$, where B is the downstream mainline volume, X is sum of the off-ramps volumes, S is the spare capacity, A is the upstream mainline volume, U is sum of the unmetered on-ramp volumes, D_n is the demand for the meter n, and D is the total demand. The Seattle bottleneck algorithm calculates both a local ramp-metering rate and a bottleneck metering rate and selects the more restrictive one. Metering rates are further adjusted for the ramp conditions and the downstream mainline volume.

The WSDOT fuzzy-logic algorithm was developed in response to the limitations of the bottleneck algorithm (i.e., it addresses the inherent issues with data accuracy and reliability in loop detectors, optimizes the mainline congestion and the ramp queues, does not require extensive system modeling, and is easy to tune using linguistic variables rather than numerical variables). This algorithm uses the mainline speed and occupancy and the ramp occupancy to calculate metering rates. Taylor et al. (1998) developed an algorithm to address the deficiencies of the WSDOT fuzzy-logic algorithm (i.e., it does not require congestion to develop before it can react, and it does not deal with the competing objectives). This algorithm has four rule groups including the mainline speed and occupancy, the downstream speed and occupancy, the ramp occupancy, and quality of the merge point, as well as six fuzzy classes including very small

(VS), small (S), medium (M), big (B), and very big (VB). Ramp meters are calculated based on the rule weight and the degree of activation of each rule outcome.

The Denver helper algorithm aims to group ramps to metering zones (each metering zone can include up to seven ramps) and is based on local traffic-responsive approach with centralized control. This algorithm deducts one metering rate from each ramp meter, if the downstream ramp meter is critical for three-consecutive 20-second periods (i.e., the ramp meter is operating at its most restrictive metering rate and the ramp occupancy exceeds the threshold value) and continues the process until all ramp meters are overridden.

The Northern Virginia algorithm aims to group freeway links to metering zones (each metering zone can include up to ten links) and calculates the metering rates as the difference between the predicted arrival demand (starting at the furthest upstream link) and the capacity of the link that contains a ramp (starting at the furthest downstream link). The system-wide area ramp metering (SWARM) algorithm compares the metering rates of two independent control algorithms SWARM1 and SWARM2 and selects the more restrictive one. SWARM algorithm is used for system-wide metering approach. SWARM1 is more complex, predicts future volumes based on the historical data, and restricts real-time volumes from exceeding the pre-determined saturation values, while SWARM2 is used for local traffic-responsive metering approach.

Since queue spillbacks on to the upstream arterial increase the delay and risk of rear-end crashes, it is not recommended to use ramp-metering algorithms that do not take ramp queues and storage capacity into account unless their metering rates are set at or above the ramp demand. Most ramp-metering algorithms either provide enough storage for worst-case queues or adjust the metering rates based on the detected queues. Queue detectors are placed on ramps upstream of the meter stop bar at critical locations. Ramp-metering algorithms either increase the metering rate at one level when queue is detected and increase it at a higher level when queue spillbacks or increase the metering rate sharply to more quickly reduce the queue length or adjust the metering rate downward to reduce the mainline congestion or use queues as an integral part of their algorithm that calculates the metering rate.

Chapter 6 presents a high-level screening matrix (Table 2-2) that recommends ramp-management strategies to address certain types of safety (at merge point, ramp terminal, and freeway mainline) and congestion-related problems (at ramp, arterial, ramp terminal, and freeway mainline), and offset certain neighborhood-related impacts and impacts that occur due to special events or construction-related activities. This chapter also proposes decision trees to help agencies select a ramp-metering strategy:

- Assess severity of ramp-metering impacts including diversion, equity, emissions on ramp, arterial impacts, public perception, and ramp geometry and spacing (i.e., closely spaced ramps, inadequate acceleration distance, sight distance, and merge/weave operations).
- Refine problem analysis including type and severity of crashes, extent and severity of mainline congestion, and neighborhood conditions.
- Analyze feasibility of ramp metering.
- Define geographic extent, local versus system-wide metering, and pre-timed versus traffic-responsive metering.

Table 2-2. High-level screening matrix

Need/Problem	Location/Reason	Ramp metering	Ramp closure	Special-use treatments	Ramp-terminal treatments
Safety	Merge point	✓	✓	✓	
	Ramp terminal		✓		✓
	Freeway	✓	✓		
Impacts	Neighborhood	✓	✓	✓	✓
	Construction	✓	✓	✓	✓
	Special events	✓	✓	✓	✓
Congestion	Freeway	✓	✓		
	Ramps		✓		✓
	Ramp terminal		✓		✓
	Arterial		✓		✓
Policy	Transit			✓	
	HOV			✓	
	Freight			✓	

Source: Table 6-1 of the FHWA Ramp Management and Control Handbook.

Chapter 8 discusses ramp-metering monitoring and operation. It is recommended to operate ramp meters at predictable times (i.e., peak periods), particularly when ramp metering is first installed, to get the staff experienced and the motorists familiar with the system. Chapter 10 provides planning and design standards for ramp metering that conform to the American Association of State Highway Officials (AASHTO) Green Book and the MUTCD. This chapter also includes specific design considerations for ramp metering that are mainly adopted from Caltrans, WSDOT, and MnDOT ramp-metering design guides that might be useful for those states that have not developed their own specific design standards and guidance for ramp management. FHWA follows the recommendations for Caltrans on flow control (i.e., appropriate number of metered lanes and release rates are based on the ramp volume), WSDOT on ramp design speed (i.e., design speed for a ramp is based on the design speed for the freeway mainline), and MnDOT on queue management (i.e., required queue storage distance is based on 10% of the pre-metered peak-hour ramp volume).

Arizona

The Arizona Department of Transportation (ADOT) Transportation Technology Group has prepared the ADOT Ramp-Metering Design Guide (2013) with detailed explanation of ramp-metering warrants, ramp-metering geometry, and ramp-metering hardware. ADOT warrants installation of a ramp meter, when both following conditions are satisfied. However, ADOT does not recommend ramp metering on freeway-to-freeway ramps.

- The freeway rightmost-lane peak-hour volume plus the ramp peak-hour volume is greater than 2050 veh/h, and the ramp peak-hour volume exceed 400 veh/h.
- Average speed on the freeway general purpose (GP) lane is less than 50 mi/h due to recurring congestion adjacent to or within two miles downstream of the ramp.

ADOT uses TxDOT guidelines on ramp metering, if the freeway rightmost-lane volumes are not available. ADOT ramp-metering geometric guidelines for the number of metered lanes, acceleration distance, queue storage distance, and stop-bar placement include:

- New interchanges shall be constructed to accommodate dual-lane ramp metering. Dual-lane metering should be considered for future installation of ramp metering on existing ramps. Ramp widening may be needed to provide the adequate queue storage distance. Fitting a dual-lane ramp meter on existing ramps may require the ramp stop bar to be placed further down to provide the adequate queue storage distance.
- The freeway speed limit or the freeway 85th-percentile speed may be used to determine the acceleration distance. Acceleration distance is increased by $G \times 65$ (where G is grade expressed as percentage), when grade exceeds 1% after the ramp stop bar and is increased by $(T - 3) \times 100$ (where T is percentage of HOVs), when percentage of HOVs exceeds 3% on the ramp.
- Queue storage distance is calculated as $14.5 \times Rate_{ram} - 12180$ for single-lane ramp meters and as $7.25 \times Rate_{ram} - 6090$ for dual-lane ramp meters. The minimum queue storage distance for both single-lane ramp meters and dual-lane ramp meters is 400 ft.
- Changes to pavement width, ramp length, pavement marking, or single-lane versus dual-lane ramp metering may be made to provide the recommended acceleration distance and queue storage distance.

ADOT defines four types of vehicle detectors:

1. Mainline detection: used for traffic-responsive ramp metering and traffic data collection, consists of two 6×6-ft loops per lane, placed close to the ramp-meter stop bar.
2. Advance queue detector detects queue spillback onto the arterial, consists of one 6×6-ft loop per lane, placed 0.36 times of the queue storage distance (has a minimum distance of 250 ft and a maximum distance of 900 ft) from the ramp-meter stop bar.
3. Demand/input detector detects presence of a vehicle at the ramp-meter stop bar, consists of two 6×6-ft loops per lane, placed 3 ft upstream of the ramp-meter stop bar.
4. Passage/output detector detects if a vehicle passes the stop bar, consists of one 6×6-ft loop per lane, placed 6 ft downstream of the ramp-meter stop bar.

California

The California Department of Transportation (Caltrans) Ramp-Metering Design Manual (2016) supplements the Highway Design Manual (HDM), California MUTCD, Caltrans Standard Plans, and Caltrans Standard Special Provisions for all geometric-related design standards that apply to ramp metering.

- A metered ramp has a practical volume range of 240-900 veh/h/ln for a typical one-vehicle-per-green operation. A minimum of one metered lane should be provided for every 900 veh/h to effectively control outside-of-range volumes (see Table 2-3).
- An HOV lane is warranted, when percentage of HOVs on the ramp exceeds 9% of the ramp peak-hour volume.

Table 2-3. Required number of metered lanes: Caltrans

Peak-hour volume (veh/h)	# Lanes
< 900	1
≥ 900 and ≤ 1800	2
> 1800	3

- For existing ramps, adequacy of queue storage distance can be evaluated using the existing peak 5-, 6-, or 15-minute arrival rates and the existing or anticipated metering rates. For new or reconstructed ramps, the minimum queue storage distance should be designed based on 7% of the design-year peak-hour volume, considering 29 ft for each queued vehicle. Ramp widening or lengthening, modifications to ramp-meter signal timing, and using system-wide traffic-responsive ramp-metering algorithms can be considered to balance out the demand across multiple ramps.
- Ramp metering deceleration distance is calculated using the HDM equations for stopping-sight distance, which are consistent with those of the AASHTO Green Book.
- A minimum of 300 ft acceleration distance should be provided.
- Regardless of the number of metered lanes, the ramp stop bar should be located a minimum of 75 ft upstream of the 23-ft separation point.

Caltrans mandates a metered HOV lane for freeway-to-freeway metering but recommends almost the same geometric-design standards as arterial-to-freeway metering. Direct HOV-to-HOV connectors may be constructed to minimize the weaving maneuvers. Caltrans has provided layouts for a typical dual-lane metered freeway loop on-ramp (1 GP lane and 1 HOV lane), a dual-lane metered successive freeway on-ramps (1 GP lane and 1 HOV lane), three-lane metered freeway diagonal on-ramp (2 GP lanes and 1 HOV lane), dual-lane metered connecter (2 GP lanes and 1 HOV lane), and three-lane metered connector (2 GP lanes and 1 HOV lane).

Caltrans defines four types of detectors installed on on-ramps and one type of detectors installed on off-ramps:

- Mainline detectors: consist of two 20×20-ft loops centered in each freeway mainline lane, placed upstream of the on-ramp gore nose opposite the ramp-meter stop bar. If a count detector is installed, the mainline detector should be placed close to the count detector.
- On-ramp queue detectors: consist of one loop per lane, placed at entrance of on-ramps.
- On-ramp demand detectors: consist of three loops per lane, placed upstream of the ramp-meter stop bar.
- On-ramp passage detectors: consist of one loop per lane, placed 7 ft downstream of the ramp-meter stop bar.
- Count detectors: shall be installed when entrance ramp passage detectors do not obtain traffic count data (i.e., speed, volume, and occupancy), placed 6 ft upstream of the merge point, placed downstream of the passage detector for single-lane entrance ramps and placed downstream of the lane-drop taper for multi-lane on-ramps.
- Off-ramp detectors: consist of one loop per lane, placed 23 ft downstream of the diverge point, or placed immediately downstream of the bifurcation point.

A metered connector may require additional queue detectors to be placed upstream and downstream (installed where sight distance is limit) of its entrance. Caltrans has provided typical detector layouts for freeway, 2-lane entrance ramp, 3-lane entrance ramp, exit ramp, and a metered connector, in addition to typical layouts of ramp-metering elements at an L-9 interchange and a full-cloverleaf interchange.

California uses the UP-ALINEA ramp metering algorithm, which estimates the downstream mainline occupancy based on the measured upstream mainline occupancy, measured ramp volume, measured upstream mainline volume, number of mainline lanes upstream of the ramp meter, and number of mainline lanes downstream of the ramp meter. UP-ALINEA is an extension to the ALINEA metering strategy which only uses mainline detectors upstream of the ramp meter to measure occupancy, volume, and speed.

Colorado

Atkins has prepared a technical memorandum (2015) on behalf of the Colorado Department of Transportation (CDOT) that recommends a three-tier approach for determining ramp-metering locations. Two of the tiers are derived from ADOT and Caltrans, and tier 3 is a descriptive warrant that recommends geometric and traffic evaluations.

- Warrant on ramp metering (derived from ADOT)
  - Sum of the peak-hour volumes on the 4-lane freeway and the ramp is less than 2650 veh/h.
  - Sum of the peak-hour volumes on the 6-lane freeway and the ramp is less than 4250 veh/h.
  - Sum of the peak-hour volumes on the 8-lane freeway and the ramp is less than 5850 veh/h.
- Number of metered lanes (derived from Caltrans)
  - Single lane when the freeway peak-hour volume is less than 900 veh/h.
  - Dual lane when the freeway peak-hour volume exceeds 900 veh/h.

Florida

The Florida Department of Transportation (FDOT) has classified its ramp-metering warrants into traffic, geometric, and safety warrants. FDOT warrants ramp metering, when one of the following conditions is satisfied (Zhu et al. 2011):

- Safety warrants
  - The freeway peak-hour volume exceeds 1200 veh/h/ln.
  - Average speed on the freeway is less than 50 mi/h.
  - The ramp peak-hour volume is 240-1200 veh/h for 1-lane ramps.
  - The ramp peak-hour volume is 400-1700 veh/h for multiple-lane ramps.
  - Sum of the peak-hour volumes on the 4-lane freeway and the ramp is less than 2650 veh/h.
  - Sum of the peak-hour volumes on the 6-lane freeway and the ramp is less than 4250 veh/h.
  - Sum of the peak-hour volumes on the 8-lane freeway and the ramp is less than 5850 veh/h.
  - Sum of the peak-hour volumes on the 10-lane freeway and the ramp is less than 7450 veh/h.

  - Sum of the peak-hour volumes on the 12-lane freeway and the ramp is less than 9050 veh/h.
  - The freeway rightmost-lane peak-hour volume exceeds 2050 veh/h.
- Geometric warrants
  - Queue storage distance is longer than $L = 0.82\ V - 0.00024342\ V^2$, where L is the required queue storage distance (ft) and V is the ramp peak-hour demand (veh/h).
  - Acceleration distance is longer than $L = 0.14\ V^2 + 3\ V + 9.21$, where L is the required safe merging distance (ft) and V is the freeway mainline prevailing speed (mi/h).
- Safety warrants (system-wide ramp metering)
  - $RHMVM$ along a segment exceeds 80 crashes per hundred million vehicle miles.
    $RHMVM = \frac{Annual\ number\ of\ crashes \times 100{,}000{,}000}{AADT \times 365 \times L}$,
    where L is segment length (mi) and AADT is average annual daily traffic (veh/day).

Georgia

The Georgia Department of Transportation (GDOT) ITS Design Manual (2008) does not consider ramp metering on freeway-to-freeway interchanges and collector-distributer (CD) roadways to not violate driver expectations. GDOT warrants ramp metering on all on-ramps with the following criteria.

- Volume to capacity ratio is greater than 0.88, and peak-hour volume exceeds 240 veh/h.
- Crash rate is greater than 2 crashes per million vehicles, and peak-hour volume exceeds 240 veh/h.

Ramp-meter stop bar should be placed upstream of the ramp gore to provide a safe acceleration distance and preserve the longest possible queue storage distance. GDOT defines three types of loop detectors:

- Queue loops: consist of a 6×6-ft loop, placed upstream of the ramp-meter stop bar.
- Presence loops: consist of a 6×40-ft loop, placed upstream of the ramp-meter stop bar.
- Passage loops: consist of a 6×6-ft loop, placed downstream of the ramp-meter stop bar.

Minnesota

Cambridge Systematics Inc. (2001) conducted a synthesis of existing research findings for the Minnesota Department of Transportation (MnDOT) to compare the ramp-metering evaluation findings of the Twin Cities with comparable metropolitan areas. They used selected measures of effectiveness such as average speed on freeway, crash rate, freeway occupancy, total travel time, freeway volume, fuel savings, benefit-cost ratio, ramp delays, arterial volume, overall travel demand, and public/motorist survey results. Based on the studies of the Texas Transportation Institute (TTI) and the Virginia Transportation Research Council, ramp metering is warranted, when the freeway mainline has one of the following conditions:

- Average speed of 30 mi/h or less during peak hours.
- Peak-hour volume between 1200 to 1500 veh/h/lane.
- High crash rates.
- Significant merging issues.

This guide considers three possible locations for ramp metering: arterial-to-freeway metering, freeway-to-freeway metering which is applicable when enough queue storage distance is available, and mainline metering which should only be used upstream of severe geometric bottlenecks, where widening is not feasible. Mainline metering can reduce travel times, help trucks avoid up-ramp stop-and-go movements, and improve emergency response times. MnDOT defines five types of ramp-metering controllers:

- Fixed-time controllers: easy to install and cheap to operate but not flexible to demand fluctuations.
- Traffic-responsive controllers: optimize metering rates based on demand variations, but do not respond efficiently to abrupt changes in demand.
- Central controllers: optimize system-wide performance but expensive and only suitable for locations with recurring congestion.
- Integrated controllers: can detect traffic conditions.
- Fuzzy-logic controllers: have short-range predictive capabilities and can handle imprecise data.

The University of Minnesota with assistance from MnDOT evaluated the effectiveness of three selected operational algorithms for coordinated ramp metering applied to a freeway section in the Twin Cities using a macroscopic simulation with real-world data. Incremental group coordination (Denver) resulted in consistently less restrictive metering. Explicit section-wide coordination (Twin Cities) produced more evenly distributed traffic patterns on the mainline. Fuzzy logic-based implicit coordination (Seattle) showed more flexibility in dealing with atypical demand patterns. If long queues continuously develop at the ramps, and further widening is not possible, MnDOT increases the metering rates, or turns off the ramp meter temporarily.

Nevada

The Nevada Department of Transportation (NDOT) (Jacobs Engineering Group Inc, 2013) has a systematic approach to identify whether there is a need for a ramp meter. NDOT warrants installation of a ramp meter on isolated ramps, when one of the individual warrants listed in Table 2-1 is satisfied.

Table 2-4. Nevada DOT isolated on-ramp metering warrants

#	Name	Condition(s)
1	Ramp volume	Ramp peak-hour volume exceeds 240 veh/h/lane
2	Safety	Crash rates within 500 ft upstream and downstream of the ramp gore point are greater than the mean crash rates for comparable sections of freeway
3	Operational (1-speed)	Average speed on the freeway is less than 50 mi/h for at least 30 minutes for 200 or more days per year
4	Operational (2-LOS)	The freeway LOS is D or worse during peak-hours
5	Volume (1)	Freeway volume and ramp volume): the peak-hour volume downstream of the ramp gore exceeds: o 4250 veh/h for three-lane freeway in each direction. o 5850 veh/h for four-lane freeway in each direction. o 7450 veh/h for five-lane freeway in each direction. o 9050 veh/h for six-lane freeway in each direction.

		○ 10650 veh/h for more than six-lane freeway in each direction.
6	Volume (2)	Sum of the peak-hour volumes on the freeway rightmost lane downstream of the ramp gore and the ramp is greater than 2100 veh/h.
7	Platoon	The ramp peak-hour volume (based on 30-second volume readings) is greater than 1100 veh/h.
8	Geometry (1)	Geometric improvements cannot provide the required acceleration distance
9	Geometry (2)	Geometric improvements cannot provide the required queue storage distance

The process for assessing ramp metering warrants for coordinated on-ramps is shown in Table 2-5.

Table 2-5. Nevada DOT coordinated on-ramp metering warrant assessment process

Step #	Warrant Condition(s) Check	Resulting Action
1	Warrant 1 satisfied?	No – Ramp meter is not warranted Yes – go to Step 2
2	Warrants 2, 3 or 4 satisfied?	No* – Ramp meter is not warranted Yes – go to Step 3
3	Warrants 5, 6 or 7** satisfied?	No – Ramp meter is not warranted Yes – go to Step 4
4	Warrant 8 satisfied?	No – Ramp meter is not warranted Yes – go to Step 5
5	Warrant 9 satisfied?	No – Ramp meter may or may not be warranted. NDOT consultation required. Yes – Ramp meter is warranted

* Check the warrants on individual ramps.
**Use engineering judgement, if only Warrant 7 is satisfied.

Source: Figure 2-1 of the NDOT Managed Lanes and Ramp Metering Part 2: Implementation Plan (Jacobs Engineering Group Inc., 2013)

NDOT uses TxDOT guidelines for selection of a ramp-metering approach. Ramp meters can operate either isolated from one another (local control) or coordinated with each other as part of a system (system-wide control). System-wide control is preferred to local control unless:

- Safety: crashes tend to be clustered at specific ramps.
- Congestion: there are limited bottlenecks, or there is a considerable separation between the bottlenecks.
- Management: there is no communication between the controller and a TMC.
- Equity: there is no desire to distribute the ramp queues among multiple locations.

Traffic-responsive ramp-metering algorithms are divided into two categories: open-loop and closed-loop. Open-loop algorithms are used for local control, while closed-loop algorithms are used for system-wide control. Examples of open-loop algorithms include speed control, demand capacity control, upstream occupancy control, and gap occupancy control. Examples of closed-loop algorithms include bottleneck algorithm and fuzzy-logic algorithm. Table 2-6 is based on the actual capacities observed on the existing metered ramps in Nevada, operating with only one-car-per-green flow control scheme and pre-timed metering.

Table 2-6. Ramp-metering capacities by number of lanes and cycle length: NDOT

Strategy	# Lanes	Cycle length (s)	Capacity (veh/h)
One vehicle per green	1	4-4.5	750-800
One vehicle per green	2	4-4.5	1500-1600
One vehicle per green	3	6-6.5	1500-1600

Error! Not a valid bookmark self-reference. provides guidelines for the required number of lanes to meter which is based on the ramp peak-hour volume (for the existing facilities) or the 20-year projected peak-hour ramp volume (for new facilities).

Table 2-7. Required number of metered lanes: NDOT

Ramp peak-hour volume (veh/h)	# Lanes
240-800	1
> 800	2

NDOT calculates the following variables sequentially to determine the required queue storage distance at a ramp-meter location.

- 20-year projected peak-hour ramp volume.
- 140-second arrival rates using a peak-hour factor of 0.8.
- Required number of metered lanes.
- Excess vehicle per 140-second cycle length: subtract the discharge rate (i.e., 31 for single lane and 62 for dual lane) from the arrival rate.
- Total queue length: multiply the excess vehicles by a vehicle spacing of 30 ft.
- Queue length per lane: divide the total queue length by the number of lanes.

The minimum acceleration distance for both single-lane and dual-lane ramp meters is 820 ft based on 45 mi/h ramp and 70 mi/h freeway (Jacobs Engineering Group Inc, 2013). The minimum queue storage distance is 480 ft for both single-lane ramp meter, 480 ft per lane for dual-lane ramp meter, and 510 ft per lane for three-lane ramp meter.

New York

The New York State Department of Transportation (NYSDOT) (1998) provides the minimum and the maximum peak-hour ramp volumes to determine the required number of metered lanes (Table 2-8).

Table 2-8. Required number of metered lanes: NYSDOT

Ramp peak-hour volume (veh/h)		# Lanes
Minimum	Maximum	
240	900	1
400	1500-1800	2

NYSDOT has adopted the ramp-metering warrants in the National Cooperative Highway Research Program (NCHRP) Report 155, Bus Use of Highways: Planning and Design Guidelines (i.e., ramp metering is warranted if the freeway operates at LOS D or worse, and adequate queue storage distance is available). However, NYSDOT does not recommend freeway-to-freeway ramp metering. Based on a report from the Connecticut Freeway Transportation Authority, NYSDOT recommends ramp metering if the available queue storage distance exceeds 10% of the ramp peak-hour volume.

Oregon

The Oregon Department of Transportation (ODOT) Traffic Signal Policy Guidelines (2017) warrants ramp metering if the ramp peak-hour volume is 240-900 veh/h for single-lane ramps and 900-1650 veh/h for dual-lane ramps.

Texas

The Texas Transportation Institute (TTI) developed special design considerations for ramp metering (2000) in cooperation with the FHWA and the Texas Department of Transportation (TxDOT). The three ramp-metering strategies described in this report are summarized in Table 2-9. One vehicle-per-green metering allows one vehicle per cycle (the minimum possible cycle length is four seconds with one second green, one second yellow, and two seconds red), multiple-vehicles-per-green metering, also known as platoon or bulk metering, allows two or more vehicles per cycle, and tandem metering, also known as two-abreast metering, allows two or more vehicles per cycle to enter the freeway. Although multiple-vehicles-per-green metering doubles the discharge rate per green indication, it does not double the throughput, since longer cycle lengths are required. Tandem metering and multiple-vehicles-per-green metering may be combined in locations with high traffic volumes. Tandem metering allows a constant headway between the vehicles on both lanes, since the two lanes never receive the same indication simultaneously. As indicated in Table 2-9, ramps can be designed with one lane, unless their demand exceeds 1200 veh/h.

Table 2-9. Ramp-metering strategies: TxDOT

Strategy	# Lanes	Cycle length (s)	Metering rate (veh/h)	Capacity (veh/h)
One vehicle per green	1	4-4.5	240-900	800-900
Multiple vehicles per green	1	6-6.5	240-1200	1100-1200
Tandem	2	-	400-1700	1600-1700

This report defines three design criteria for ramp metering, including minimum stopping sight distance to back of queue, queue storage distance, and distance from meter to merge.

- The minimum stopping sight distance from the centerline of the cross street to the back of the design queue should be 75 m. It is more desirable to use 100 m permitting two lane changes for right turns from the cross street and higher ramp approach speeds.
- The minimum queue storage distance for the three ramp metering strategies are provided in Figure 2-1. For single-lane ramp meters, $L = 0.25V - 0.00007422V^2$ ($V \leq 1600$ veh/h), where L is the minimum queue storage distance required (m) and V is the ramp volume.

- Distance from meter to merge (i.e., acceleration distance required to reach various freeway merging speeds) is defined for various classes of vehicles (Figure 2-2).

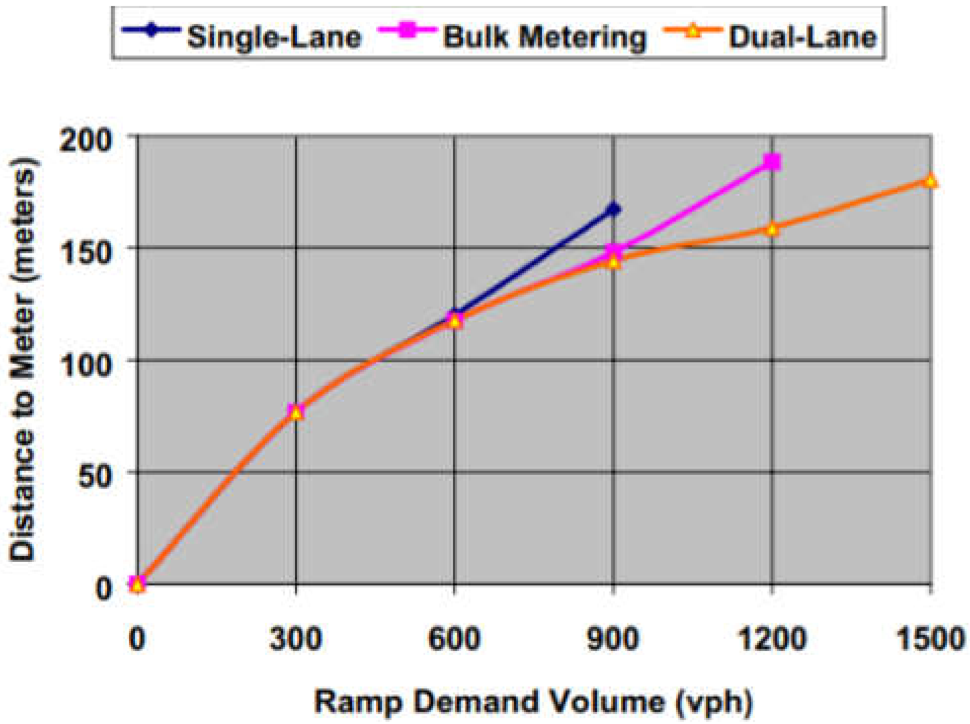


Figure 2-1. Minimum required queue storage distance
Source: TxDOT

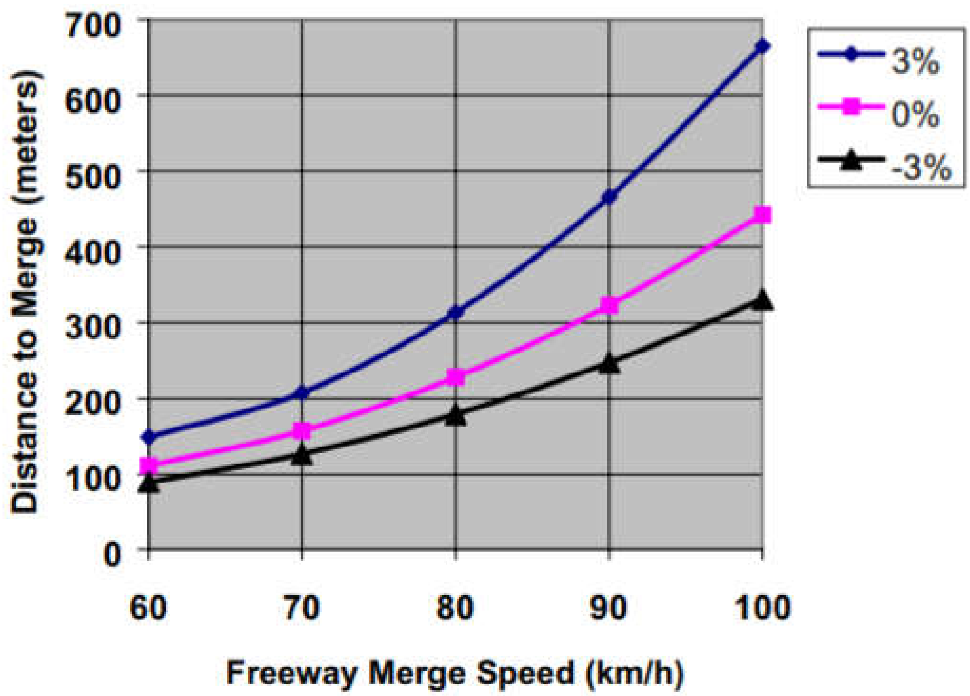


Figure 2-2. Minimum required acceleration distance
Source: TxDOT

Utah

The Utah Department of Transportation (UDOT) (2006) determines the required numbered of on-ramp lanes as a function of ramp volume and percentage of high-occupancy vehicles on the ramp (Table 2-10).

Table 2-10. Ramp lanes configuration: UDOT

Ramp type	Ramp volume (veh/h)	% HOV
1 SOV	600-900	<10
1 SOV/ 1 HOV	600-900	>10
2 SOV	900-1350	<10
2 SOV/1 HOV	900-1350	>10
3 SOV	1350-1720	<10
3 SOV/1 HOV	1350-1720	>10

Washington

Wilbur Smith Associates (2006) has provided an overview of lessons learned, best practices, implementation criteria, and effects on traffic patterns of selected ramp-metering implementations in Seattle with single-lane, dual-lane, HOV bypass, and multi-lane ramp configurations. Based on their research, single-lane ramp meters in Seattle operate as dual-lane ramp meters during peak hours allowing vehicles to use the ramp shoulder. The Washington State Department of Transportation (WSDOT) considers deployment of ramp metering when crash rates are high (no threshold specified), ramp demand exceeds 1500 veh/h/lane, and mainline loop detector occupancy exceeds 20%. WSDOT prefers corridor-wide ramp metering as opposed to site-specific ramp metering to encourage motorists use adjacent ramps as bypass for the metered ramps. Ramp meters in Seattle operate under system-wide traffic-responsive control and fuzzy-logic algorithm, activated per time of day (6 AM to 10 AM and 3 PM to 7 PM during weekdays), incidents, and special events.

WSDOT has prepared a set of formal guidelines to get the public involved throughout the planning, design, implementation, and operation stages of the ramp-metering projects. WSDOT educate drivers on use of ramp-metering systems as well as enforcement issues and improve driver acceptance through a provision of a comment and modification design period (FHWA Ramp Management and Control Handbook). For example, based on several public comments (WSDOT Blogspot), WSDOT turned off a ramp meter and converted the HOV lane to a second GP lane for a better ramp utilization.

Wisconsin

Chapter 3 of the Wisconsin Department of Transportation (WisDOT) Intelligent Transportation Systems Design Manual (2000) classifies ramp meters based on the number of metered lanes required and the usage of HOV lanes for priority treatment. An HOV lane is warranted when percentage of HOVs on the ramp exceeds 9% of the ramp peak-hour volume.

- Single-lane ramp meters (SOV): used at locations where the peak-hour design-year volume is less than 720 veh/h, and an HOV lanes is not feasible.
- Dual-lane ramp meters (SOV/HOV): used at locations where the peak-hour design-year volume is less than 720 veh/h, and an HOV lane is warranted.

- Dual-lane ramp meters (2 SOV): used at locations where the peak-hour design-year volume exceeds 720 veh/h, and an HOV lane is not feasible.
- Three-lane ramp meters (2 SOV/HOV): used at locations where the peak-hour design-year volume exceeds 720 veh/h, and an HOV lane is warranted. Dual-lane meters are preferred to three-lane meters due to safety considerations, unless considerable right-of-way is provided.
- Freeway-to-freeway meters: commonly designed with no HOV lane due to geometric constraints.

Geometric requirements for metered ramps depend on several factors including peak-hour volume (affecting the queue storage distance and the ramp width), percentage of heavy vehicles (affecting the ramp width), mainline design speed (affecting the acceleration distance), available right-of-way (affecting the ramp width and the ramp length), enforcement (determining whether an enforcement zone is desired or not), and construction funding (affecting the ramp width, the ramp length, the acceleration distance, and the priority treatment). Metered ramps must provide storage for a minimum of 10% of the existing or anticipated peak-hour volume. Table 2-11 provides recommended and minimum widths for metered ramp lanes based on the ramp-meter configuration. WisDOT has provided typical layouts for single-lane slip ramps, single-lane loop ramps, dual-lane slip ramps, dual-lane loop ramps, three-lane slip ramps (non-separated HOV), and three-lane slip ramps (separated HOV).

Table 2-11. Ramp-meter lane width requirements: WisDOT

Ramp type	**Ramp lane width (ft)**	
	Recommended	Minimum
SOV	12	12
SOV/HOV	28	24
2 SOV	24	24
2 SOV/HOV	40	36
HOV	16	12

WisDOT has implemented two types of detector stations at large scale:

- Loop detector stations: used for permanent detection statewide.
- Microwave detector stations: above-ground units mounted either over a traffic lane (overhead configuration) or along the side of the freeway (side-fire configuration), used for permanent detection when the freeway pavement is relatively new, and used for temporary detection during construction projects. Microwave detectors in side-fire configuration (which calculate the speed) have proven to be more accurate than those in overhead configuration (which measure the speed).

WisDOT defines several types of loop detectors:

- Demand loops: consist of a 6×20-ft loop per lane (in District 2) or two 6×8-ft loops per lane (in District 1), placed 25 ft upstream of the ramp-meter stop bar.
- Passage loops: consist of a 6×6-ft loop, placed 10 ft downstream of the ramp-meter stop bar.
- Queue loops: 6-ft long and sized to fit the ramp width, placed upstream of the ramp-meter stop bar.

- Entrance-ramp reporting loops: should fit the entrance ramp such that no vehicle can avoid passing over them, placed immediately downstream of the ramp-meter stop bar.
- Exit-ramp reporting loops: should fit the off-ramp such that no vehicle can avoid passing over them.
- Turing-count reporting loops: typically installed on ramps with an island.
- Mainline loops: placed upstream of the ramp gore.

Research Papers/Projects

Chang and Li (2002) proposed a coordinated, system-wide, traffic-responsive ramp-metering control model with dynamic OD estimation to determine metering rates for multiple coordinated interchanges within a corridor. Based on the Payne's high-order continuum traffic flow model, a linear quadratic optimal control problem is constructed with an objective to minimize the difference square between the actual flow rate and the nominal flow rate during peak hours. This dynamic model considers flow rate, density, speed, and the driver reaction time. A closed-loop feedback control is used to increase the system robustness to the demand uncertainties, and the Kalman filter method for real-time OD estimation of on-ramp trips is used to increase the system accuracy. The proposed iterative algorithm requires the historical upstream flow rates and the real-time detected flow rates on each segment at the initial time step to reach a global optimum flow rate on the mainline. This study does not consider availability of adequate queue storage distance. Two scenarios including no on-ramp flow control and fixed OD are simulated to study the necessities of ramp metering during peak hours and the dynamic OD estimation.

TTI in collaboration with TxDOT and FHWA (Bonneson et al., 2003) suggest that on-ramps with meters provide acceleration distances that are 1.3 times as long as those recommended in the 2011 AASHTO Green Book (see Table 2-12).

Table 2-12. Recommended acceleration distances for ramp meters: TxDOT

Arterial/freeway design speed (mi/h)	**Acceleration distance (ft)**							
	On-ramp design speed (mi/h)							
	15	20	25	30	35	40	45	50
30	182	-	-	-	-	-	-	-
35	286	208	-	-	-	-	-	-
40	390	351	273	156	-	-	-	-
45	637	572	494	364	208	-	-	-
50	858	793	715	585	455	169	-	-
55	1170	1053	104	871	715	416	195	-
60	1482	1430	1326	1183	1040	715	546	234
65	1755	1703	1586	1456	1300	1001	780	481
70	2028	1976	1846	1755	1599	1300	1066	754
75	2249	2119	2054	1963	1846	1508	1352	1014

Source: Table 2-10 values from Bonneson et al. (2003) times 1.3

Table 2-13 provides a summary of the desirable and the retrofit queue storage distance of several DOTs as well as the recommended queue storage distance for ramp meters on non-frontage-road facilities. Colorado DOT recommends putting the stop bar as close as possible to the freeway gore to maximize the queue storage distance. Stop bar to freeway gore is not directly comparable with acceleration distance, since some acceleration occurs in the speed-change lane associated with the ramp entrance.

Table 2-13. Recommended queue storage distances for ramp meters

	Queue storage (ft)		**Stop bar to freeway gore (ft)**	
State DOT	Retrofit	Desirable	Retrofit	Desirable
Arizona	76	1010	340-450	1400
California	-	1000*	-	660
Illinois	-	-	300	300
Michigan**	-	-	250	250
Minnesota***	300	500	250-300	400
Oregon	-	-	250	250
Virginia	400	1400	300-350	300-350
Washington	500	1000	-	700

* Used for three-lane ramps and ramps with peak-hour volume exceeding 1500 veh/h.
** Metered ramps are on a downgrade approach.
*** Stop bar to freeway gore should be 500-600 ft, if an HOV lane is provided.

Tian (2007) defined an interchange as a system of integrated components (i.e., arterial, freeway, ramp, and frontage road), and determined traffic-responsive ramp-metering rates, stochastic freeway mainline capacity, and ramp queues of a metered diamond interchange. He considered two types of signal phasing schemes developed for operation of a diamond interchange including the three-phase and the TTI four-phase. The three-phase signal phasing scheme is suitable when there is an enough space between the two signals to store the arterial left-turn queues, and the TTI four-phase is suitable when there is a limited space between the two signals. Tian et al. (2002, 2004) considered the effect of ramp queue spillbacks on operation of a diamond interchange. They evaluated the performance of a metered diamond interchange for various queue spillback scenarios and ramp queue distributions using performance measures such as queue length, number of stops, and delay. They also showed traffic-responsive ramp-metering strategy results in more efficient traffic operation than fixed-time ramp metering.

Tian (2007) proposed additional detection, communication, and signal control devices for a metered diamond interchange to effectively detect the ramp queues and optimize the mainline traffic. One intermediate queue detector and one boundary queue detector are proposed to be located on each external approach of the arterial and on each off-ramp (i.e., four intermediate detectors and four boundary detectors), and one queue spillback or interference detector is proposed to be located on each frontage road (two queue spillback detectors). The intermediate queue detectors are used to make some minor adjustments to the phase splits based on the available queue storage space. The signal remains in normal operation, unless a boundary queue detector detects a vehicle. If interference detectors detect queue spillbacks on the frontage road, the signal switches to candidate phases based on the type of phasing scheme and the queue length and holds these phases until the on-ramp queues are dissipated. The candidate phases to be held are associated with those movements (i.e., the through movement on the frontage road and left-turn movement on the arterial) that feed the on-ramps.

Kotsialos et al. (2004) applied an advanced motorway optimal control (AMOC) strategy (with two competing objectives of efficiency and equity) for optimized operation of ramp meters under any combination of network topology, demand, and operational constraints considering available ramp queue storage distance. Alyousef et al. (2013) evaluated effectiveness of wireless sensor networks (which collect traffic-related data and send them to a traffic control center for further processing) for coordinated ramp metering on stack, cloverleaf, and turbine interchanges. Wang (2013) provided the maximum storage needs and corresponding peak-hour demands for a sample of metered on-ramps mainly with hook, loop, diagonal, and connector configurations located in Los Angeles. Pham et al. (2013) evaluated performance of a traffic-responsive fuzzy-logic control algorithm for a metered diamond interchange in Auckland, New Zealand under different traffic conditions. The interchange was equipped with a queue detector (placed 70 m upstream of the ramp-meter stop bar), check-in detector (placed 413 m upstream of the ramp-meter stop bar), upstream mainline detector (placed 160 m upstream of the on-ramp gore), downstream mainline detector (placed 200 m downstream of the on-ramp gore), and queue occupancy detector (placed 40 m upstream of the ramp-meter stop bar) for each of the nine movements. Li et al. (2017) developed a stochastic model for estimation of off-ramp and arterial queue lengths at single-lane and dual-lane roundabout interchanges under various traffic conditions and considering different interchange ramp spacings and percentage of heavy vehicles on off-ramps. The entry capacities were calibrated for different types of passenger cars and heavy vehicles with different critical and follow-up headways. The results show that sum of the entrance and circulating demands impacts off-ramp and arterial queue lengths; interchange ramp spacing does not have a significant impact on off-ramp or arterial queue lengths; and percentage of heavy vehicles on off-ramps has impact on off-ramp queue length.

Hadi et al. (2017) have compared the minimum, maximum, and average acceleration distance requirements of AASHTO Green Book (2011), conservative design (Tian et al., 2016), aggressive design (Tian et al., 2016), and NCHRP Report 505 (Harwood et al., 2003) for passenger cars and trucks (Table 2-14). Based on the conservative design (i.e., trucks require 60 percent more acceleration distance comparing to passenger cars), average acceleration distances in Miami and Seattle are not adequate for trucks.

Table 2-14. Recommended acceleration distances in selected cities

	Acceleration distance (ft)			**Acceleration distance for passenger cars (ft)**			**Acceleration distance for trucks (ft)**		
Cities	Min	Avg	Max	AASHTO	Conserv-ative design*	Aggress-ive design*	NCHRP Report 505	Conserv-ative design*	Aggress-ive design*
LA	585	670	720	1200	1080	780	2000	1728	1248
Denver	575	690	785	1200	1080	780	2000	1728	1248
Minne-apolis	425	523	635	1200	1080	780	2000	1728	1248
Atlanta	500	581	725	1200	1080	780	2000	2030	1248
Seattle	800	938	1050	1410	1269	917	2490	2030	1466
Miami	850	964	1025	1200	1080	780	2000	2030	1248

Source: Table 7-4 of FDOT Project BDV29-977-25.
* Tian, 2016

Summary

The existing ramp-metering design manuals (Table 2-15) mainly focus on operational, safety, and environmental benefits of ramp-metering deployments, metering strategies, geographic extents, metering approaches, metering algorithms, queue management, and flow control. The majority of the existing ramp-metering design manuals have specific ramp-metering warrants (either developed by themselves or adopted from other states) that are categorized into traffic criteria, geometric criteria, and safety criteria (Table 2-16).

Table 2-15. States with specific guidance on ramp-metering design

State	Ramp-metering design guide
Arizona	Ramp Metering Design Guide (2013)
California	Ramp Metering Design Manual (2016) Highway Design Manual (2018)
Georgia	Intelligent Transportation Systems Design Manual (2008)
Nevada	Managed Lanes and Ramp Metering Manual (2013)
New York	Highway Design Manual (1998)
Oregon	Traffic Signal Policy and Guidelines (2017)
Texas	Ramp Metering Algorithms and Approaches for Texas (2004)
Utah	Active Traffic Management System Design Manual (2006)
Wisconsin	Intelligent Transportation Systems Design Manual (2000)

Table 2-16. Ramp metering warrants

Warrant category	Criteria	State
Traffic	Mainline volume	Florida, Minnesota, Texas, Washington
	Ramp volume	Arizona, Colorado, Florida, Nevada, New York, Oregon
	Mainline and ramp volume	Arizona, Florida, Colorado, Nevada,
	v/c ratio	Georgia, Wisconsin
	Mainline speed	Arizona, Florida, Minnesota, Nevada
	Level of service	Nevada, New York
	Occupancy/density	Washington
Geometric	Acceleration distance	Florida, Nevada
	Queue storage distance	Florida, Nevada, New York
Safety	Crash rate	Florida, Georgia, Minnesota, Nevada
	Merging issue	Minnesota

Source: Adopted from Table 3-3 of the FDOT Project BDV29-977-25.

Some of the states have specific design considerations for ramp metering (Table 2-17), and some have adopted their ramp-metering design guidelines from the national ramp-metering design standards (Table 2-18).

Table 2-17. States with specific geometric design considerations for ramp metering

Geometric design consideration	State
Number of metered lanes	Arizona, California, Nevada, New York, Oregon, Texas, Utah, Wisconsin
Ramp HOV treatment	California, Utah, Wisconsin
Metering rate	Arizona, Nevada, Texas
Acceleration distance	Arizona, California, Nevada, Texas
Deceleration distance	California, North Carolina, Texas
Queue storage distance	Arizona, California, Nevada, Texas, Wisconsin
Stop-bar placement	Arizona, California, Georgia, Nevada
Vehicle detection	Arizona, California, Georgia, Nevada, Wisconsin

Table 2-18. Ramp-metering design guidance

Ramp-metering design standards	Guidance
AASHTO Green Books	Acceleration distance (adopted by Caltrans and GDOT) Queue storage distance (adopted by Texas)
FHWA Ramp Management and Control Handbook	Number of metered lanes (adopted from Caltrans) Ramp design speed (adopted from WSDOT) Queue storage distance (adopted from MnDOT)
Manual on Uniform Traffic Control Devices	Ramp signage

Most of the existing research has largely been focused on ramp signal-timing algorithms and/or the geometric design of the on-ramp itself, irrespective of the interchange design. ADOT has considered ramp metering (queue storage distance and advance queue detector length) as a function of interchange design for single point urban interchange (SPUI), and Caltrans has provided layouts for a typical dual-lane metered freeway loop on-ramp, a dual-lane metered successive freeway on-ramps, three-lane metered freeway diagonal on-ramp, dual-lane metered connecter, and three-lane metered connector.

3. TRUCK ACCELERATION DISTANCES

AASHTO References for Design of Highway and Street

Most of the State DOTs, including Florida, use AASHTO guidelines for design of acceleration lanes. Even though trucks require much longer distance to accelerate than passenger cars, AASHTO bases its recommended acceleration lengths on passenger cars, since it assumes that slower entries are unavoidable and are publicly accepted. AASHTO suggests increasing the acceleration lengths or, if feasible, locating the entries on downgrades, if there are substantial number of heavy vehicles on the ramp. Despite numerous updates and revisions, the 2018 AASHTO Green Book still recommends the same acceleration lengths as the 1994 AASHTO Green Book.

The 1965 AASHTO Blue Book includes taper lengths as part of the acceleration lengths (Table 3-1), while other editions consider length of acceleration lane not including taper (i.e., acceleration length is measured from point of tangency of the last ramp curve to the point where the ramp lane width becomes less than 12 ft). Nevertheless, the 1973 AASHTO Red Book recommends the same acceleration lengths as the 1965 AASHTO Blue Book. The 1965 AASHTO Blue Book determined the acceleration lengths for passenger cars from studies conducted in the late 1930's. The uniform acceleration formula in the 1965 AASHTO Blue Book (Eq. 3-1) is based on the simple acceleration-velocity-distance kinematics relationship assuming constant acceleration rates between each of two speeds and decreasing acceleration rates as initial speeds increase.

$$X = \frac{(1.47V_m)^2 - (1.47V_i)^2}{2a} \tag{3-1}$$

where

X	Acceleration length (ft),
V_m	Merging speed (mi/h),
V_i	Initial speed (mi/h),
a	Acceleration rate (ft/s^2), and
1.47	Constant used to convert velocity units of mi/h to ft/s (calculated from 5280 ft/s divided by 3600 s/h)

Note that the highway design speeds and on-ramp design speeds are excluded for all the modified tables in this report, since they did not have any contribution to the calculation of acceleration lengths.

Table 3-1. Minimum acceleration lengths (ft) on 2% grade or less

Merging speed (mi/h)	**Initial speed (mi/h)**								
	0	14	18	22	26	30	36	40	44
23	190	-	-	-	-	-	-	-	-
31	380	320	250	220	140	-	-	-	-
39	760	700	630	580	500	380	160	-	-
47	1170	1120	1070	1000	910	800	590	400	170
53	1590	1540	1500	1410	1330	1230	1010	830	580

Modified version of what was originally in the 1954 AASHTO Blue Book.

The 2018 AASHTO Green Book provides the minimum acceleration lengths required for passenger cars on 2% grade or less (Table 3-2). These values are used as a base to produce the minimum acceleration lengths required for passenger cars on 3% to 6% upgrades and −6% to −3% downgrades with increment of 1%. The values in Table 3-7 through Table 3-10 are derived from multiplying the provided adjustment factors (Table 3-3-Table 3-6) by the recommended acceleration lengths in Table 3-2. A uniform 50:1 to 70:1 taper is recommended for tapered on-ramps with lengths of 1300 ft or longer. Unlike downgrades, the provided speed-change lane adjustment factors for upgrades are based upon the initial speed.

Table 3-2. Minimum acceleration lengths (ft) on 2% grade or less (2018 AASHTO Green Book)

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	180	140	-	-	-	-	-	-	-
27	280	220	160	-	-	-	-	-	-
31	360	300	270	210	120	-	-	-	-
35	560	490	440	380	280	160	-	-	-
39	720	660	610	550	450	350	130	-	-
43	960	900	810	780	670	550	320	150	-
47	1200	1140	1100	1020	910	800	550	420	180
50	1410	1350	1310	1220	1120	1000	770	600	370
53	1620	1560	1520	1420	1350	1230	1000	820	580
55	1790	1730	1630	1580	1510	1420	1160	1040	780

Table 3-3. Speed change lane adjustment factors for −6% to −5% grade (2018 AASHTO Green Book)

Merging speed (mi/h)	All initial speeds
31	0.6
35	0.575
39	0.55
43	0.525
47	0.5
50	0.5
53	0.5

Table 3-4. Speed change lane adjustment factors for −4% to −3% grade (2018 AASHTO Green Book)

Merging speed (mi/h)	All initial speeds
31	0.7
35	0.675
39	0.65
43	0.625
47	0.6
50	0.6
53	0.6

Table 3-5. Speed change lane adjustment factors for 3% to 4% grade (2018 AASHTO Green Book)

Merging speed (mi/h)	Initial speed (mi/h)			
	20	30	40	50
31	1.3	1.3	-	-
35	1.3	1.35	-	-
39	1.3	1.4	1.4	-
43	1.35	1.45	1.45	-
47	1.4	1.5	1.5	1.6
50	1.45	1.55	1.6	1.7
53	1.5	1.6	1.7	1.8

Table 3-6. Speed change lane adjustment factors for 5% to 6% grade (2018 AASHTO Green Book)

Merging speed (mi/h)	Initial speed (mi/h)			
	20	30	40	50
31	1.5	1.5	-	-
35	1.5	1.6	-	-
39	1.5	1.7	1.9	-
43	1.6	1.8	2.05	-
47	1.7	1.9	2.2	2.5
50	1.85	2.05	2.4	2.75
53	2.0	2.2	2.6	3.0

Table 3-7. Minimum acceleration lengths (ft) on −6% to −5% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
31	216	180	162	126	72	-	-	-	-
35	322	282	253	219	161	92	-	-	-
39	396	363	336	303	248	193	72	-	-
43	504	473	425	410	352	289	168	79	-
47	600	570	550	510	455	400	275	210	90
50	705	675	655	610	560	500	385	300	185
53	810	780	760	710	675	615	500	410	290

Values are calculated from the information in the AASHTO 2018 Green Book.

Table 3-8. Minimum acceleration lengths (ft) on −4% to −3% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
31	252	210	189	147	84	-	-	-	-
35	378	331	297	257	189	108	-	-	-
39	468	429	397	358	293	228	85	-	-
43	600	563	506	488	419	344	200	94	-
47	720	684	660	612	546	480	330	252	108
50	846	810	786	732	672	600	462	360	222
53	972	936	912	852	810	738	600	492	348

Values are calculated from the information in the 2018 AASHTO Green Book.

Table 3-9. Minimum acceleration lengths (ft) on 3% to 4% grade

Merging speed (mi/h)	Initial speed (mi/h) 20	30	40	50
31	351	156	-	-
35	572	378	-	-
39	793	630	182	-
43	1094	972	464	-
47	1540	1365	825	288
50	1900	1736	1232	629
53	2280	2160	1700	1044

Values are calculated from the information in the 2018 AASHTO Green Book.

Table 3-10. Minimum acceleration lengths (ft) on 5% to 6% grade

Merging speed (mi/h)	Initial speed (mi/h) 20	30	40	50
31	405	180	-	-
35	660	448	-	-
39	915	765	247	-
43	1296	1206	656	-
47	1870	1729	1210	450
50	2424	2296	1848	1018
53	3040	2970	2600	1740

Values are calculated from the information in the 2018 AASHTO Green Book.

The AASHTO Green Book suggests high-speed on-ramps be located on downgrades, and longer acceleration lengths be provided for upgrade on-ramps.

Deen

Deen (1957) studied 55 loaded semi-trailer trucks with a single rear axle and 39 loaded semi-trailer trucks with tandem rear axles, driving on a 1600-ft long on-ramp acceleration lane on approximately level grade (1400 ft on 0% grade and 200 ft on 0.4% grade). Since the studied single rear-axle and tandem rear-axle semi-trailers showed approximately identical acceleration capabilities, Deen combined both into a single semi-trailer truck category. As all the existing design standards at his time were developed using the mean acceleration rate of passenger cars, he felt the same approach should be used for heavy commercial vehicles. Using distance vs. speed plots, Deen calculated the required acceleration length for semi-trailers accelerating at an average acceleration rate to reach various highway design speeds starting from a stopped position. Deen further used these graphs to calculate the distance traveled between each two desired speeds (Table 3-11).

Table 3-11. Minimum acceleration lengths (ft) for semi-trailer trucks on level grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	5	10	14	18	22	26	30	34
22	290	275	240	190	110	-	-	-	-
29	700	685	650	600	520	410	210	-	-
35	1240	1225	1190	1140	1160	950	750	460	100
40	1820	1805	1770	1720	1640	1530	1330	1040	680

Assuming trucks drive 5 mi/h slower than passenger cars, Deen found that the recommended acceleration lengths in the AASHTO 1954 Blue Book (Table 3-2) are adequate for:

- single-unit trucks for all merging speeds of 35 mi/h or less, and
- semi-trailer trucks for all merging speeds of 22 mi/h or less.

Based on Deen's study, the recommended acceleration lengths in the AASHTO 1954 Blue Book are not adequate for trucks accelerating to the modern freeway design speeds.

NCHRP Report 505

Harwood et al. (2003) developed a truck speed profile model (TSPM) implemented in a Microsoft Excel spreadsheet to anticipate when an added climbing lane might be warranted. This design tool computes the expected speed of a truck at each location of a site of interest. Truck characteristics inputs needed are:

- desired speed,
- initial speed,
- weight-to-power ratio, and
- weight-to-frontal area ratio)

Roadway characteristics inputs needed are:

- vertical profile, and
- elevation above sea level

Whenever the computed speed drops 10 mi/h below the speed in advance of the upgrade, then a climbing lane is warranted. Table 3-12 and Table 3-13 provide the maximum weight-to-power ratios of a truck to be able to achieve the given condition (i.e., accelerating from the initial speed to the merging speed over the specified distance) in Table 3-2.

Table 3-12. Maximum weight-to-power ratios (lb/hp) capable of reaching the given conditions in Table 3-2 on level grade

Merging speed*	**Initial speed (mi/h)**								
(mi/h)	0	14	18	22	26	30	36	40	44
23	105	140	-	-	-	-	-	-	-
27	110	120	130	-	-	-	-	-	-
31	105	115	120	125	120	-	-	-	-
35	120	120	125	135	135	135	-	-	-
39	120	120	120	120	120	125	145	-	-
43	120	120	115	120	120	120	120	130	-
47	110	115	115	115	115	115	150	120	130
50	110	110	110	110	115	115	110	110	110
53	105	105	105	105	105	105	105	105	105
55	105	105	100	100	105	105	105	105	100

* Average speed on the ramp is assumed to be 5 mi/h less than the average speed of the through traffic.
Source: Modified version of Table 64 in the NCHRP Report 505.

Table 3-13. Maximum weight-to-power ratios (lb/hp) capable of reaching the given conditions in Table 3-2 on 2% grade

Merging speed	**Initial speed (mi/h)**								
(mi/h)	0	14	18	22	26	30	36	40	44
23	65	110	-	-	-	-	-	-	-
27	65	100	100	-	--	-	-	-	-
31	80	85	95	95	100	-	-	-	-
35	90	95	95	100	100	100	-	-	-
39	90	90	90	95	95	95	110	-	-
43	90	90	85	90	90	90	90	100	-
47	85	85	85	85	85	85	85	85	90
50	85	85	80	80	80	80	80	80	80
53	80	80	80	80	80	80	80	80	75
55	80	80	80	80	80	80	80	80	75

Source: Modified version of Table 65 in the NCHRP Report 505.

The recommended acceleration lengths in Table 3-14 are calculated from the following equations. These equations and related calculations are covered in detail St. John and Harwood (1986). The recommended acceleration lengths in Table 3-14 are, on average, 1.8 times the recommended acceleration lengths specified in Table 3-2.

$$a_c = -0.2445 - 0.0004\max(10, V_t) - \frac{0.021C_{de}\,[\max(10, V_t)]^2}{\frac{W}{A}} - \frac{222.6C_{pe}}{\frac{W}{NHP}\max(10, V_t)} - gG \tag{3-2}$$

$$a = \frac{a_c + \frac{15368C_{pe}}{\frac{W}{NHP}\max(10, V_t)}}{1 + \frac{14080}{\frac{W}{NHP}}[\max(10, V_t)]^2} \tag{3-3}$$

$$a_e = \begin{cases} \frac{10a}{10 + 1.5\frac{a}{|a|}(a - a_c)}, & V_t < 10 \\ \frac{0.4a\ V_t}{0.4V_t + 1.5\frac{a}{|a|}(a - a_c)}, & V_t \geq 10 \end{cases} \tag{3-4}$$

$$V_{pl} = V_t + a_e \tag{3-5}$$

$$V_{dl} = \begin{cases} V_d, & |V_d - V_t| < 1.2 \\ \min\,[1.2 + 0.108(V_d - V_t) + V_t, V_d], & V_d - V_t \geq 1.2 \\ V_d - 1.2, & V_d - V_t \leq -1.2 \end{cases} \tag{3-6}$$

$$V_{t+1} = \min\,(V_{pl}, V_{dl}) \tag{3-7}$$

$$X_{t+1} = X_t + V_t + 0.5(V_{t+1} - V_t) \tag{3-8}$$

where

X_t	Location at time *t* (ft),
V_d	Desired speed (ft/s),
V_t	Speed at time *t* (ft/s),
V_{pl}	Performance-limited speed (ft/s),
V_{dl}	Driver-limited speed (ft/s),
a_c	Coasting acceleration (ft/s^2),
a_p	Power acceleration (ft/s^2),
a_e	Effective acceleration (ft/s^2),
C_{pe}	Altitude correction factor for elevation, (considered as [1-0.00004×local elevation (ft)] for gasoline engines)
C_{de}	Horsepower correction factor for elevation, (considered as [1-0.00000688×local elevation (ft)]$^{4.255}$ for gasoline engines)
W	Gross vehicle weight (lb),
NHP	Net horsepower,
A	Projected vehicle frontal area (ft^2),
g	Acceleration of gravity (=32.2 ft/s^2), and
G	Local grade (decimal)

Table 3-14. Minimum acceleration lengths (ft) for 180 lb/hp trucks on level grade

Merging speed	Initial speed (mi/h)								
(mi/h)	0	14	18	22	26	30	36	40	44
23	275	160	-	-	-	-	-	-	-
27	400	300	230	-	-	-	-	-	-
31	590	475	400	310	170	-	-	-	-
35	800	700	630	540	400	240	-	-	-
39	1100	1020	950	850	720	560	200	-	-
43	1510	1400	1330	1230	1100	920	580	240	-
47	2000	1900	1830	1740	1600	1430	1070	760	330
50	2490	2380	2280	2230	2090	1920	1560	1220	800
53	3060	2960	2900	2800	2670	2510	2140	1810	1260
55	3520	3430	3360	3260	3130	2960	2590	2290	1850

Source: Table 66 of the NCHRP Report 505.

Fitzpatrick and Zimmerman

Fitzpatrick and Zimmerman (2007) decided to propose potential updates to the acceleration lengths recommended in the 2004 AASHTO Green Book. Since it was unclear how the acceleration lengths were calculated in the 2004 AASHTO Green Book, they assumed that the same procedure was used as in the 1965 AASHTO Blue Book (Eq. 3-1). Table 3-15 provides the acceleration rates used to reproduce the acceleration lengths in the 2004 AASHTO Green Book.

Table 3-15. Acceleration rates (ft/s^2) used to reproduce the acceleration lengths in Table 3-2

Merging speed	Initial speed (mi/h)								
(mi/h)	0	14	18	22	26	30	36	40	44
23	3.18	2.58	-	-	-	-	-	-	-
27	3.03	2.64	2.70	-	-	-	-	-	-
31	2.88	2.70	2.55	2.45	2.40	-	-	-	-
35	2.58	2.44	2.34	2.25	2.20	2.20	-	-	-
39	2.28	2.17	2.13	2.04	2.00	1.91	-	-	-
43	2.14	2.04	1.99	1.93	1.91	1.84	1.80	-	-
47	1.99	1.90	1.85	1.82	1.82	1.77	1.70	1.64	-
50	1.95	1.87	1.86	1.82	1.79	1.76	1.70	1.65	1.60
53	1.90	1.83	1.86	1.81	1.76	1.74	1.70	1.66	1.65
55	1.90	1.83	1.86	1.81	1.76	1.74	1.70	1.68	1.65

Fitzpatrick and Zimmerman believed that vehicles enter limited-access roads at speeds significantly higher than those assumed in the 2004 AASHTO Green Book, therefore require much longer distance to accelerate. They proposed new acceleration lengths (Table 3-16) using a constant acceleration rate of 2.5 ft/s^2 (based on the Geometric Design Guide for Canadian Roads, 1999) and new merging speeds (i.e., highway design speeds used in the 2004 AASHTO Green Book).

Table 3-16. Minimum acceleration lengths (ft) for trucks

Merging speed (mi/h)		Initial speed (mi/h)								
2004 AASHTO Green Book	Fitzpatrick and Zimmerman	0	15	20	25	30	35	40	45	50
23	30	389	292	216	-	-	-	-	-	-
27	35	529	432	357	259	-	-	-	-	-
31	40	691	594	519	421	303	-	-	-	-
35	45	875	778	702	605	486	346	-	-	-
39	50	1080	983	908	810	691	551	389	-	-
43	55	1307	1210	1134	1037	918	778	616	432	-
47	60	1556	1459	1383	1286	1167	1026	864	681	475
50	65	1826	1729	1653	1556	1437	1297	1134	951	746
53	70	2118	2020	1945	1848	1729	1588	1426	1243	1037
55	75	2431	2334	2258	2161	2042	1902	1740	1556	1351

Source: Table 5 of "Potential Updates to 2004 Green Book's Acceleration Lengths for Entrance Terminals".

Gattis et al.

Gattis et al. (2008) examined attributes associated with 526 tractor-trailer trucks accelerating from a stopped position on tapered on-ramps with grades of −0.6%, −0.2%, 0%, 0.1%, and 0.8%. They considered −0.6% grade as downgrade, −0.2%, 0%, and 0.1% grades as level grades, and 0.8% grade as upgrade. Gattis et al. further developed a model (Equation 9), via regression analysis, to predict average and 10th-percentile speeds, with a 90% confidence level, for tractor-trailer trucks traveling a given distance. In attempt to correct the issues with the truck speed models (i.e., negative slope and asymptotic), it was decided to develop new models eliminating the first 1000 ft data at the beginning of each dataset.

$$V = a_0 + Xa_1 + X^2a_2 \tag{3-9}$$

where

X	Acceleration length (ft),
V	Merging speed (mi/h), and
a_0, a_1, a_2	Coefficients of distance (Table 3-17)

Table 3-17. Average and 10th percentile speed model coefficients for tractor-trailer trucks

	Speed (mi/h)	a_0	a_1	a_2
Downgrade	Average	19.8869	0.0201	-2.44×10^{-6}
	10th percentile	15.1563	0.0186	-1.84×10^{-6}
Level	Average	22.2720	0.0169	-1.80×10^{-6}
	10th percentile	19.6650	0.0151	-1.41×10^{-6}
Upgrade	Average	15.4647	0.0223	-3.17×10^{-6}
	10th percentile	12.2413	0.0214	-3.12×10^{-6}

Source: Values are derived from the information in Gattis et al.'s study.

The recommended acceleration lengths in Table 3-18 through Table 3-20 are generated through solving Equation 3-9 for the merging speeds given in those tables. Even though vehicles require longer acceleration lengths on steeper grades, some of the generated acceleration lengths for level grade (Table 3-19) are significantly lower than those for downgrades (Table 3-18). The reason is that Gattis et al. proposed some lower coefficients for level grades compared with those for downgrades (no explanation was provided), which will consequently lead to lower acceleration lengths for level grades compared with those for downgrades.

Table 3-18. Predicted acceleration lengths (ft) for tractor-trailer trucks on downgrade

Percentile distance	Merging speed (mi/h) 30	35	40	45	50	55	60
90th	-	1178	1545	1957	2432	3014	3852
50th	-	-	1132	1497	1922	2452	3265

Source: Values are calculated from the information in Gattis et al. (2008) study.

Table 3-19. Predicted acceleration lengths (ft) for tractor-trailer trucks on level grade

Percentile distance	Merging speed (mi/h) 30	35	40	45	50	55	60
90th	-	1095	1534	2030	2614	3364	4707
50th	-	-	1164	1582	2066	2662	3531

Source: Values are calculated from the information in Gattis et al. (2008) study.

Table 3-20. Predicted acceleration lengths (ft) for tractor-trailer trucks on upgrade

Percentile distance	Merging speed (mi/h) 30	35	40	45	50	55	60
90th	-	1279	1691	2237	-	-	-
50th	-	1000	1329	1726	2239	3277	-

Source: Values are calculated from the information in Gattis et al. (2008) study.

ITE Traffic Engineering Handbook

The ITE Traffic Engineering Handbook (Kraft et al., 2009) provides maximum acceleration rates of tractor-semitrailer trucks on level grade from a stopped position to merging speeds of 10-50 mi/h, in 10 mi/h increments, at various initial speeds. As shown in Table 3-21, for a given initial speed, acceleration rates decrease with an increase ing merging speeds.

Table 3-21. Maximum acceleration rates (ft/s^2) for tractor-semitrailer trucks on level grade

Weight to power ratio (lb/hp)	Initial speed (mi/h) 0	0	0	0	0	20	30	40	50
	Merging speed (mi/h) 10	20	30	40	50	30	40	50	60
100	2.9	2.3	2.2	2.0	1.6	2.1	1.5	1.0	0.6
200	1.8	1.6	1.5	1.2	1.0	1.3	0.8	0.5	0.4
300	1.3	1.3	1.2	1.1	0.6	1.0	0.6	0.3	-
400	1.3	1.2	1.1	0.7	-	0.9	0.4	-	-

Source: Values are derived from the information in the ITE Traffic Engineering Handbook.

Research Papers Related to Maximum Acceleration Rates of Trucks

Proctor et al. (1995) calculated average acceleration rates of 0.064 g (2.06 ft/s^2), 0.073 g (2.35 ft/s^2), and 0.118 g (3.18 ft/s^2) for 219 heavy trucks with flatbed, box, and bobtail configurations using their distance vs. time graphs. Long (2000) suggested to use linearly decreasing acceleration models as opposed to piecewise-constant acceleration models used in the AASHTO Green Books, since they better replicate maximum acceleration capabilities and actual driver behaviors. Long showed that single-unit trucks have 45% lower acceleration rates at lower speeds and 300%-400% lower acceleration rates at higher speeds compared with the values used in the 1994 AASHTO Green Book. Poplin (2002) examined 17 lightly-loaded (average weights of 30,654 lb), 8 moderately-loaded (average weight of 42,330 lb), and 11 heavily-loaded trucks (average weight of 74,607 lb) over a distance of approximately 300 ft. Poplin observed that those trucks were able to reach speeds of 9 mi/h-23 mi/h from a stopped position with average acceleration rates of:

- 0.7 ft/s^2-3.2 ft/s^2 for lightly-loaded trucks,
- 0.4 ft/s^2-2.2 ft/s^2 for moderately-loaded trucks, and
- 0.4 ft/s^2-1.9 ft/s^2 for heavily-loaded trucks, respectively.

Mehar et al. (2013) examined acceleration capabilities of heavy trucks in empty, half-loaded, and fully loaded conditions. Figure 3-1 shows the fitting of exponential functions (Eq. 3-10) to average acceleration (a_{avg}) vs. speed (v) of heavy vehicles in three loading states on a 4-lane divided highway. The produced acceleration profiles on a 6-lane divided highway were statistically identical to those on a 4-lane divided highway.

$$a_{avg} = \begin{cases} 2.19e^{-0.03}, & for\ empty\ heavy\ vehicles \\ 1.65e^{-0.04v}, & for\ half-loaded\ heavy\ vehicles \\ 0.98e^{-0.03v}, & for\ fully-loaded\ heavy\ vehicles \end{cases} \tag{3-10}$$

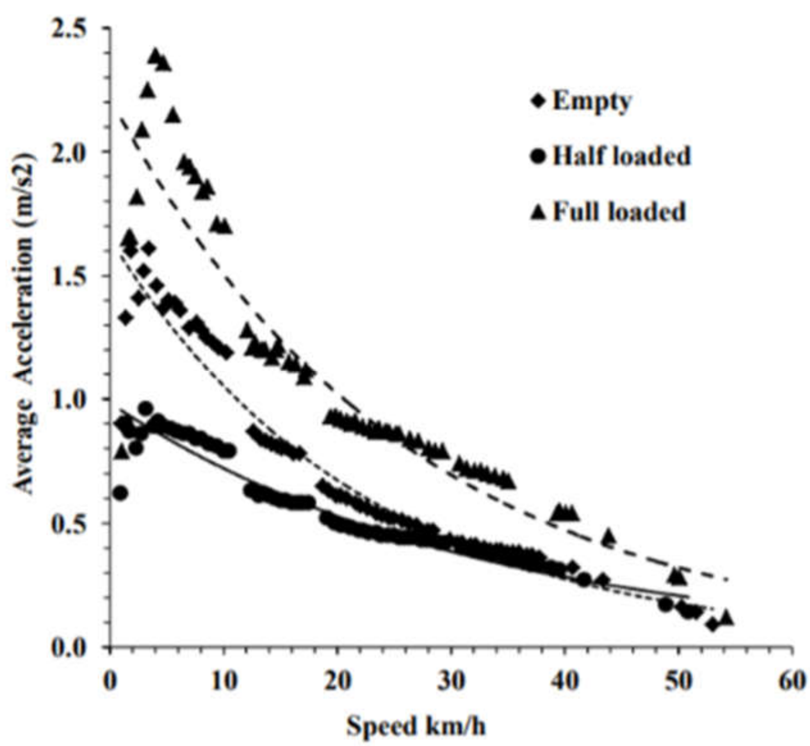


Figure 3-1. Acceleration of heavy vehicles on 4-lane divided highways
Source: Figure 5 of Mehar et al. study

Yang

Yang (2017) studied the impacts of ramp metering operations on acceleration capabilities of trucks. This study examined 44 light trucks, 114 medium trucks, and 71 heavy trucks and found average acceleration rates of 2.82 ft/s^2, 2.46 ft/s^2, and 1.96 ft/s^2, respectively. Yang suggested to consider 15th-percentile acceleration rates (i.e., approximately 30% less than average acceleration rates) instead of average acceleration rates for conservative design of acceleration lanes in order to accommodate safe merging speeds for 85% of the drivers. Assuming a piecewise-constant acceleration model, Yang developed minimum distances (Table 3-22 and Table 3-23) required for trucks to accelerate from a stopped position at the ramp-meter stop bar to the desired merging speed. However, Yang did not consider the impact of ramp geometry, including ramp grade, on the required acceleration lengths.

Table 3-22. Minimum acceleration lengths (ft) for medium trucks on metered on-ramps

Percentile distance	Merging speed (mi/h)						
	30	35	40	45	50	55	60
85th	525	760	1050	1400	1815	2290	2830
50th	370	555	790	1080	1430	1840	2320
15th	255	390	570	790	1065	1395	1785

Source: Table 6-6 of Yang (2017).

Table 3-23. Minimum acceleration lengths (ft) for heavy trucks on metered on-ramps

Percentile distance	Merging speed (mi/h)						
	30	35	40	45	50	55	60
85th	685	975	1320	1725	2190	2720	3320
50th	505	735	1020	1355	1750	2205	2720
15th	410	610	855	1155	1510	1925	2405

Source: Table 6-6 of Yang (2017).

TruckSim Results

The AASHTO Green Book is used as a reference in Florida for design of acceleration lanes. Based on the input received from the FDOT personnel, who were involved in the process of planning, design, and operation of ramp metering systems, the recommended acceleration lengths in the AASHTO Green Book were not sufficient to accommodate safe merging speeds for trucks (particularly for large trucks). This is because AASHTO has based its studies on passenger cars and has not provided any guidelines on design of acceleration lanes for trucks. To better consider realistic acceleration capabilities of trucks, the research team examined different acceleration scenarios using the TruckSim simulation tool.

TruckSim is a tool developed by the Mechanical Simulation Corporation for simulating dynamic behavior of single-unit trucks and tractor-semi-trailer combinations (http://carsim.com/products/trucksim/). TruckSim includes provisions for interfacing with Matlab/Simulink to simulate response of vehicles to driver inputs (e.g., steering, braking, and acceleration) and environment (e.g., road and weather) with active controls. TruckSim is superior to other tools for having pre-defined vehicle models, fast runtime, easy-to-use interface, and what-if analysis. The following assumptions were used for the simulation runs (Figure 3-2-Figure 3-4):

- Initial speed, open-loop throttle
- Ramp to full throttle in 0.5 seconds
- No open-loop braking pressure
- Auto shift and auto clutch (all gears)
- No offset with 1 second preview
- Constant friction factor = 0.85
- Surface coefficient for tire rolling resistance = 1
- Truck weight = 53,000[1] lb and 80,000[2] lb

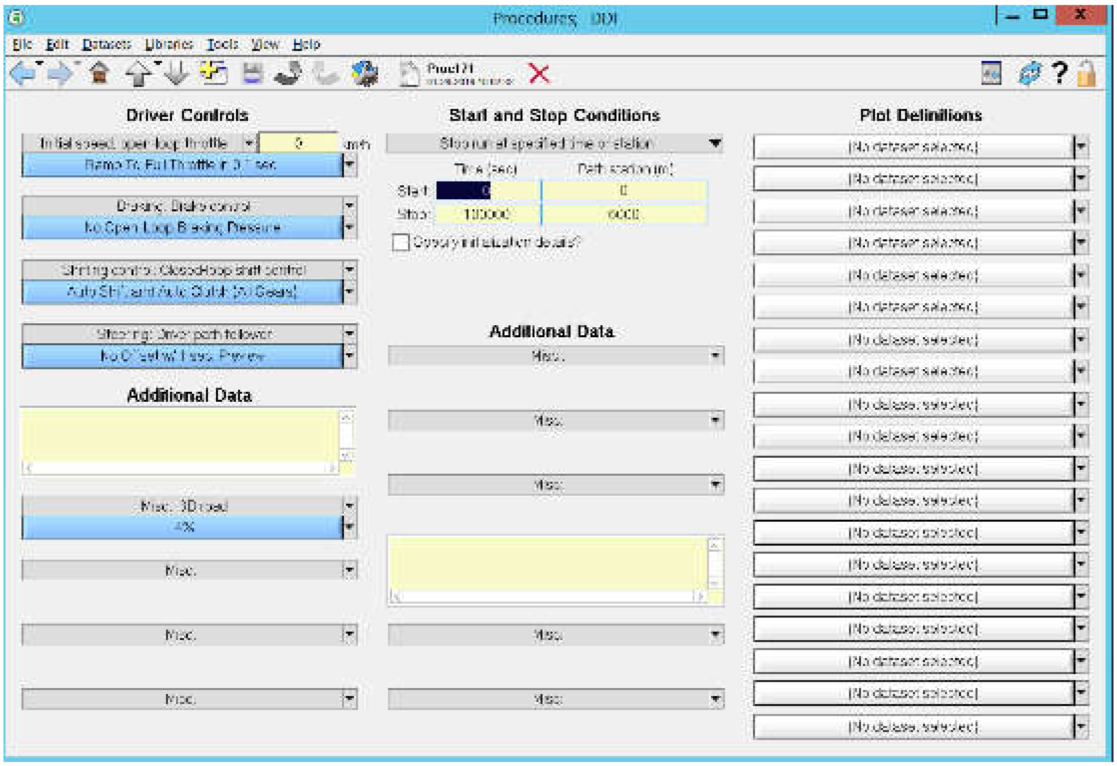


Figure 3-2. TruckSim driver controls inputs

[1] Based on Florida weigh-in-motion data. More information can be found in the references: Washburn. and Ozkul (2013) and Ozkul (2014)

[2] Maximum legal load without a permit.

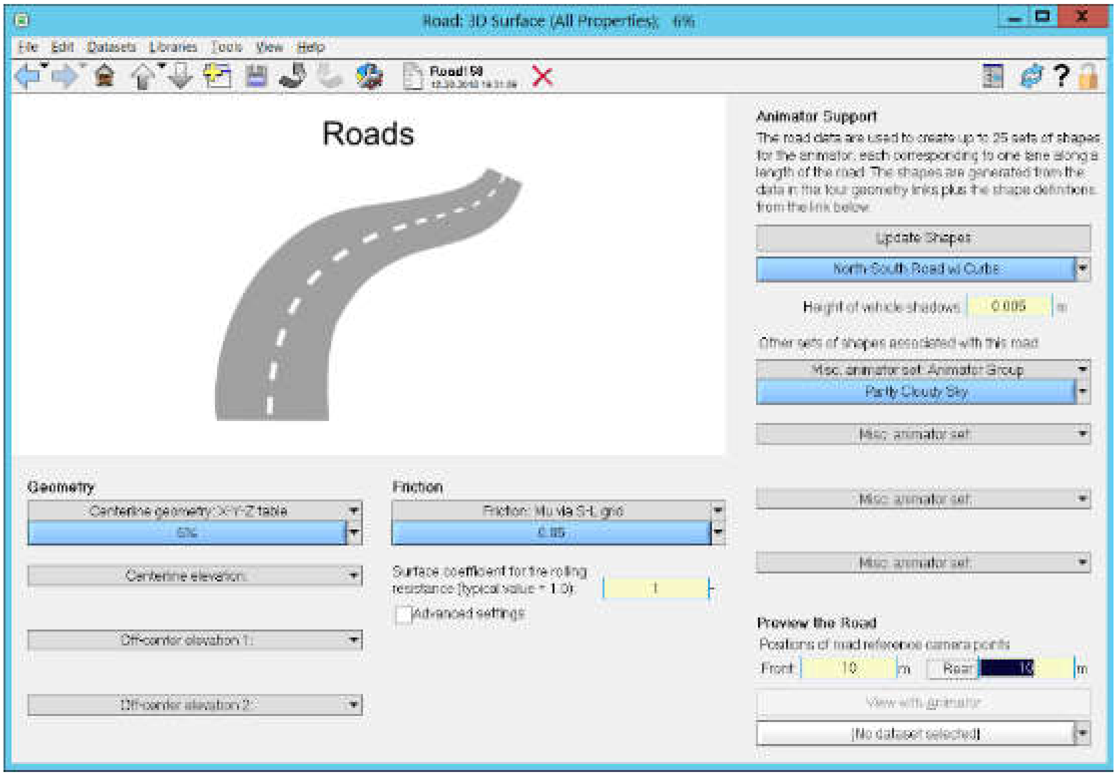


Figure 3-3. TruckSim roadway design inputs

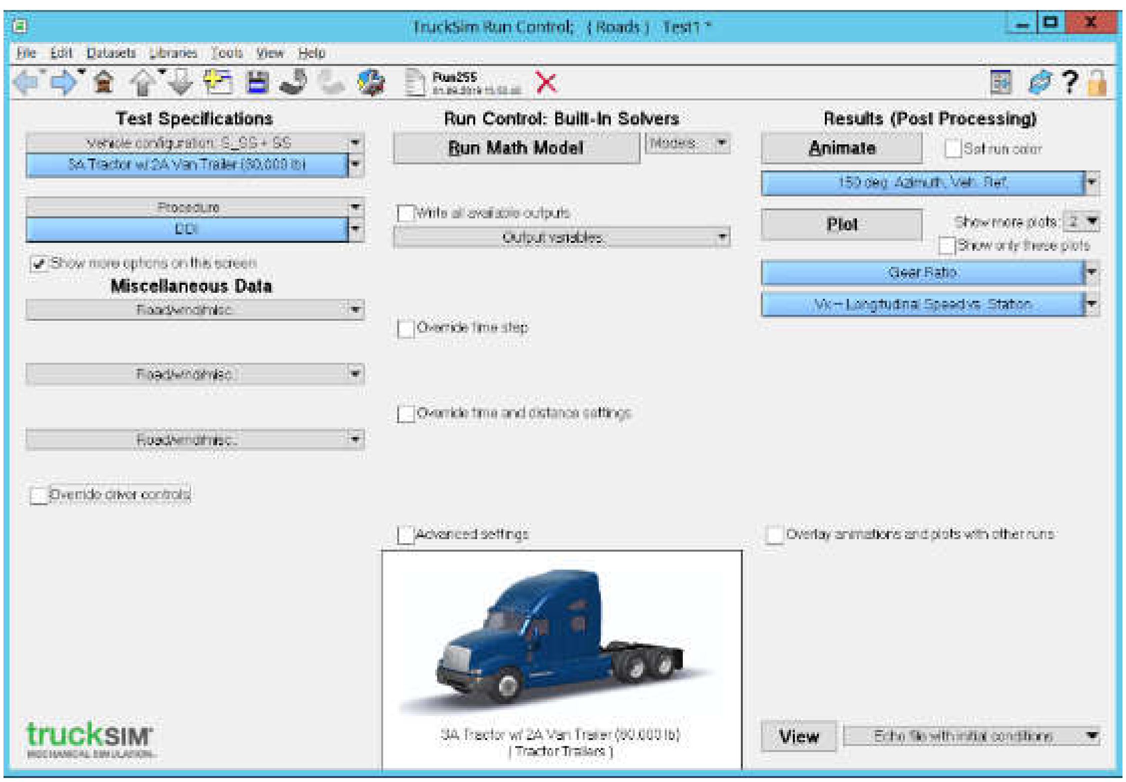


Figure 3-4. TruckSim run control page

Using the speed vs. distance outputs, the required acceleration lengths for 53,000-lb trucks and 80,000-lb trucks were calculated for the initial speeds and merging speeds used in the NCHRP Report 505 (Table 3-24 through Table 3-31).

Table 3-24. Minimum acceleration lengths (ft) for 53,000-lb trucks on −2% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	205	141	94	57	-	-	-	-	-
27	298	234	187	130	69	-	-	-	-
31	396	332	284	219	158	84	-	-	-
35	506	442	395	329	269	208	-	-	-
39	691	627	579	514	453	354	165	-	-
43	862	798	751	685	625	525	378	177	-
47	1065	1001	953	888	827	728	610	408	193
50	1305	1241	1193	1128	1067	968	801	599	384
53	1517	1453	1405	1340	1279	1180	1012	811	596
55	1671	1606	1559	1494	1433	1334	1166	965	750

Table 3-25. Minimum acceleration lengths (ft) for 53,000-lb trucks on level grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	233	163	111	70	-	-	-	-	-
27	355	285	233	192	85	-	-	-	-
31	468	399	346	306	198	112	-	-	-
35	612	543	490	449	342	283	-	-	-
39	863	794	741	701	594	483	264	-	-
43	1105	1036	983	943	835	724	598	276	-
47	1400	1331	1278	1238	1131	1019	958	636	300
50	1772	1702	1650	1609	1502	1391	1259	937	601
53	2114	2045	1992	1952	1844	1733	1602	1279	944
55	2369	2300	2247	2207	2099	1988	1857	1534	1199

Table 3-26. Minimum acceleration lengths (ft) for 53,000-lb trucks on 2% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	271	196	135	82	-	-	-	-	-
27	430	355	295	242	111	-	-	-	-
31	585	511	450	397	266	156	-	-	-
35	790	716	655	602	471	361	-	-	-
39	1187	1113	1052	999	868	758	666	-	-
43	1597	1522	1461	1408	1278	1168	883	636	-
47	2140	2065	2004	1951	1821	1711	1396	1160	680
50	2972	2897	2836	2783	2652	2542	2162	1744	1392
53	3868	3793	3733	3679	3549	3439	3058	2653	2288
55	4615	4541	4480	4427	4296	4186	3805	3429	3036

Table 3-27. Minimum acceleration lengths (ft) for 53,000-lb trucks on 4% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	327	247	179	110	-	-	-	-	-
27	558	478	410	341	169	-	-	-	-
31	803	722	655	586	414	228	-	-	-
35	1163	1083	1015	946	774	588	-	-	-
39	2116	2036	1968	1899	1727	1541	2014	-	-
43	3472	3392	3324	3255	3083	2897	3370	5034	-
47	7321	7240	7173	7104	6932	6746	7218	8882	11564
50	-	-	-	-	-	-	-	-	-
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Table 3-28. Minimum acceleration lengths (ft) for 80,000-lb trucks on −2% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	242	168	112	59	-	-	-	-	-
27	355	280	224	155	71	-	-	-	-
31	482	407	351	273	189	88	-	-	-
35	627	553	496	418	334	252	-	-	-
39	851	777	720	642	558	441	204	-	-
43	1073	999	942	865	780	663	476	220	-
47	1335	1261	1204	1126	1042	925	772	515	240
50	1619	1545	1488	1411	1326	1209	1015	758	484
53	1888	1814	1757	1679	1595	1478	1284	1027	752
55	2082	2008	1951	1874	1789	1673	1478	1221	947

Table 3-29. Minimum acceleration lengths (ft) for 80,000-lb trucks on level grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	291	210	143	79	-	-	-	-	-
27	452	371	304	240	98	-	-	-	-
31	617	535	469	405	263	134	-	-	-
35	827	746	679	615	474	390	-	-	-
39	1175	1094	1028	963	822	687	398	-	-
43	1537	1456	1389	1325	1184	1048	913	414	-
47	1982	1901	1835	1770	1629	1494	1467	968	449
50	2509	2428	2361	2297	2156	2020	1931	1432	914
53	3041	2960	2893	2829	2688	2552	2463	1964	1446
55	3440	3359	3293	3229	3087	2952	2862	2363	1845

Table 3-30. Minimum acceleration lengths (ft) for 80,000-lb trucks on 2% grade

Merging speed (mi/h)	Initial speed (mi/h) 0	14	18	22	26	30	36	40	44
23	372	280	199	105	-	-	-	-	-
27	617	526	444	350	154	-	-	-	-
31	894	802	721	627	430	210	-	-	-
35	1278	1186	1105	1011	814	594	-	-	-
39	2064	1973	1891	1797	1601	1381	1024	-	-
43	3036	2945	2864	2770	2573	2353	1996	1687	-
47	4548	4457	4375	4281	4085	3865	3655	3378	3110
50	9355	9264	9182	9088	8892	8672	8316	8025	7700
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Table 3-31. Minimum acceleration lengths (ft) for 80,000-lb trucks on 4% grade

Merging speed (mi/h)	Initial speed (mi/h) 0	14	18	22	26	30	36	40	44
23	534	430	331	176	-	-	-	-	-
27	1069	965	866	711	390	-	-	-	-
31	1925	1822	1723	1568	1246	661	-	-	-
35	4370	4266	4167	4012	3691	3105	-	-	-
39	-	-	-	-	-	-	-	-	-
43	-	-	-	-	-	-	-	-	-
47	-	-	-	-	-	-	-	-	-
50	-	-	-	-	-	-	-	-	-
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Not surprisingly, the acceleration distances generated from TruckSim were significantly longer than those provided in the literature.

SwashSim Results

The research team also examined acceleration distances with the microscopic simulation tool SwashSim. SwashSim implements a detailed vehicle acceleration model and includes detailed vehicle and powertrain characteristics for several classes of commercial truck. The approach used to model vehicle maximum acceleration in SwashSim is described in Washburn and Ozkul (2013) and Ozkul (2014). The Time Step Data output file includes outputs such as position, velocity, and acceleration for every simulation time step, which can be used to calculate the required distance to accelerate from a stop to a specified merging speed.

Table 3-32 through Table 3-37 show the calculated acceleration lengths for 53,000-lb and 80,000-lb trucks on level, 2%, and 4% grades.

Table 3-32. Minimum acceleration lengths (ft) for 53,000-lb trucks on level grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	193	125	93	36	-	-	-	-	-
27	290	222	190	133	60	-	-	-	-
31	412	344	312	256	183	79	-	-	-
35	582	514	482	426	353	248	-	-	-
39	805	738	706	649	576	472	254	-	-
43	1077	1010	978	921	848	744	526	314	-
47	1454	1386	1354	1297	1224	1120	901	690	440
50	1828	1761	1728	1671	1598	1495	1275	1064	814
53	2227	2160	2128	2069	1997	1895	1674	1463	1214
55	2539	2472	2440	2382	2309	2207	1986	1775	1526

Table 3-33. Minimum acceleration lengths (ft) for 53,000-lb trucks on 2% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	225	151	151	151	-	-	-	-	-
27	353	279	279	279	82	-	-	-	-
31	522	448	448	448	251	110	-	-	-
35	781	707	707	707	510	369	-	-	-
39	1158	1085	1085	1085	887	746	427	-	-
43	1663	1589	1589	1589	1391	1249	932	577	-
47	2558	2484	2484	2484	2284	2142	1825	1471	1025
50	3883	3810	3810	3810	3610	3468	3151	2798	2351
53	5344	5271	5271	5271	5071	4929	4612	4262	3812
55	7147	7074	7074	7074	6874	6732	6415	6065	5615

Table 3-34. Minimum acceleration lengths (ft) for 53,000-lb trucks on 4% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	274	274	274	274	-	-	-	-	-
27	462	462	462	462	125	-	-	-	-
31	737	737	737	737	400	181	-	-	-
35	1285	1285	1285	1285	947	730	-	-	-
39	2503	2503	2503	2503	2166	1949	11341	-	-
43	-	-	-	-	-	-	-	-	-
47	-	-	-	-	-	-	-	-	-
50	-	-	-	-	-	-	-	-	-
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Table 3-35. Minimum acceleration lengths (ft) for 80,000-lb trucks on level grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	309	309	309	309	-	-	-	-	-
27	468	468	468	468	101	-	-	-	-
31	672	672	672	672	305	132	-	-	-
35	962	962	962	962	595	422	-	-	-
39	1360	1360	1360	1360	992	819	449	-	-
43	1851	1851	1851	1851	1484	1311	941	570	-
47	2569	2569	2569	2569	2202	2028	1658	1287	839
50	3344	3344	3344	3344	2978	2803	2431	2061	1614
53	4178	4178	4178	4178	3813	3638	3266	2896	2449
55	4898	4898	4898	4898	4532	4358	3985	3616	3169

Table 3-36. Minimum acceleration lengths (ft) for 80,000-lb trucks on 2% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	403	277	212	87	-	-	-	-	-
27	667	541	477	351	176	-	-	-	-
31	1045	919	854	730	553	248	-	-	-
35	1764	1637	1574	1449	1273	968	-	-	-
39	3166	3040	2977	2851	2676	2371	1566	-	-
43	-	-	-	-	-	-	-	-	-
47	-	-	-	-	-	-	-	-	-
50									
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Table 3-37. Minimum acceleration lengths (ft) for 80,000-lb trucks on 4% grade

Merging speed (mi/h)	Initial speed (mi/h)								
	0	14	18	22	26	30	36	40	44
23	604	450	363	162	-	-	-	-	-
27	1506	1352	1265	1067	708	-	-	-	-
31	-	-	-	-	-	-	-	-	-
35	-	-	-	-	-	-	-	-	-
39	-	-	-	-	-	-	-	-	-
43	-	-	-	-	-	-	-	-	-
47	-	-	-	-	-	-	-	-	-
50	-	-	-	-	-	-	-	-	-
53	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-

Figure 3-5 though Figure 3-16 provide a comparison of the speed profiles and acceleration profiles produced in SwashSim and TruckSim for 53,000-lb and 80,000-lb trucks, accelerating from standstill on a 30,000-ft section of a level grade, 2% upgrade, and 4% upgrade. The followings are key factors contributing to differences between the acceleration lengths produced in SwashSim and TruckSim, particularly at high merging speeds:

1. SwashSim uses a drivetrain efficiency of 0.8 for large trucks, while TruckSim uses values between 0.92 and 0.95. For a large truck with a 10-speed transmission, TruckSim uses a transmission efficiency equal to 0.92 for gears 1 through 7 and 0.95 for gears 8 through 10.
2. Trucks in SwashSim shift gears according to two different principles: retaining a maximum acceleration on a grade and slowing down for leaders based on speed thresholds. Trucks in TruckSim shift according to transmission output speeds, which are a function of throttle position. Shift schedules are specified separately for changes between each gear as well as for upshifting and downshifting.
3. TruckSim has predefined engines with a max power of 300 kW (402 hp) and 330 kW (443 hp). Both engines have a max speed equal to 2100 rpm and idle speed equal to 800 rpm. SwashSim has a large truck engine with a max power around 520 hp, max speed equal to 2200 rpm, and idle speed equal to 700 rpm.

SwashSim takes into an account the transmission gear-shifting capabilities of commercial trucks, being able to output the transmission gear selected at each time step of the simulation. The underlying transmission shifting logic in SwashSim is close to the actual field commercial truck performance coded in TruckSim. The typical drivetrain efficiency, drive axle slippage, differential gear ratio, and gear shift up/down speeds of large trucks are coded in SwashSim (Ozkul 2014).

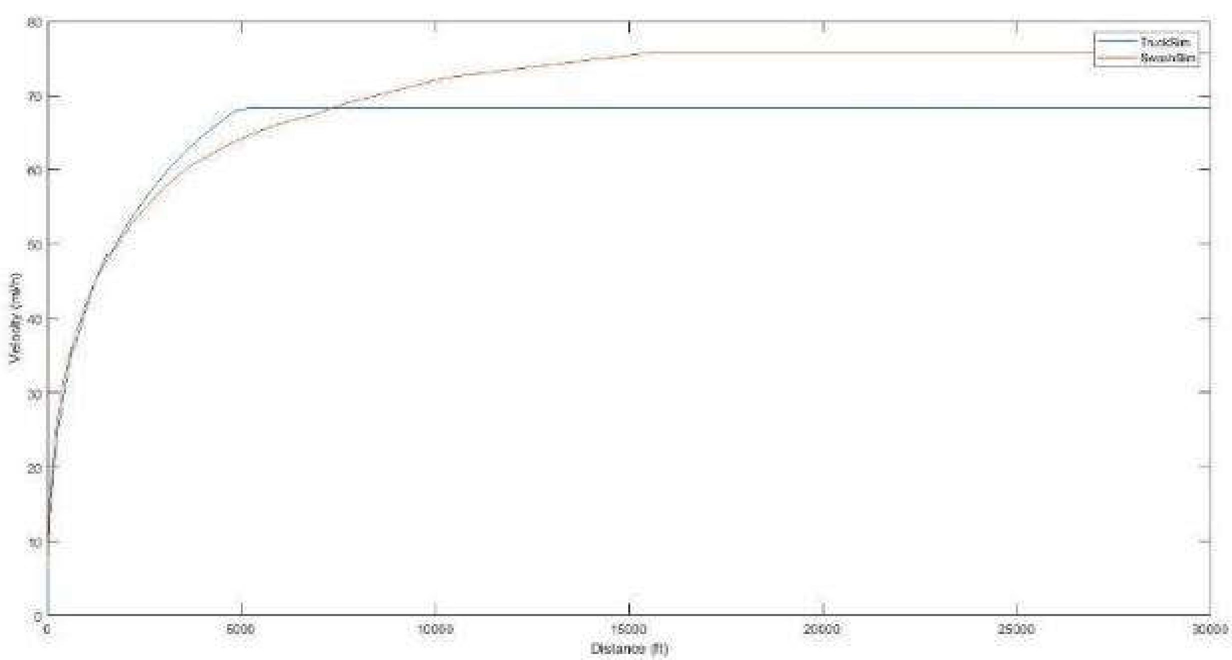


Figure 3-5. Velocity profiles for 53,000-lb truck on level grade

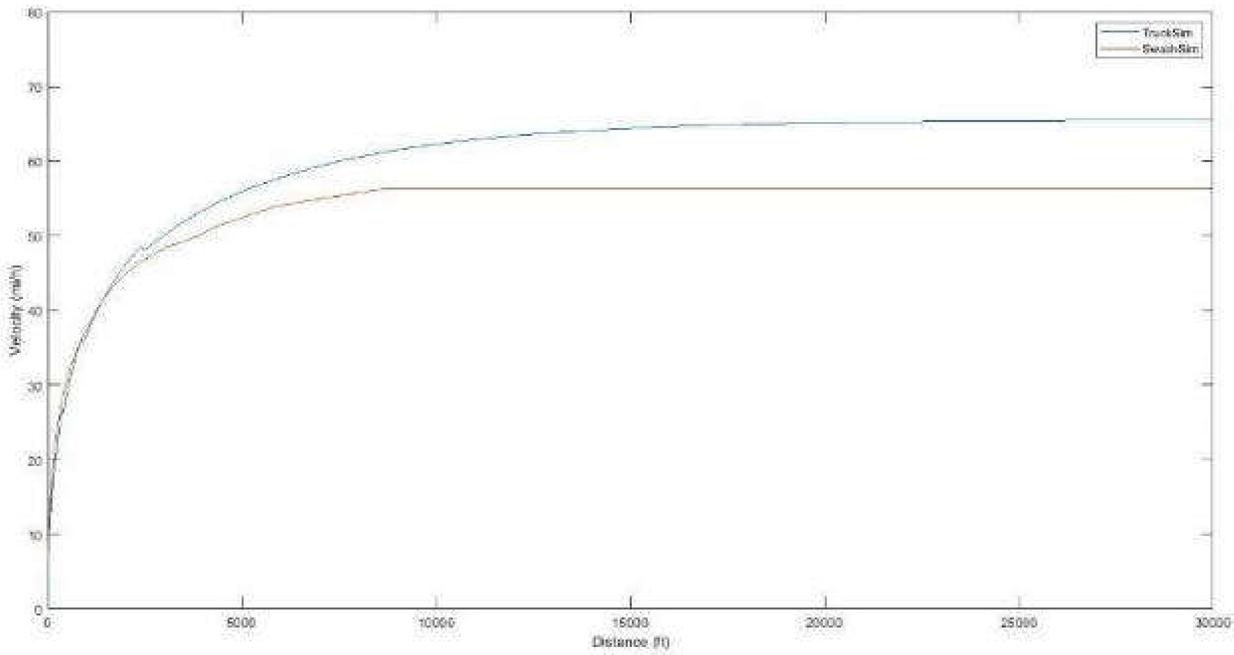


Figure 3-6. Velocity profiles for 53,000-lb truck on 2% grade

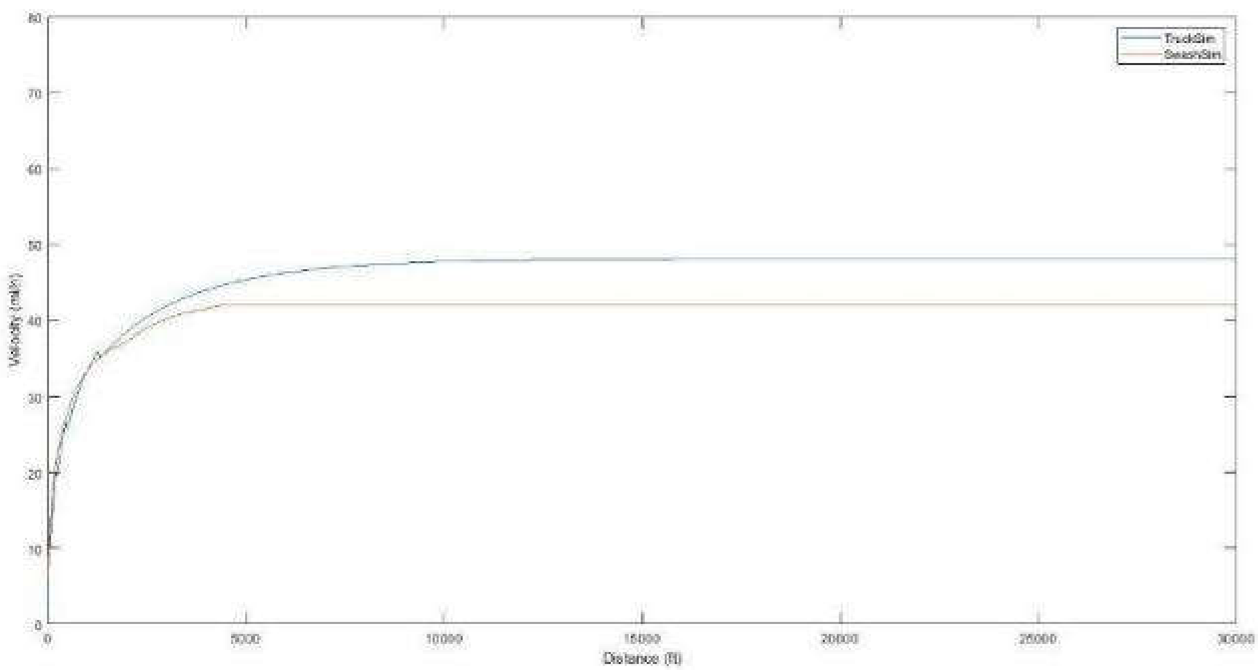


Figure 3-7. Velocity profiles for 53,000-lb truck on 4% grade

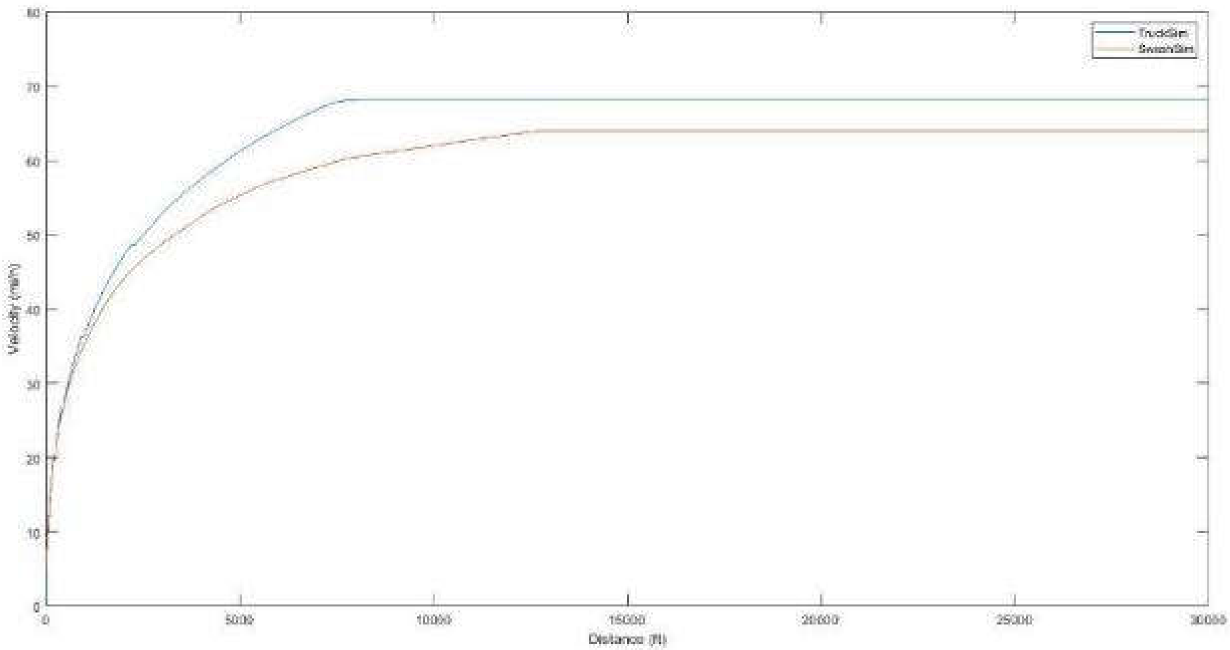


Figure 3-8. Velocity profiles for 80,000-lb truck on level grade

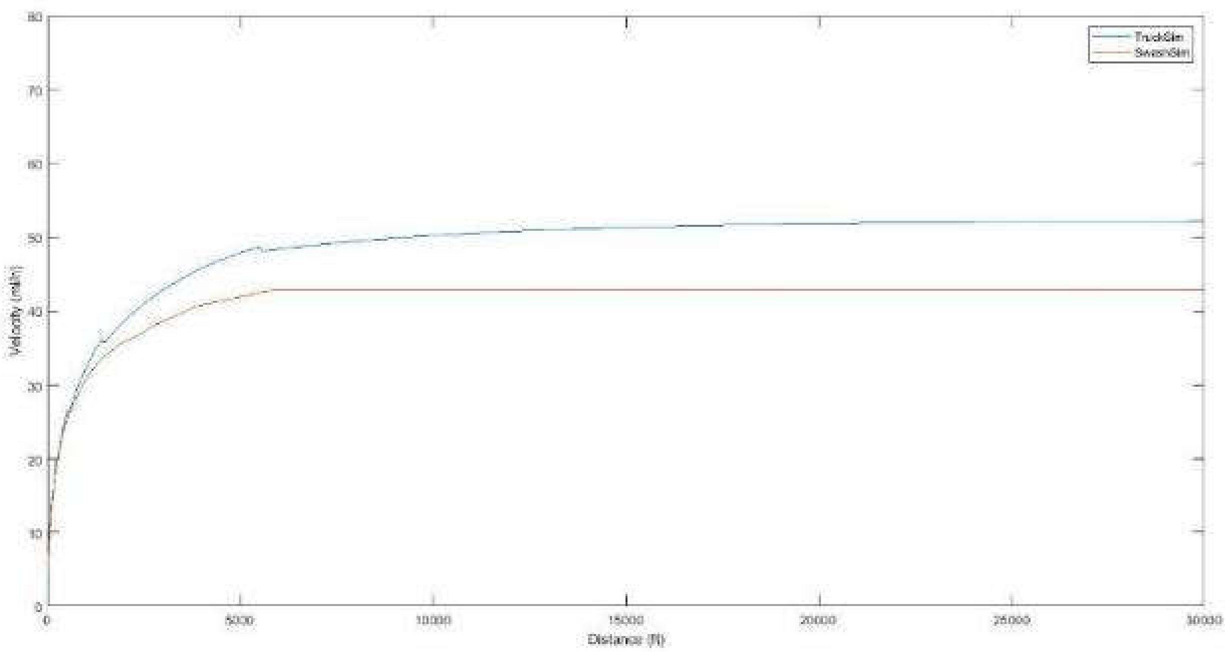


Figure 3-9. Velocity profiles for 80,000-lb truck on 2% grade

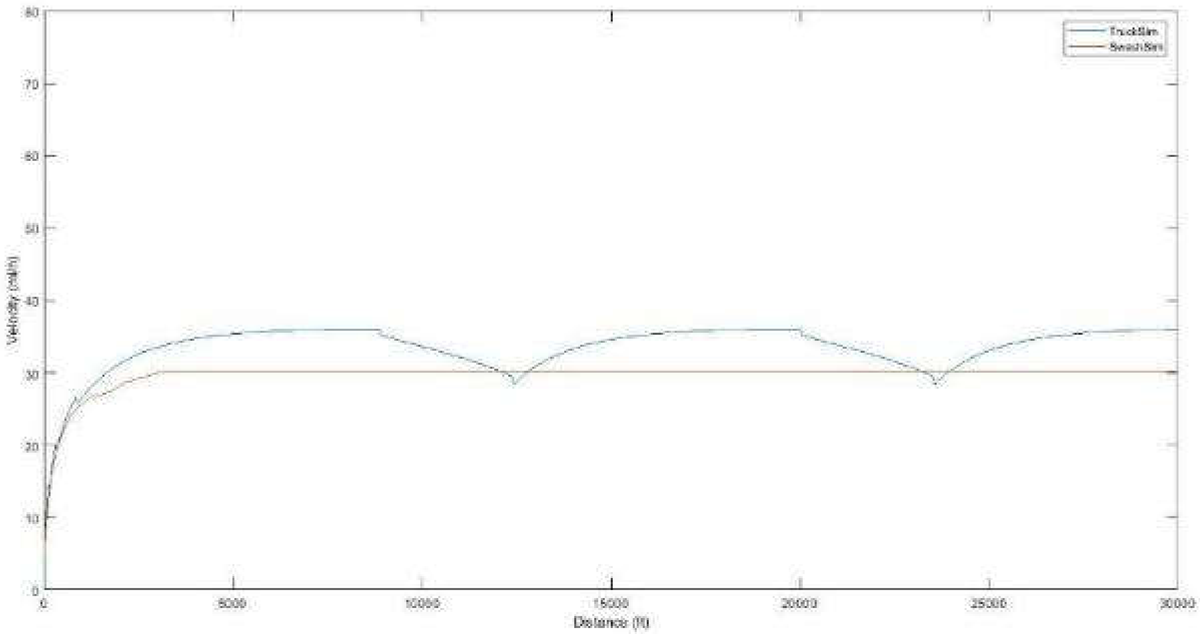


Figure 3-10. Velocity profiles for 80,000-lb truck on 4% grade

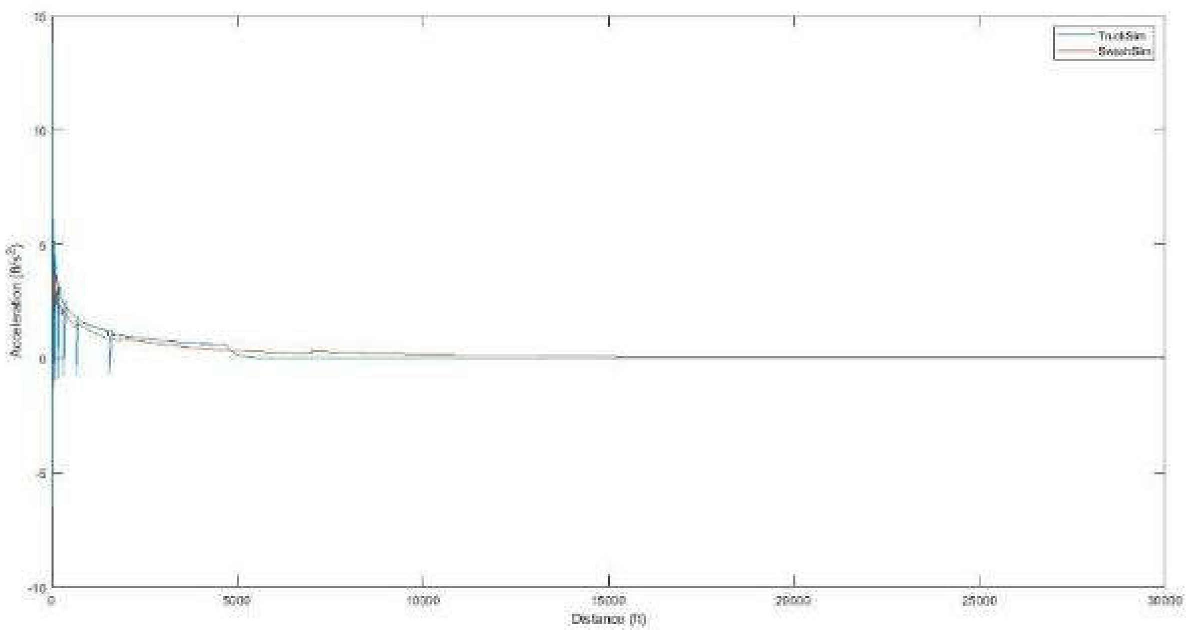


Figure 3-11. Acceleration profiles for 53,000-lb truck on level grade

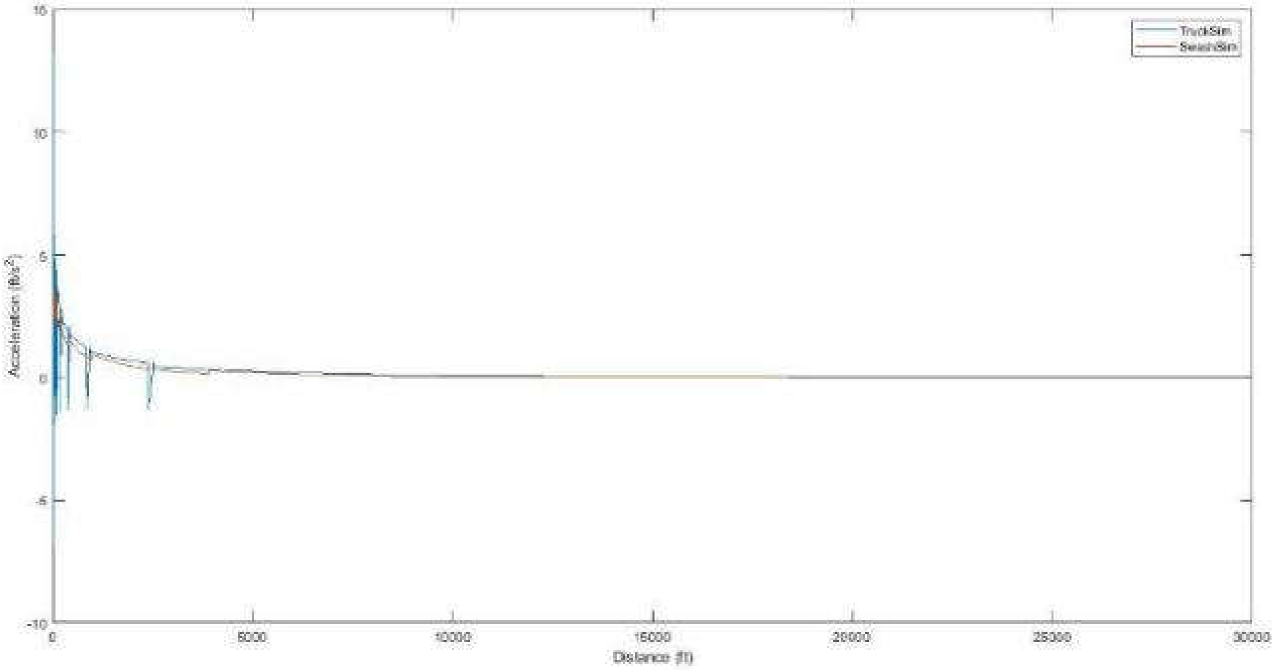


Figure 3-12. Acceleration profiles for 53,000-lb truck on 2% grade

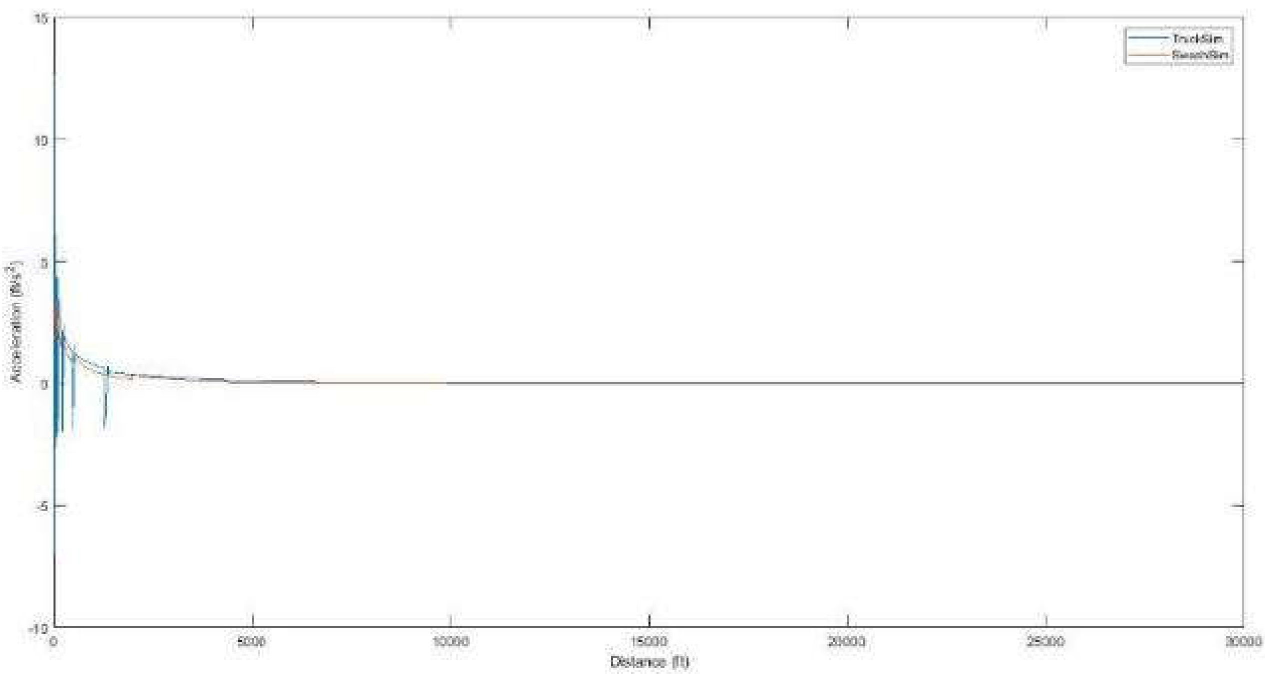


Figure 3-13. Acceleration profiles for 53,000-lb truck on 4% grade

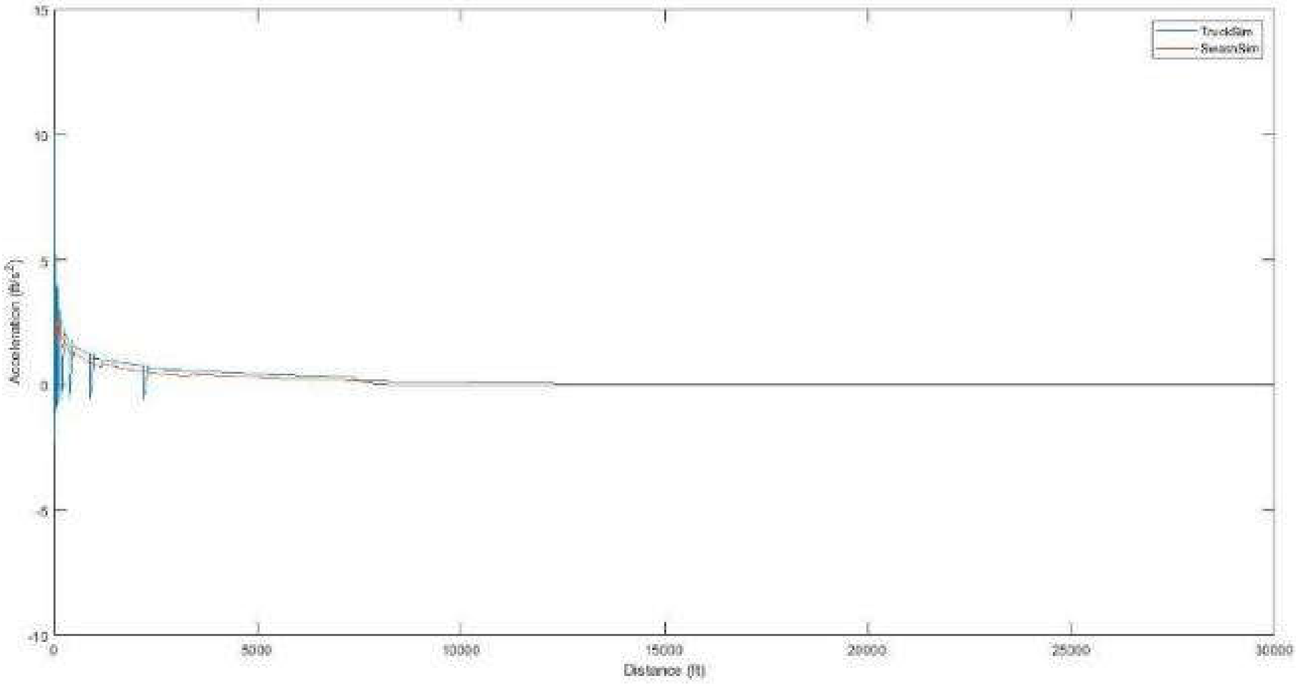


Figure 3-14. Acceleration profiles for 80,000-lb truck on level grade

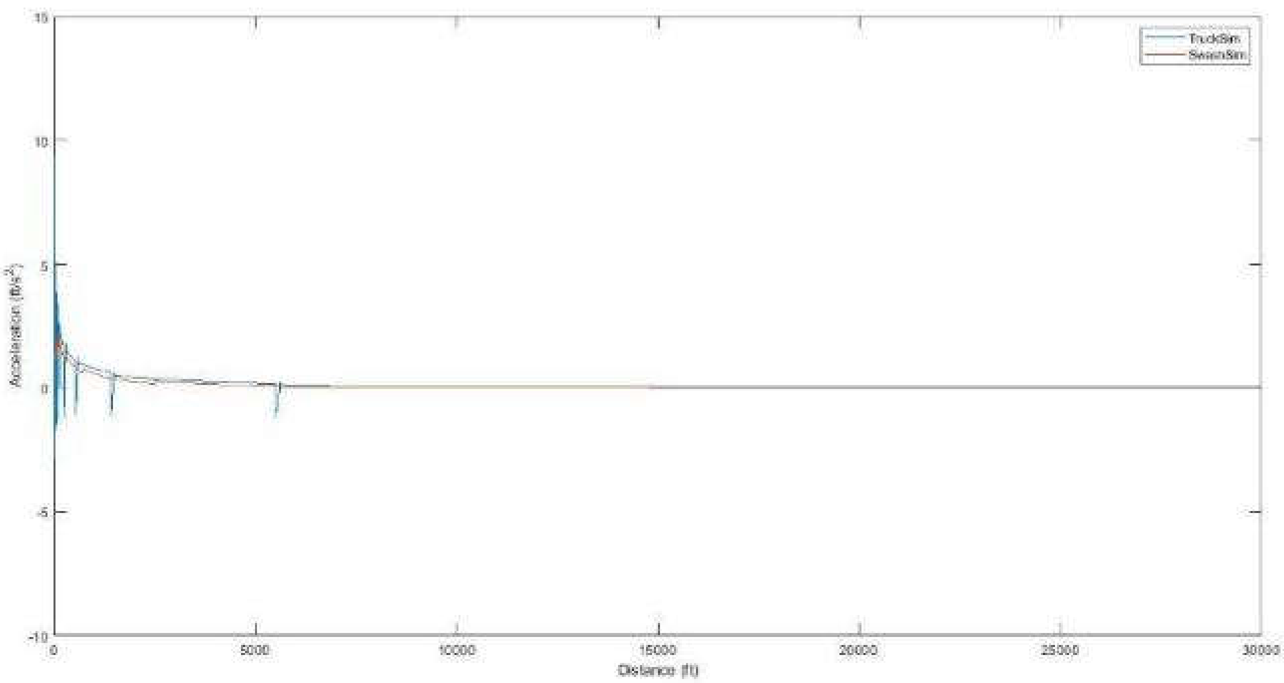


Figure 3-15. Acceleration profiles for 80,000-lb truck on 2% grade

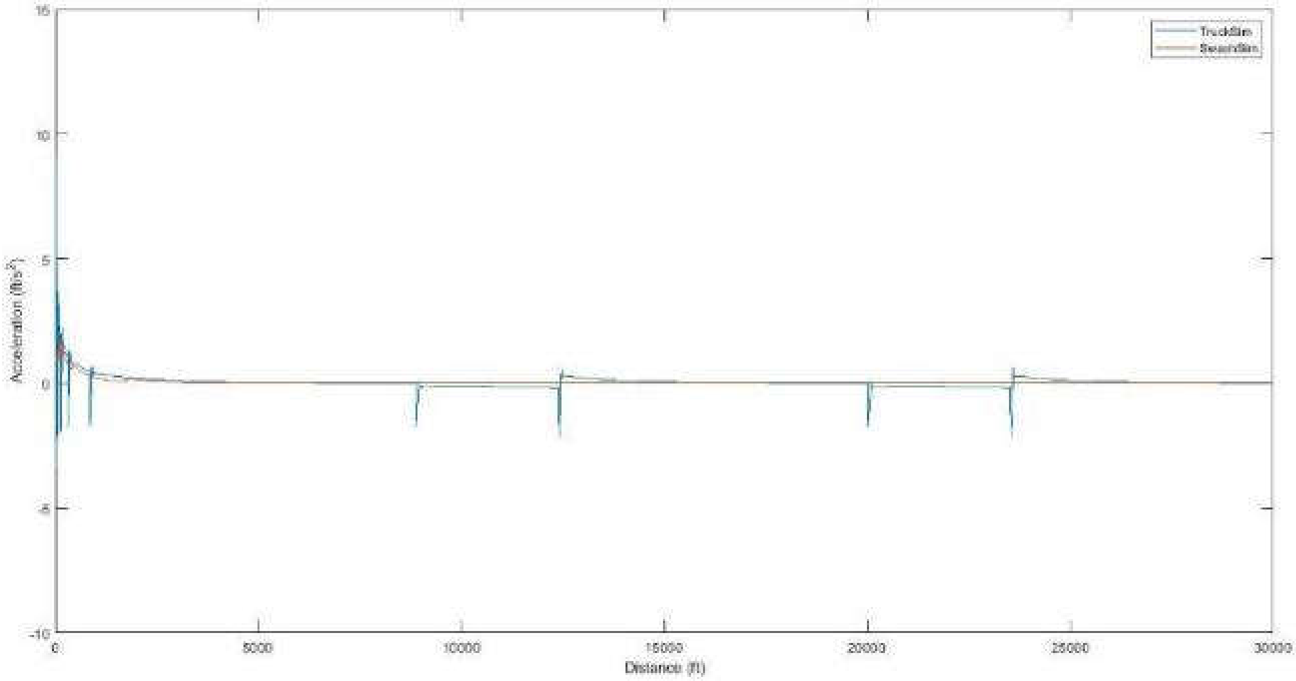


Figure 3-16. Acceleration profiles for 80,000-lb truck on 4% grade

Summary

To study the impacts of ramp metering on acceleration distances for large trucks, all the recommended acceleration lengths in the literature are compared for an initial speed of 0 mi/h (assuming vehicles stop at a ramp meter stop bar and then accelerate to reach the highway design speed). Since all the studies have level grade in common, the comparisons are only made for level grades. As shown in Table 3-38 and Table 3-39, SwashSim-based values are the most conservative for 80,000-lb trucks, TruckSim-based values are the most conservative for 53,000-lb trucks, and AASHTO-based values are least conservative. Note that some of the comparisons may not seem reasonable, since they are based on different types of trucks (as indicated in the table footnotes).

It is found from the literature that TruckSim-based values assume the most realistic current truck and roadway design characteristics, AASHTO-based values assume the least realistic, or outdated, truck characteristics, and Yang-based values assume the least realistic roadway design characteristics.

Table 3-38. Comparison of the recommended acceleration lengths (ft) for trucks on level grades (from literature)

	Literature review					
Merging speed (mi/h)	AASHTO* 1954	AASHTO* 2011	Deen**	Hardwood#	Fitzpatrick	Yang##
22	-	-	290	-	-	-
23	190	180	-	275	-	-
27	-	280	-	400	-	-
29	-	-	700	-	-	-
30	-	-	-	-	389	685
31	760	360	-	590	-	-
35	-	560	1240	800	529	975
39	-	720	-	1100	-	-
40	-	-	1820	-	691	1320
43	-	960	-	1510	-	-
45	-	-	-	-	875	1725
47	1170	1200	-	2000	-	-
50	-	1410	-	2490	1080	2190
53	1590	1620	-	3060	-	-
55	-	1790	-	3250	1307	2720
60	-	-	-	-	1556	3320
65	-	-	-	-	1826	-
70	-	-	-	-	2118	-
75	-	-	-	-	2431	-

*Passenger cars on grades of 2% or less; ** Semi-trailer trucks.
180 lb/hp trucks; ## 90th-percentile acceleration lengths for medium trucks.

Table 3-39. Comparison of the recommended acceleration lengths (ft) for trucks on level grades (from this study)

	This study			
Merging speed (mi/h)	TruckSim@	TruckSim@@	SwashSim@	SwashSim@@
22	-	-	-	-
23	233	291	193	309
27	355	452	290	468
29	-	-	-	-
30	-	-	-	-
31	468	617	412	672
35	612	827	582	962
39	863	1175	805	1360
40	-	-	-	-
43	1105	1537	1077	1851
45	-	-	-	-
47	1400	1982	1454	2569
50	1772	2509	1828	3344
53	2114	3041	2227	4178
55	2369	3440	2539	4898
60	-	-	-	-
65	-	-	-	-
70				
75	-	-	-	-

@ 53,000-lb trucks; @@ 80,000-lb trucks.

4. INTERCHANGE OPERATIONS PERFORMANCE

Relevant Performance Measures

This section discusses relevant performance measures for interchange operations when ramp metering is implemented and how those measures are impacted by interchange configuration.

Literature Review for Interchange/Ramp Metering Performance Measures

Although there are numerous performance measures found in the literature for evaluation of ramp metering strategies, there are no specific measures adopted in a formal analysis methodology for the combination of interchange design and ramp metering. What little discussion there is in the literature on this topic is summarized as follows.

Chien (2001) used two groups of MOEs to assess performance of ramp-metering algorithms for I-80 in New Jersey: headway, throughput, and total delay as MOEs for pre-timed metering; and capacity, headway, metering rate range, total throughput, and total delay as MOEs for demand/capacity metering. Chu et al. (2002) evaluated effectiveness of ramp-metering algorithms for a 6-mile segment of I-405 in California using generalized total vehicle travel time, average mainline travel time, average on-ramp waiting time, and average origin-destination travel time. Chu et al. (2004) further selected vehicle-hours traveled, average mainline travel time, and total on-ramp delay as performance measures to evaluate ramp-metering algorithms. Levinson and Zhang (2004) looked at inter-day and intra-day travel time variation to test effectiveness of ramp meters in the Twin Cities.

Horowitz et al. (2005) conducted a series of experiments in VISSIM to evaluate candidate metering strategies for I-210 using total passenger hours, passenger kilometers, average mainline speed, and average throughput as performance measures. Xie et al. (2011) performed a study to quantify the benefits of ramp metering along a stretch of US-95 in Las Vegas, NV. They considered several traditional performance measures, including average travel speed, speed standard deviation, inter-quartile speed range, travel time index, and buffer index. They also proposed two performance measures—delay volume (which quantifies temporal, spatial, and intensity of congestion) and average vehicle delay (with traffic volume as a weighting factor). Ishak et al. (2013) compared speeds, travel times, and level of service distributions of the before and after periods of ramp-metering deployment on two segments of I-12 in Louisiana.

Tian (2013) proposed throughput, maximum queue, 95th percentile queue, 50th percentile queue, average delay, queue flush rate, metering attainability (%) (i.e., percent of the time metering rate corresponds to a red interval that results in complete stop at the ramp-meter stop bar), ramp queue spillback (%), and queue blockage to signal (%) as output performance measures for ramps and throughput and average delay as output performance measures for freeway mainlines to evaluate integrated operation of ramp-metering system and diamond interchange. Su et al. (2014) used mean and standard deviation of total delay per distance traveled as a measure to compare the proposed coordination strategy with the current control plans in the field. Shehada and Kondyli (2019) evaluated ramp-metering algorithms in terms of travel time reliability, queue length, throughput, and congestion duration, considering demand distributions throughout a calendar year.

Highway Capacity Manual

The sixth edition of the Highway Capacity Manual (HCM) contains a relatively new analysis methodology for interchange ramp terminals (Chapters 23 and 34). This analysis method uses the service measure ‘Experienced Travel Time’, defined as follows:

$$ETT = \sum d_i + \sum E\,TT$$

where d_i is the control delay at each junction i encountered on the path through the facility, and *EDTT* is the extra distance travel time. Currently, ramp metering is not explicitly considered in this methodology (under the “Limitations of the Methodology” section (p. 27), “Ramp metering and its resulting spillback of vehicles into the interchange” is listed). However, the general methodology framework is amenable to the concept of defining a travel path all the way through the on-ramp to the point of freeway mainline entry and including any applicable ramp meter delay. This is a potential approach to evaluating interchange designs that include ramp metering. However, the ramp delay component is largely driven by the ramp metering rate as determined from freeway conditions, irrespective of the interchange design. So, another performance measure, or measures, is likely necessary to be able to assess the combined operational performance of the interchange and freeway mainline. Furthermore, it can be difficult to make “apples to apples” comparisons of interchange designs without controlling for certain parameters—for example, geographic footprint, or similar.

Performance Measures Considered in this Study

There are a variety of performance measures that may be used to assess the operational quality of an interchange with ramp metering. The following measures are likely to be meaningful in evaluating interchange operations with ramp metering.

Percent time in queue override mode

Increasing the metering rate, from the base value determined as a function of freeway mainline conditions, is generally undesirable. Advance queue override activations, due to the queue backing up to the adjoining arterial roadway, are especially undesirable from a freeway operations perspective, as it increases the likelihood of a flow breakdown. When this situation occurs, arterial operations (and safety) are also often adversely affected. Thus, it is necessary to avoid these kind of queue backups. The percentage of time that the meter gets set to its maximum rate (‘queue-flush’ mode) is arguably the most significant performance measure for interchange operational quality, when ramp metering is present.

Ramp meter throughput

The ramp meter throughput, or queue discharge rate, is measured from passage detectors immediately downstream of the ramp meter stop bar. This measure is directly correlated with the percent time in queue override mode and ramp meter delay measures. Higher absolute values are not necessarily better than lower values. The more relevant measure is the percentage of time the metering rate deviates from the base value (the value based only on the freeway mainline conditions). The major reason for this deviation is the activation of queue override adjustments.

High-occupancy bypass vehicles can also result in adjustments to the base metering rate, but this topic is beyond the scope of this document.

Ramp meter delay

Ramp meter delay is the additional travel time for a vehicle to travel from entry into the interchange area to the freeway merge point, versus if no ramp meter was installed or active. In general, higher metering rates result in less delay. The challenge is to avoid an increased metering rate from queue override. If the base metering rate can be maintained, then the delay is directly proportional to the difference between the on-ramp arrival rate and the metering rate. For any given freeway mainline conditions, the base metering rate will vary slightly as a function of the metering algorithm employed. Of course, with any type of traffic signal, there is some driver expectation for delay. The focus with this measure specific to ramp metering should be on the extremes. Very low values of delay is an indicator that the ramp metering system may not be warranted for those conditions. Excessive ramp meter delay can lead to significant public backlash, such as happened in Minneapolis and led to the 'ramp metering experiment'[1].

Interchange volume throughput

It is important to ensure that the interchange configuration can accommodate the expected vehicular volume demands, also considering the specific origin-destination travel paths. The capacity of the arterial roadway is generally governed by the signalized intersections (except for full cloverleaf designs, which do not contain signals within the interchange area). The methods of the Highway Capacity Manual (TRB, 2016) can be used to determine the capacity of signalized intersections. Signalized intersection capacity is largely a function of number of lanes per movement and the ratio of green time to cycle length assigned to each movement. Designs with fewer timing stages will generally be more efficient due to less lost time. Some interchange configurations are also more space-efficient and can accommodate more travel lanes within the same right-of-way. The capacity of the on-ramp is simply a function of the metering rate(s).

It is also important to be cognizant of upstream or downstream restrictions in flow. If the on-ramp queue is backing up to the arterial roadway, the capacity will not be limited by the signalized intersection design, but rather the ramp roadway capacity. Capacity restrictions on the arterial just downstream of the interchange area may complicate operations within the interchange area. Additionally, the interchange cannot serve more traffic than is able to enter the interchange area. Thus, the interchange design should also be matched to the upstream and downstream conditions of the interchange area. This topic is beyond the scope of this document. Likewise, topics such as "metering" upstream of the interchange area in order to limit the feeding rate of traffic to the on-ramp are not addressed.

Average travel speed

There are a number of origin-destination pairs through the interchange:

- Arterial entry to arterial exit
- Arterial entry to near on-ramp

[1] https://en.wikipedia.org/wiki/Ramp_meter#Minneapolis%E2%80%93Saint_Paul_ramp_meter_experiment. http://www.dot.state.mn.us/rampmeter/finalreport.html

- Arterial entry to far on-ramp
- Freeway exit to arterial exit
- Freeway exit to freeway entry (but rare)

The arterial entry to exit movement is usually only influenced by the arterial signal delay, but on-ramp queue backups into the intersection area can create additional delay. Travel paths that include the on-ramp will have much lower average speeds than the arterial through movement due to the much higher average delay of the ramp meter than the arterial signal(s).

Experimental Design

With the basis for the objective of this project being planning-level guidance, the focus of the above measures is on relative differences rather than absolute values. Thus, the central element of the experimental design is to simulate a given set of traffic and control characteristics across several common interchange forms to identify relative differences in performance and advantages and disadvantages of each as a function of ramp-metering operations.

Interchange Configurations

The following interchange forms were considered in this study:

- Diamond
- DDI
- SPUI
- Partial cloverleaf (ParClo)
- Full cloverleaf with a collector-distributor (FullClo with C-D)
- Full cloverleaf without a collector-distributor (FullClo w/o C-D)

The first three interchange configurations do not include any loop ramps, whereas the other three types include two or four loop ramps. For a given interchange form (Diamond, DDI, SPUI, etc.), geometric, traffic, and control characteristics are specified.

The geometric configuration of these interchange forms is based on existing interchanges in Florida and/or common configurations for the given form. Other interchanges of these forms, with ramp metering installations, throughout the country can be viewed with the Google Earth KMZ file at https://github.com/swash17/FDOT_IRM.

Diamond Interchange

This interchange form is modeled after the diamond interchange configuration that was recently replaced with a diverging diamond configuration, at I-75/University Parkway (Sarasota; Latitude/Longitude: 27°23'19.13"N/82°26'55.64"W). The geometry of the diamond interchange defined within the simulation program is illustrated in Figure 4-1 and Figure 4-2.

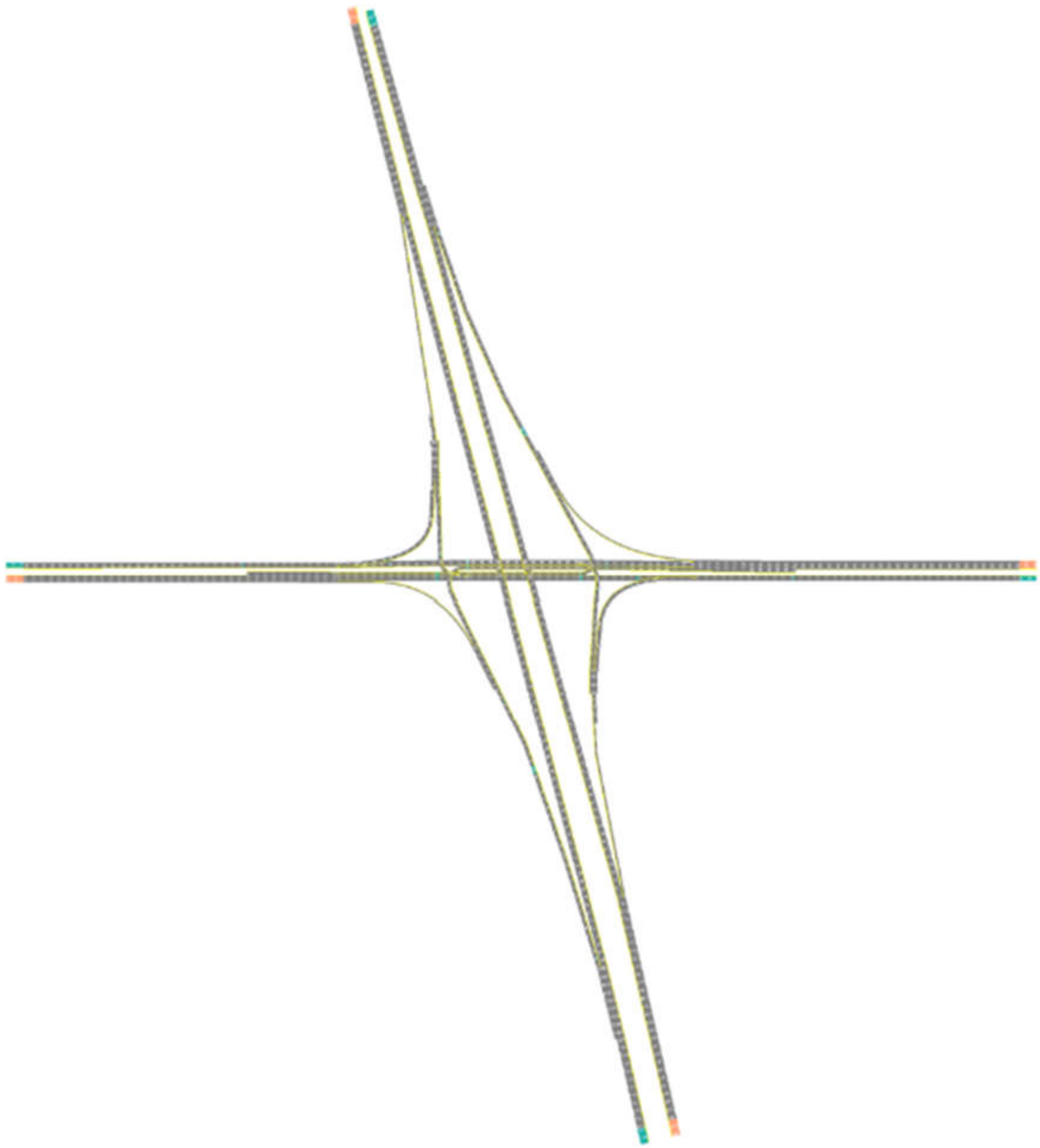


Figure 4-1. Diamond interchange previously at I-75/University Parkway: zoomed-out view

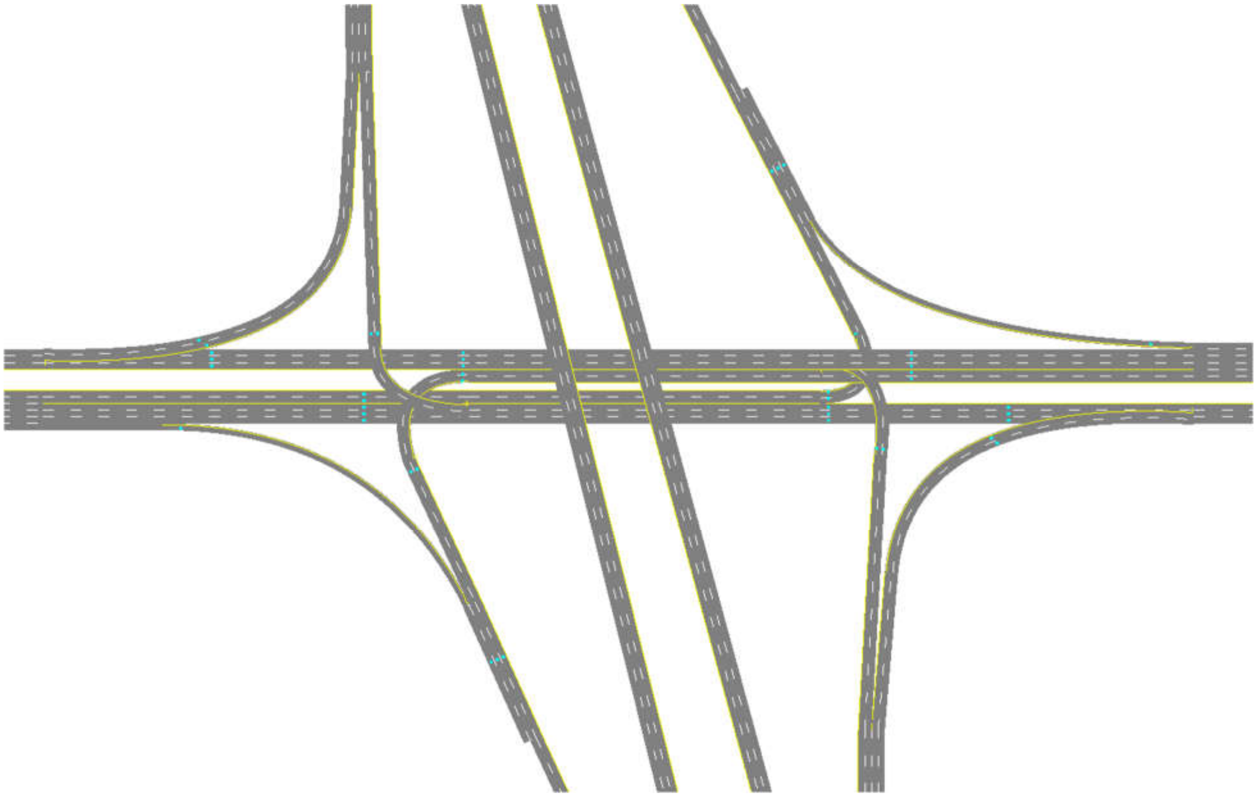


Figure 4-2. Diamond interchange previously at I-75/University Parkway: zoomed-in view

Diverging Diamond Interchange (DDI)

This interchange form is modeled after the diverging diamond interchange configuration currently in place at I-75/University Parkway (Sarasota; Latitude/Longitude: 27°23'19.13"N/82°26'55.64"W). The geometry of the diamond interchange defined within the simulation program is illustrated in Figure 4-3 and Figure 4-4.

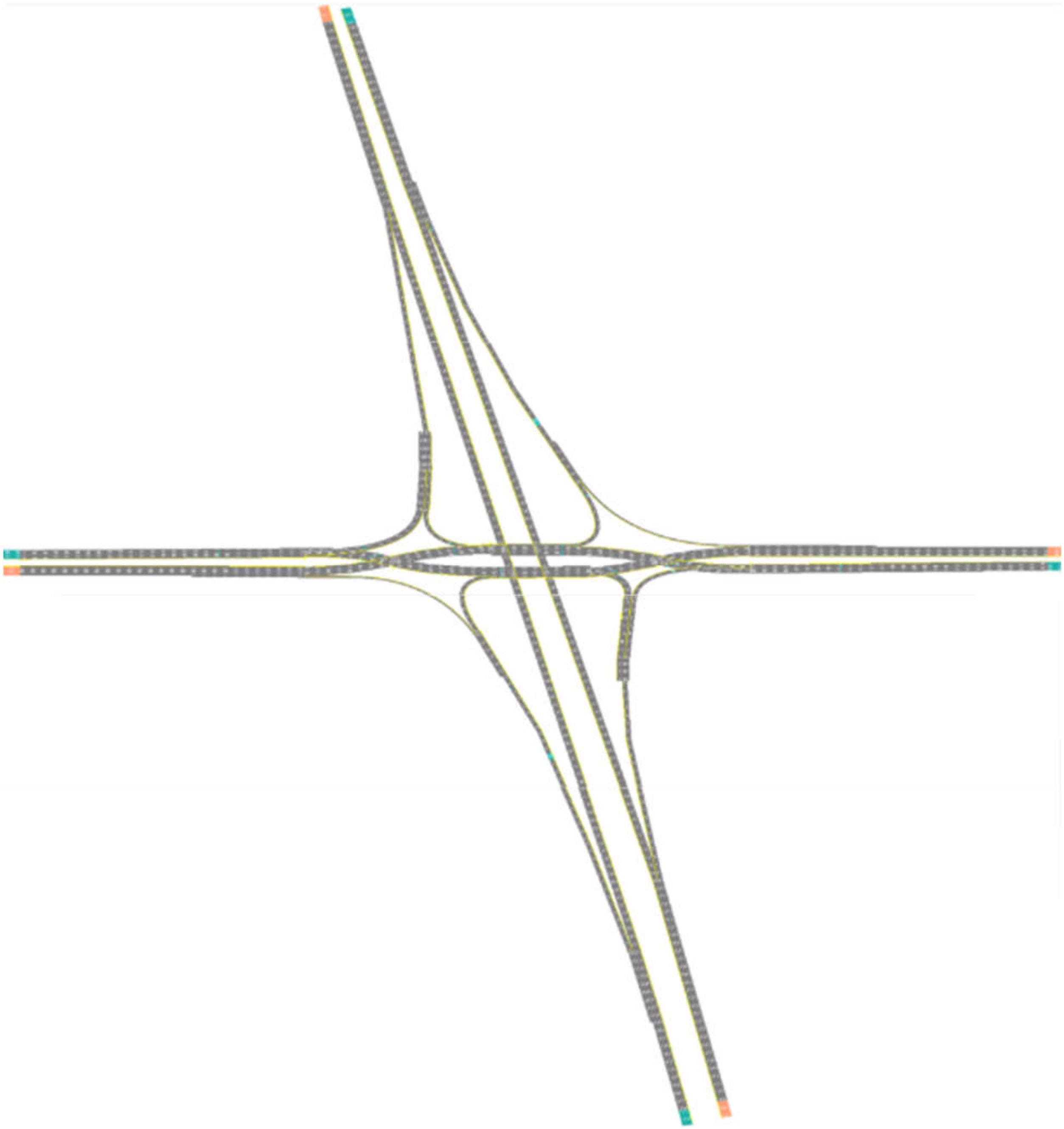


Figure 4-3. I-75/University Parkway DDI: zoomed-out view

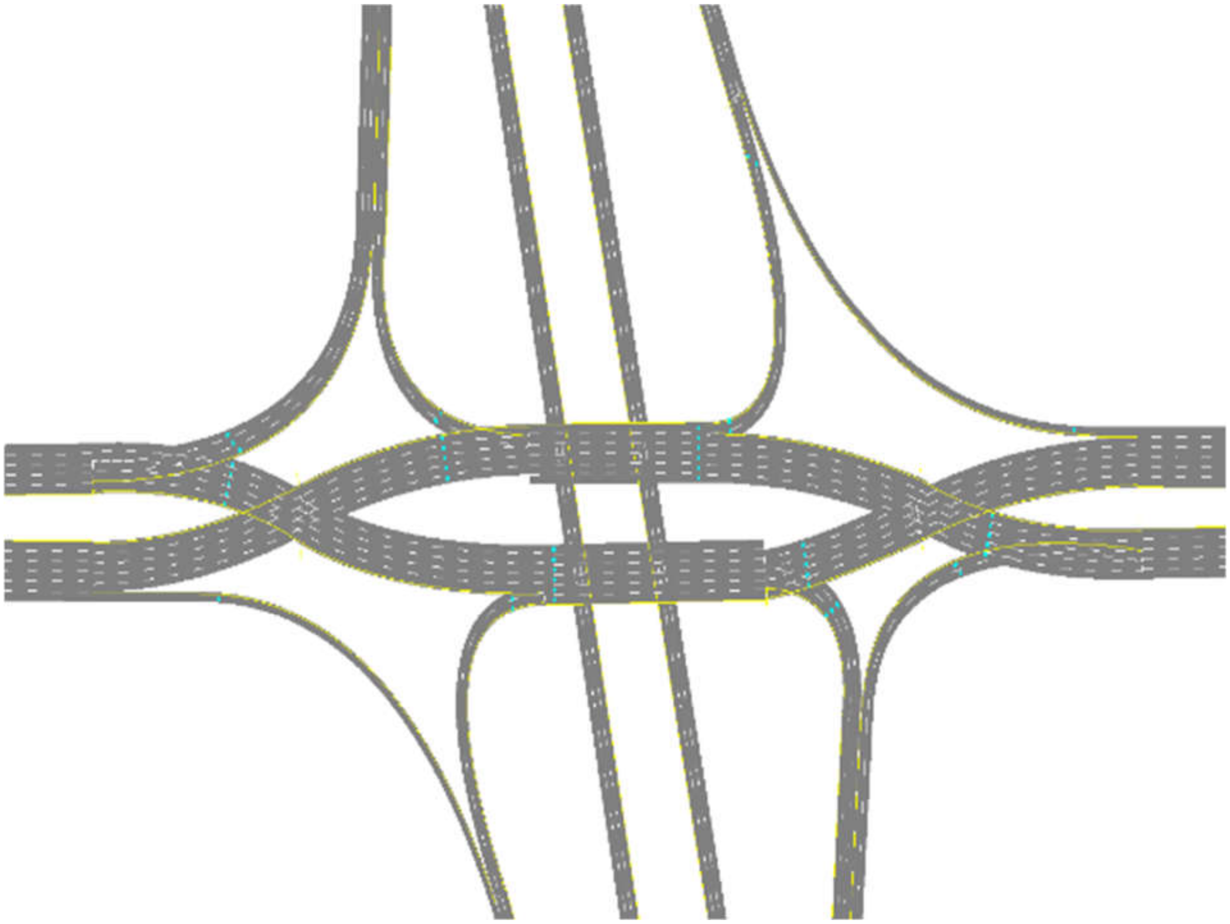


Figure 4-4. I-75/University Parkway DDI: zoomed-in view

The DDI design is intended to improve on the operational efficiency of a diamond interchange by making the left-turn movements function, essentially, like right-turn movements. Consequently, the intersection signal phasing pattern and access to the on-ramps is simplified. This may also lead to other advantages over the diamond interchange when ramp metering is present.

Single Point Urban Interchange (SPUI)

This interchange form is generally modeled after the SPUI configuration currently in place at I-295/US-90 (Jacksonville; Latitude/Longitude: 30°17'13.02"N/ 81°31'18.27"W). The geometry of the SPUI defined within the simulation program is illustrated in Figure 4-5 and Figure 4-6.

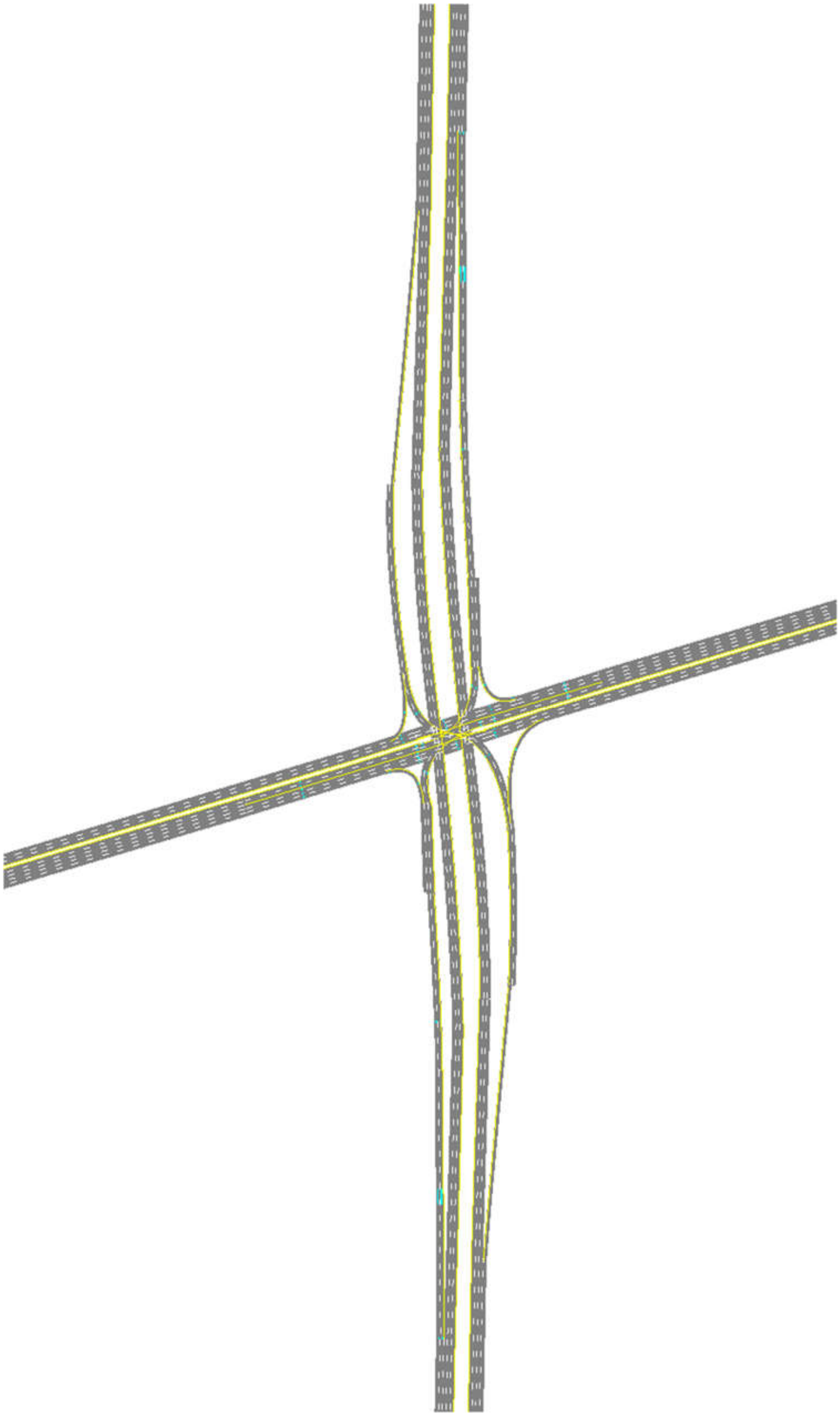


Figure 4-5. Single-point urban interchange (SPUI): zoomed-out view

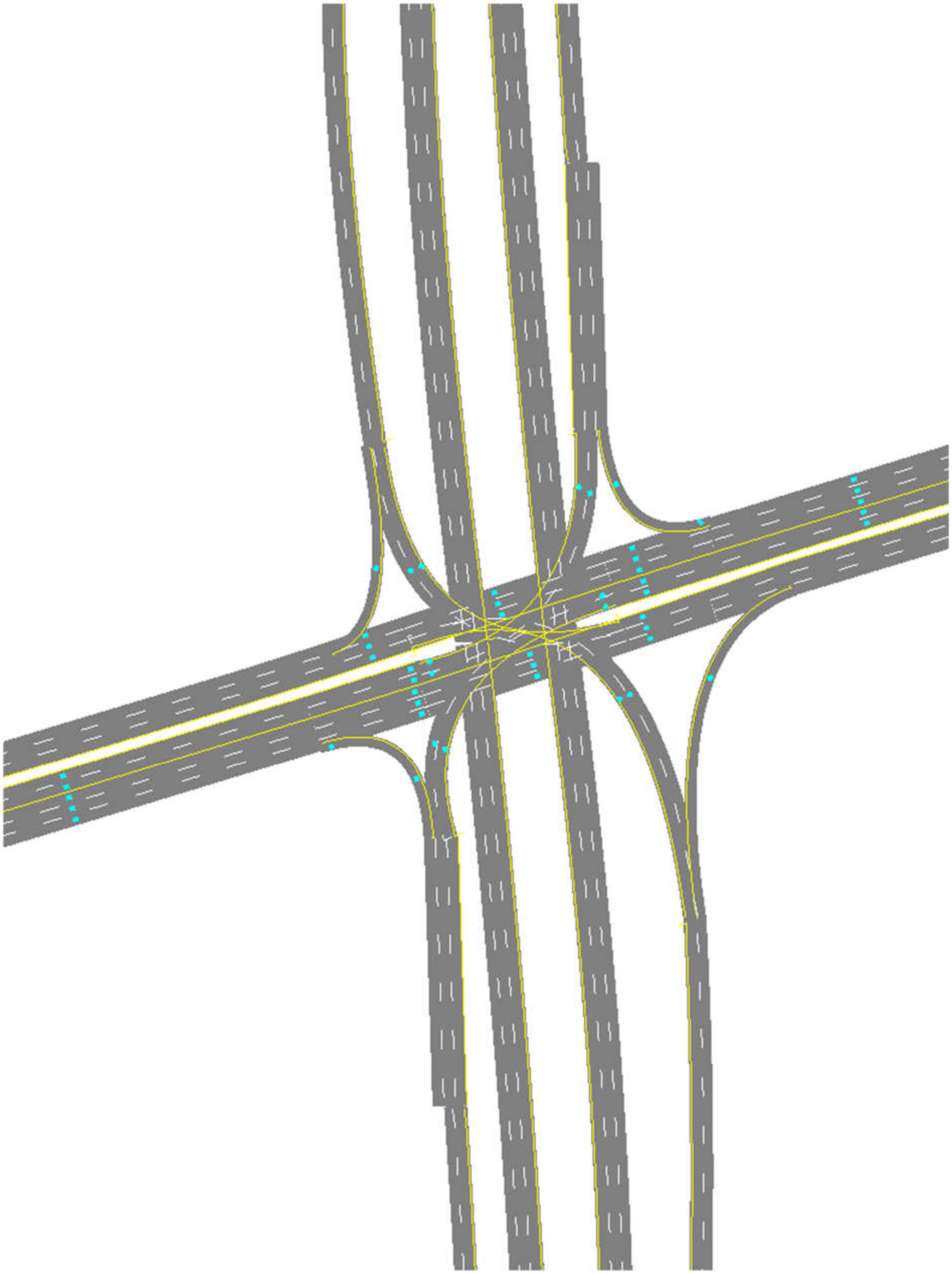


Figure 4-6. Single-point urban interchange (SPUI): zoomed-in view

Partial Cloverleaf Interchange (ParClo)

This interchange form is generally modeled after the ParClo configuration currently in place at Tierra Moorpark Freeway/Rejada Rd (Los Angeles; Latitude/Longitude: 34°16'0.36"N/118°51'4.31"W). This design is also sometimes referred to as a ParClo-A configuration because the loop ramps are on-ramps. A ParClo-B configuration uses loop ramps for off-ramps. The geometry of the ParClo defined within the simulation program is illustrated in Figure 4-7 and Figure 4-8.

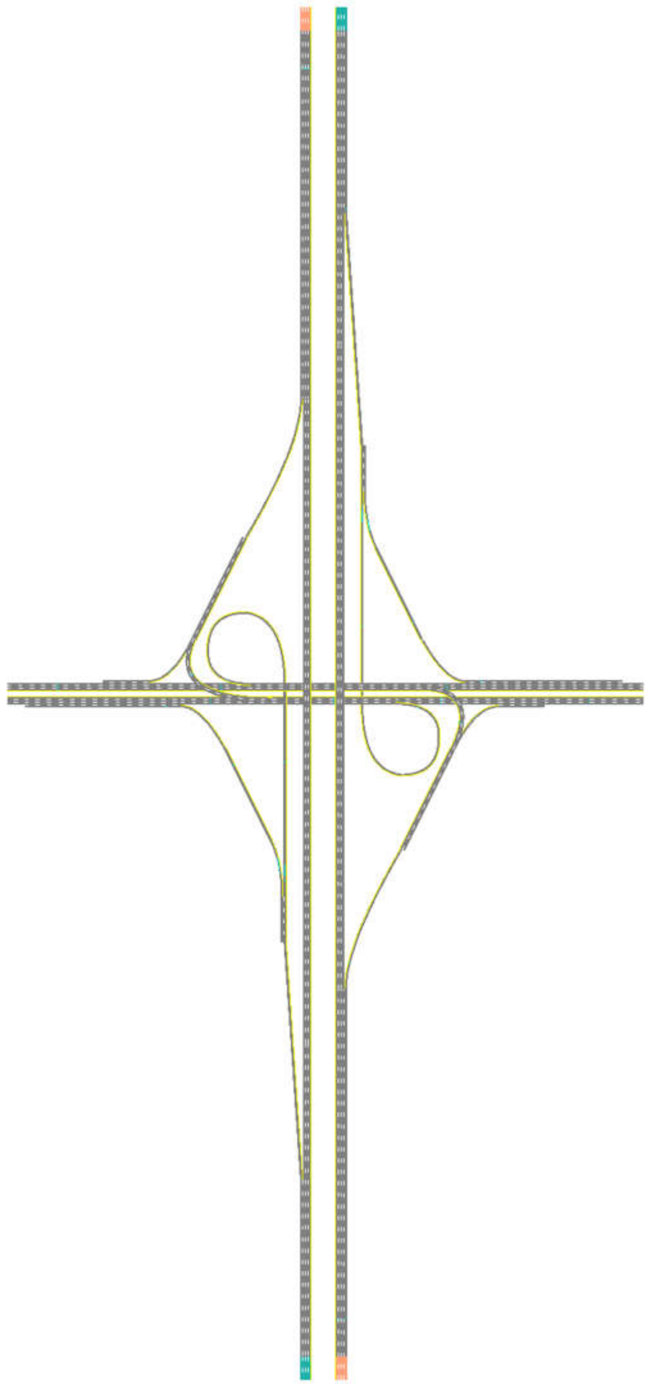


Figure 4-7. Partial cloverleaf interchange: zoomed-out view

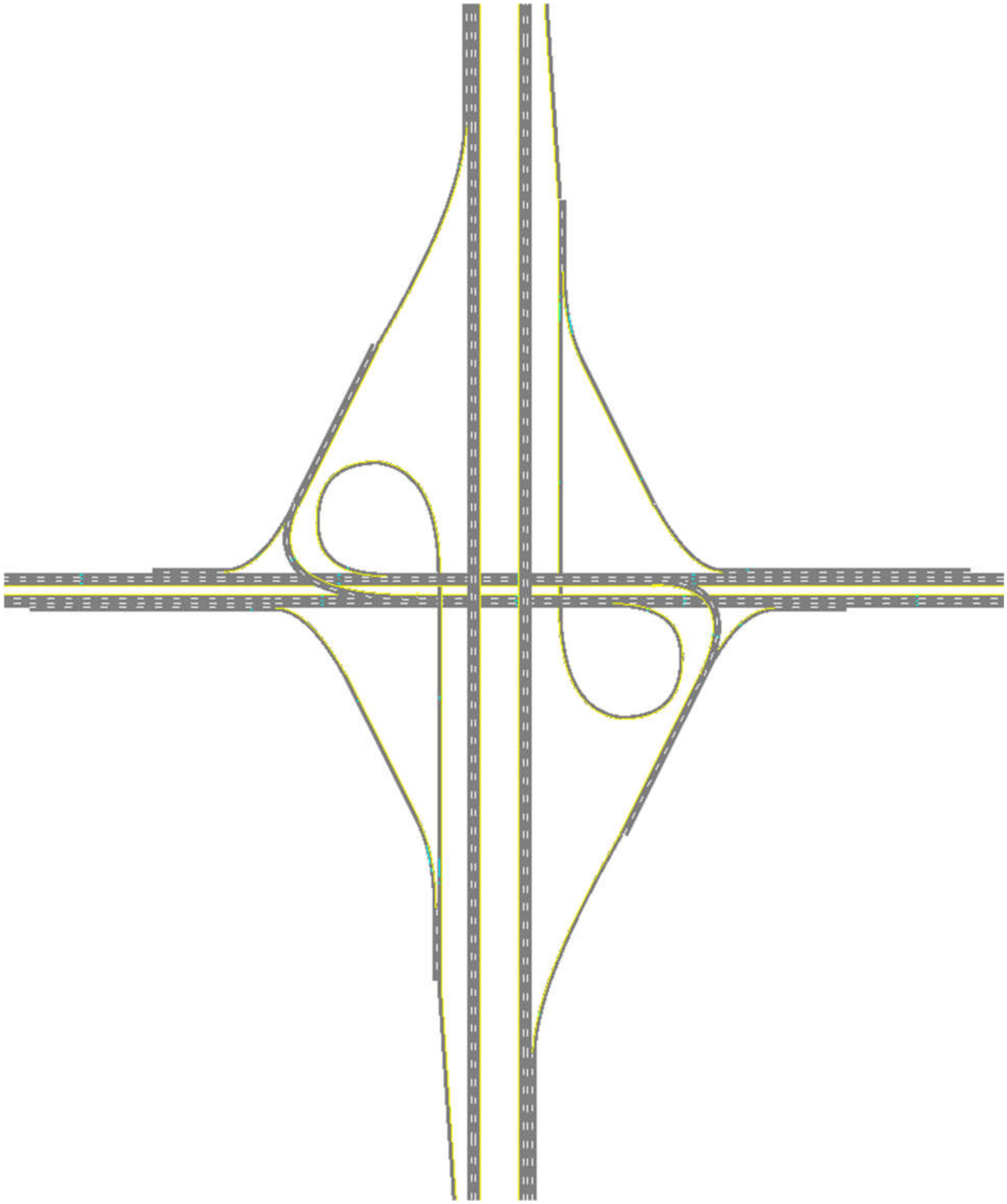


Figure 4-8. Partial cloverleaf interchange: zoomed-in view

Full Cloverleaf Interchange, with Collector-Distributor

This interchange form is modeled after the full cloverleaf interchange in Fort Lewis, WA (Latitude/Longitude: 47° 6'15.13"N, 122°35'18.17"W). A key feature of this design is that it includes a collector-distributor roadway for the freeway weaving movements. Thus, the ramp meter for the loop on-ramp is placed downstream near the connection with the on-ramp lane for the arterial right-turn movement. This design provides more queue storage for the loop-ramp movement than full interchange design without a collector-distributor; however, care must be taken to avoid a situation where the queue blocks the loop-ramp exit movement. The geometry of this interchange defined within the simulation program is illustrated in Figure 4-9 and Figure 4-10.

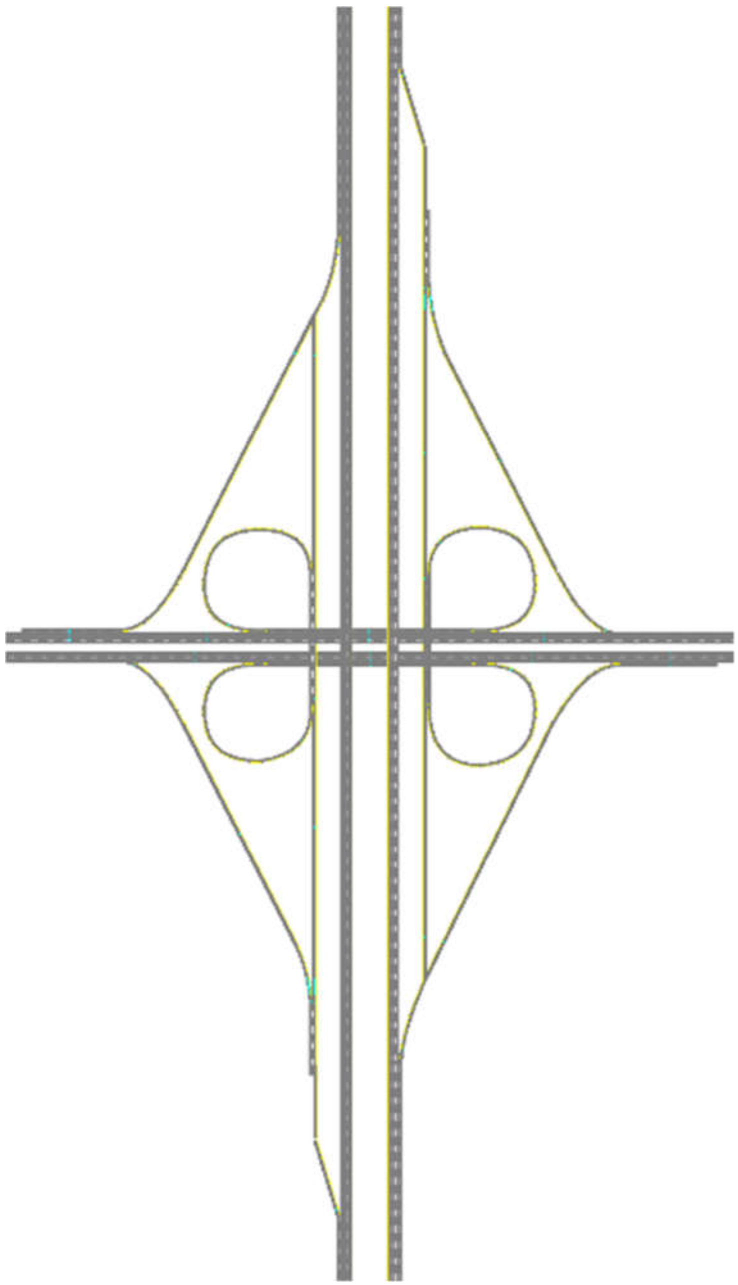


Figure 4-9. Full cloverleaf interchange with collector-distributor: zoomed-out view

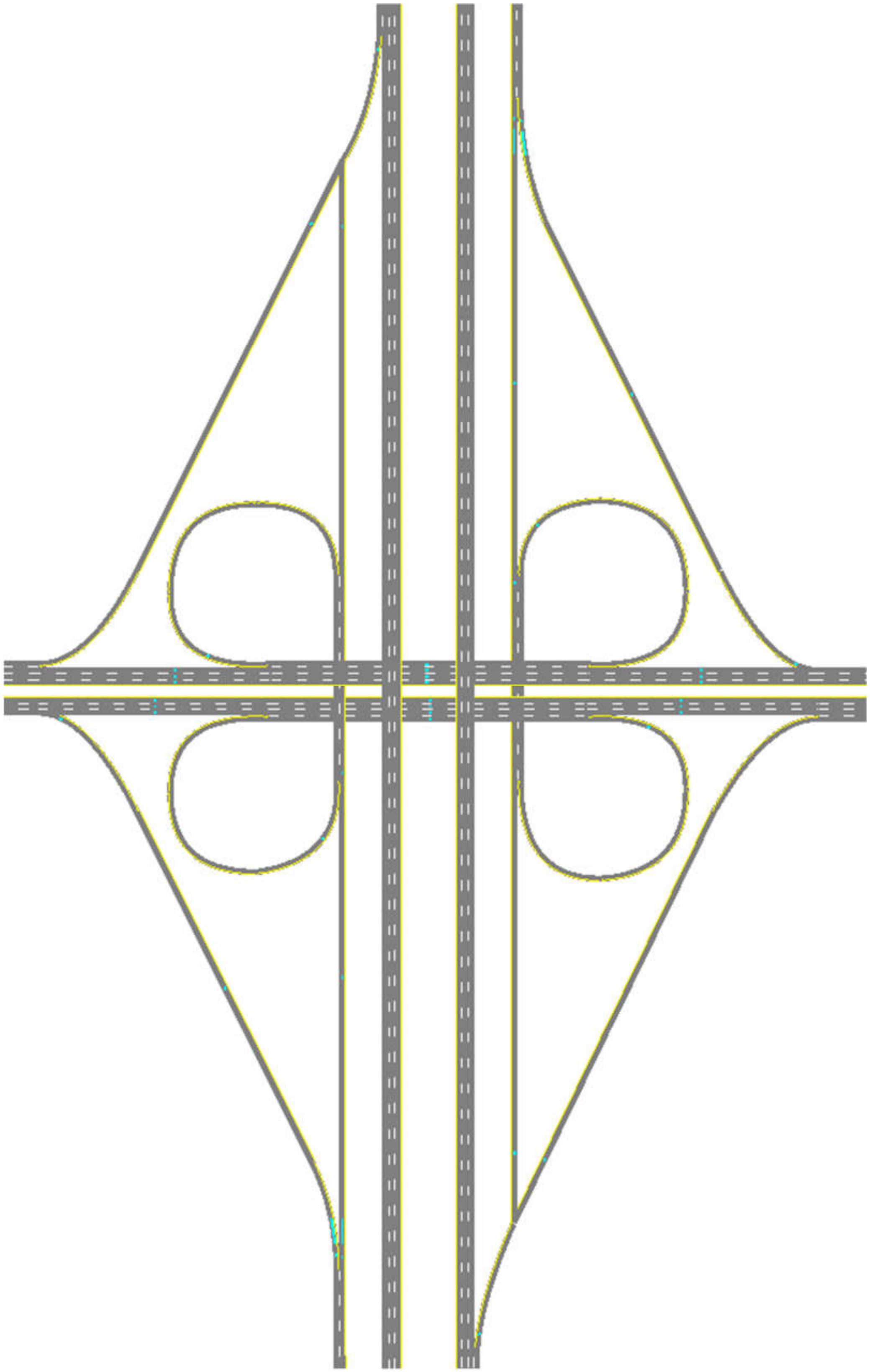


Figure 4-10. Full cloverleaf interchange with collector-distributor: zoomed-in view

Full Cloverleaf Interchange, without Collector-Distributor

This interchange form is modeled after the full cloverleaf interchange configuration that was recently replaced with a diverging diamond configuration, at SR-836/NW 27th Ave (Miami; Latitude/Longitude: 25°47'5.32"N/80°14'22.20"W). This interchange configuration is also very similar to one in Mountain View, CA (37°22'41.36"N, 122° 4'3.53"W). A key feature of this design is that it does not include a collector-distributor roadway for the freeway weaving movements. Thus, the ramp meter for the loop on-ramp must be placed on the loop portion of the ramp. The geometry of this interchange defined within the simulation program is illustrated in Figure 4-11 and Figure 4-12.

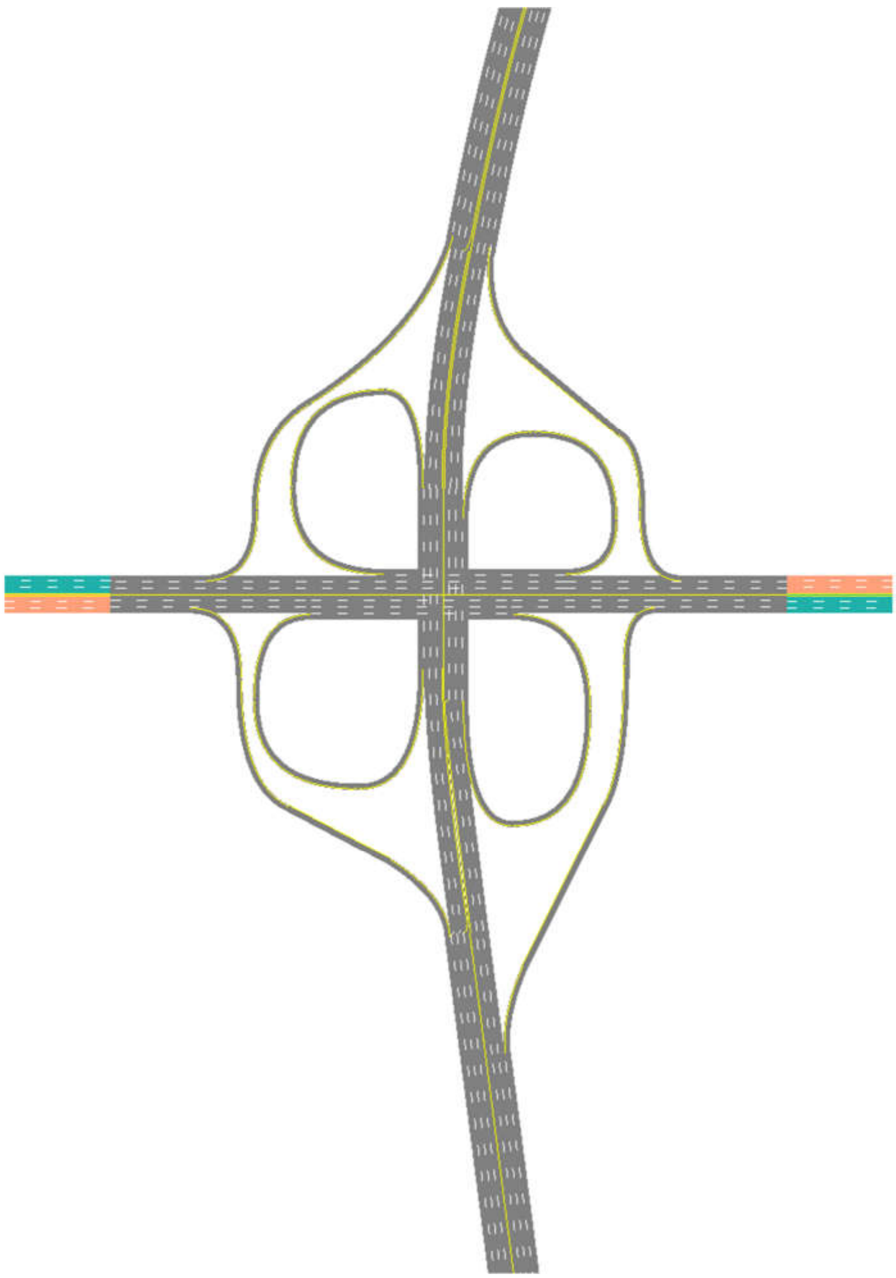


Figure 4-11. Full cloverleaf interchange without collector-distributor: zoomed-out view

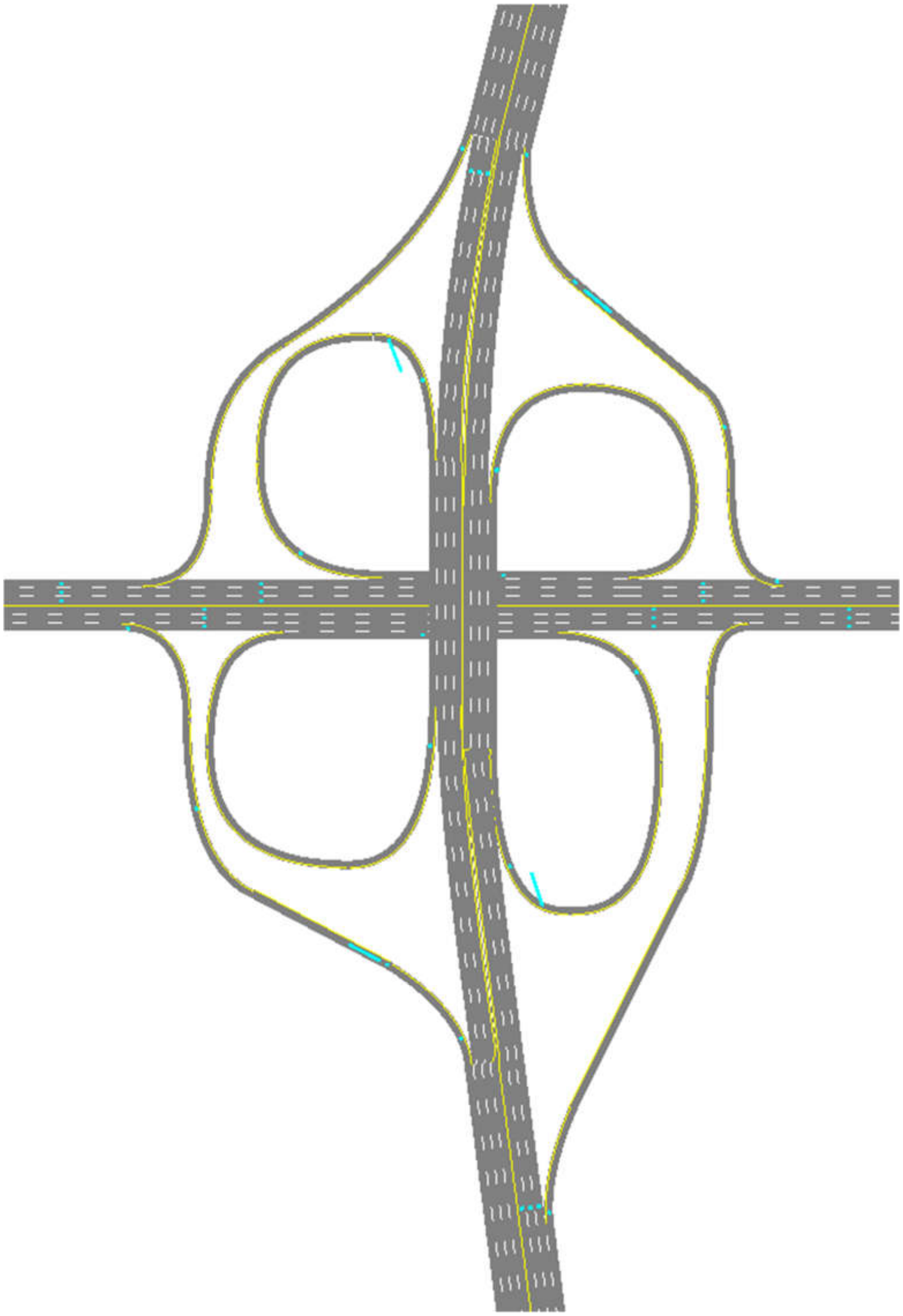
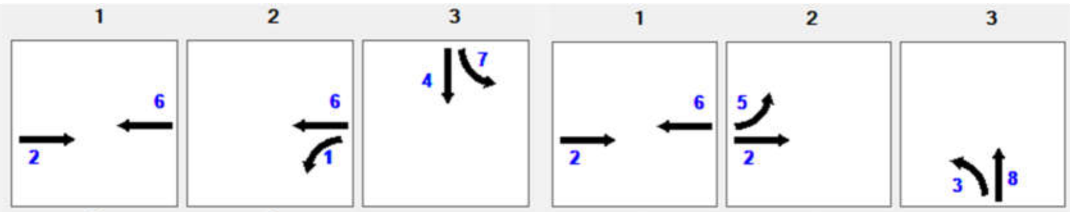
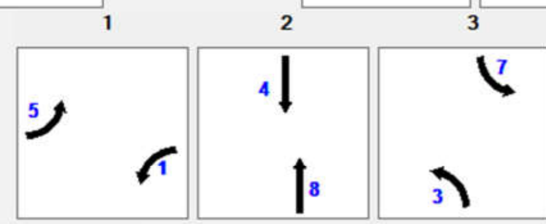


Figure 4-12. Full cloverleaf interchange without collector-distributor: zoomed-in view

Arterial Signal Control Characteristics

The signalized intersection phasing/timing plan will impact the vehicle throughput capability of the arterial and the arrival pattern of vehicles to the on-ramp. The signal phasing pattern is set according to standard practice for the given interchange form and/or consistent with FDOT-supplied signal timing plans. The phase times are set to approximately minimize vehicular delay for the specified scenario demand volumes, using pre-timed control, and include coordination in the peak travel direction. The phasing plan for each interchange configuration is shown in Table 4-1.

Table 4-1. Signal-timing pattern for each interchange configuration in simulation

	Timing Stage Movements	
	West arterial intersection	**East arterial intersection**
Diamond		
DDI		
SPUI		
ParClo		
FullClo	**Not Applicable**	

For the ParClo and SPUI interchanges, the arterial runs north-south in the simulation network configuration. The figures as displayed in the previous section are rotated for display purposes. Thus, the intersection labeled ‘west’ is actually the ‘south’ intersection, and likewise, the intersection labeled ‘east’ is actually the ‘north’ intersection.

Since the mapping of the dual-ring phases to the movements at a DDI intersection is unconventional, Figure 4-13 is provided for clarification.

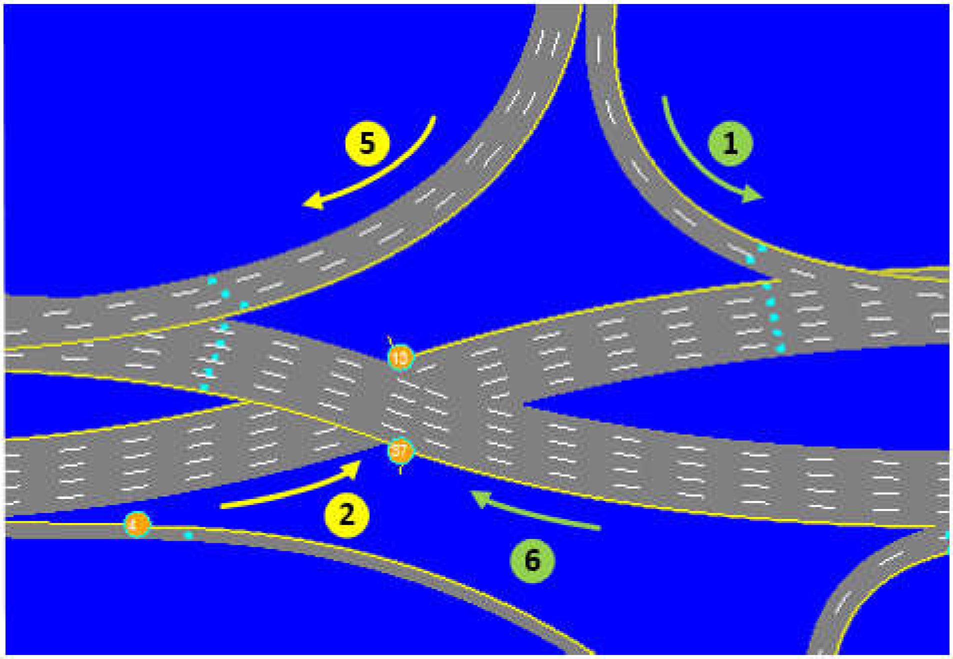


Figure 4-13. Phase-movement assignment at DDI intersection.

The phase times and cycle lengths vary by scenario and are provided in the experimental design section of this document.

Ramp Metering Control Characteristics

Ramp metering is implemented at all on-ramps (2 or 4 ramps, depending on interchange design). The general ramp meter and detector configuration is illustrated in Figure 4-14.

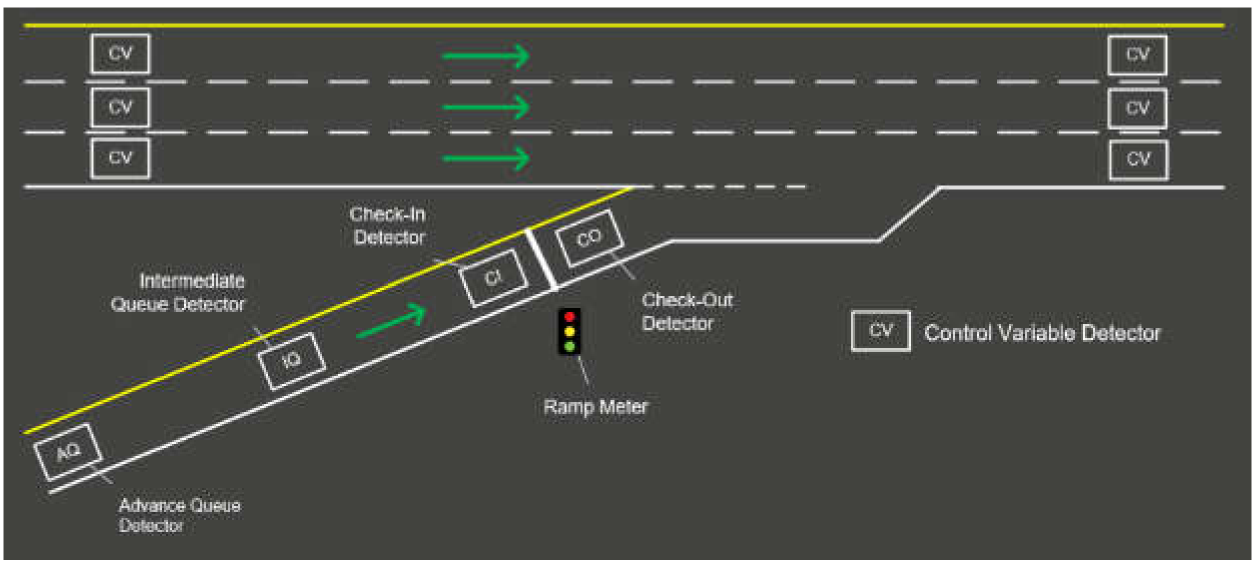


Figure 4-14. General ramp metering setup.

In order to reduce simulation run times, a pre-timed ramp-metering scheme was employed. Consequently, it was necessary to only simulate the minimum traffic volume on the freeway mainline that would generate the desired off-ramp demand volumes. The base metering rate employed in these simulation runs was determined from previous experiments, in which a demand/capacity ramp-metering algorithm was utilized. The metering rates employed in the simulation runs are given in Table 4-2.

Table 4-2. Ramp metering timing parameters

Ramp Meter Timing Parameters	**Rate (veh/h)**
Minimum	240
Maximum	900
Base[1]	240/550
Additional[2]	180
Queue-flush rate[3]	= Maximum

[1] The base rate is the metering rate as determined from the freeway mainline traffic conditions, without consideration of adjustments as a function of on-ramp queuing conditions. This value is constrained by the minimum and maximum metering rates. The first value corresponds to the 'high demand' freeway mainline traffic condition, whereas the second value corresponds to the 'medium demand' freeway mainline traffic condition, which corresponds to approximate v/c ratios of 0.85 and 0.60, respectively.
[2] The additional rate is added to the base rate when the intermediate on-ramp queue detector reaches the predefined occupancy threshold (default value from Table 3.1 of FDOT, 2009 reference) for the specified aggregation interval.
[3] The queue-flush rate is initiated when the advance queue detector reaches the predefined occupancy threshold (default value from Table 3.1 of FDOT, 2009 reference) for the specified aggregation interval. This rate is set equal to the maximum metering rate.

Since pre-timed metering is used in this project, freeway mainline detectors are unnecessary. The stop bar location for the on-ramp is based on acceleration distance for commercial trucks, as discussed in Chapter 3.

The ramp metering controller is set up such that a separate phase is assigned to each metered lane. Thus, for an on-ramp with two lanes, for example, the total metering rate would be divided in half for each lane. There is a key distinction, however, between the non-loop ramp interchange configurations (Diamond, DDI, SPUI) and the loop-ramp interchange configurations (ParClo, FullClo with C-D, FullClo w/o C-D) in terms of how the queue detectors interact with the ramp controllers.

For the first group (non-loop), all intermediate and advance queue detectors for a given on-ramp tie into the override checks for each phase of the controller. Figure 4-15 shows the detector setup for the Diamond interchange southbound on-ramp. If any one of the intermediate or advance queue detectors (circled) meet their corresponding occupancy thresholds, the metering rate for both phases (lanes) of the on-ramp will be adjusted accordingly.

Figure 4-15. Diamond interchange southbound ramp metering detector setup.

For the second group (loop), the intermediate and advance queue detectors are phase (lane) specific. Figure 4-16 shows the detector setup for the ParClo interchange east/northbound on-ramp. If the intermediate or advance queue detectors (circled) for a given lane meet their corresponding occupancy thresholds, the metering rate for only that respective phase (lane) of the on-ramp will be adjusted accordingly.

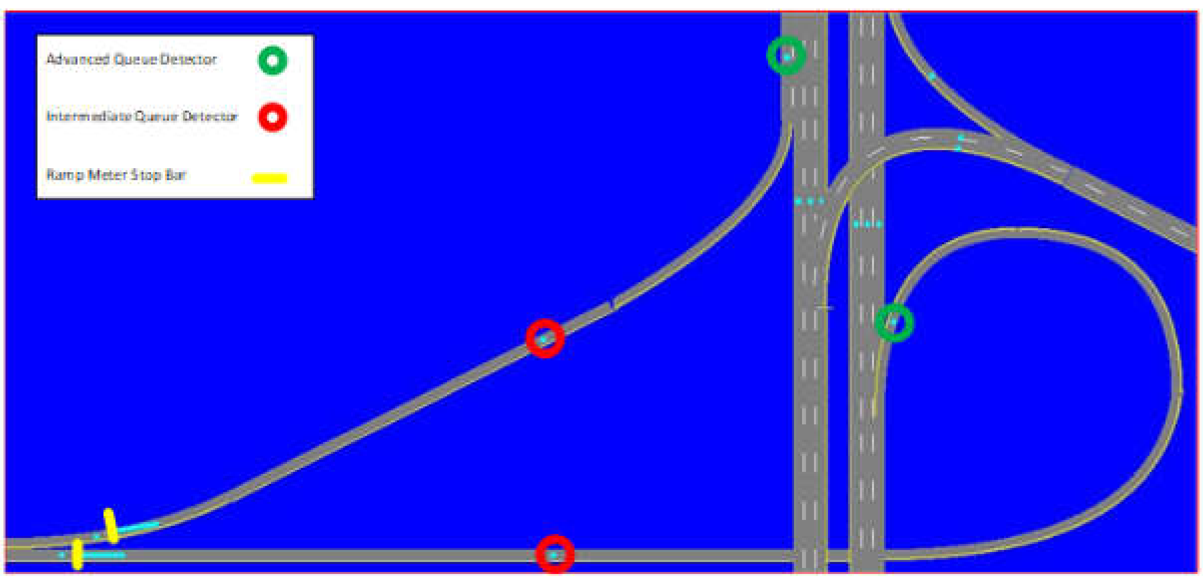


Figure 4-16. ParClo interchange east/northbound ramp metering detector setup.

The ramp-metering control configuration is described in Table 4-3.

Table 4-3. Ramp metering control configurations

	Diamond		DDI		SPUI		ParClo			
Ramp Location/Type	SB On-Ramp	NB On-Ramp	SB On-Ramp	NB On-Ramp	SB On-Ramp	NB On-Ramp	SB Direct On-Ramp	NB Direct On-Ramp	SB Loop On-Ramp	NB Loop On-Ramp
Total Ramp Length (ft)	2893	2685	3078	2746	1869	1872	2233	2233	3138	3138
Metered Lanes	2	2	2	2	2	2	1	1	1	1
Stop-bar/ramp meter Location[1] (ft)	1617	1258	1531	1226	1416	1422	1371	1371	1326	1326
Intermediate Queue Detector Location[2] (ft)	485	555	570	547	503	503	500	500	500	500
Advance Queue Detector Location[2] (ft) Left Turn/Right Turn	893 1297	901 1436	1078 1358	1083 1459	1290 1461	1290 1312	935	935	1822	1822
Queue Storage Space Available (lane-ft) Total/Left/Right	2738 2054 2178	2829 2068 2377	3183 2448 2273	3306 2444 2334	3046 2859 3014	3048 2859 3016	935	935	1822	1822

[1] Location denotes the distance upstream of the on-ramp merge point. This distance also corresponds to the acceleration distance. The presence detector is located immediately upstream of the stop bar.
[2] Distance upstream of the ramp meter stop bar.

Table 4-4. Ramp metering control configurations (cont.)

	FullClo with Collector-Distributor				FullClo without Collector-Distributor			
Ramp Location/Type	SB Direct On-Ramp	NB Direct On-Ramp	SB Loop On-Ramp	NB Loop On-Ramp	SB Direct On-Ramp	NB Direct On-Ramp	SB Loop On-Ramp	NB Loop On-Ramp
Total Ramp Length (ft)	1935	1968	2803	2832	792	873	1228	1128
Metered Lanes	1	1	1	1	1	1	1	1
Stop-bar/ramp meter Location[1] (ft)	687	701	690	698	241	205	184	136
Intermediate Queue Detector Location[2] (ft)	551	575	551	531	273	336	501	499
Advance Queue Detector Location[2] (ft)	1251	1274	949	913	543	658	846	814
Queue Storage Space Available (lane-ft) Left/Right	1251	1274	949	913	543	658	846	814

[1] Location denotes the distance upstream of the on-ramp merge point. This distance also corresponds to the acceleration distance. The presence detector is located immediately upstream of the stop bar.

[2] Distance upstream of the ramp meter stop bar.

Traffic Characteristics

Varying arterial demand volumes, truck percentages, and arterial directional splits are used in the experimental design. The selected values were based on the desire to represent the following conditions:

- High and medium levels of arterial demand volume
- Low/medium and medium/high levels of truck percentage in the traffic stream
- Balanced, median, and 90th percentile levels of direction split of demand volume on the arterial
- High and medium levels of freeway demand volume, reflected by the metering rate

The specific settings for the above factors are given in Table 4-5.

Table 4-5. Experimental design traffic factors and levels

Variable	Levels
Traffic Demand Volume (Total Both Arterial Directions)[a] (veh/h)	3600 2800
Arterial Mainline Directional Split[b]	65/35 55/45 50/50
Truck Percentage (50/50 split of single-unit trucks and tractor-trailers)	4 8
Base ramp metering rates[d] (veh/h)	240[c] 550[d]

[a] Freeway off-ramp volume (based roughly on provided Ocala interchange volume count data)
[b] The eastbound (EB) or northbound (NB) travel direction of the arterial was designated as the peak direction
[c] Minimum, corresponding to high freeway mainline demand volume
[d] Medium, corresponding to medium freeway mainline demand volume

The high level of arterial traffic demand is intended to provide steady queuing at the arterial signals, but not oversaturated conditions (cycle failures). These traffic conditions also lead to significant on-ramp queuing, such that the advance queue detector would occasionally reach its threshold conditions for triggering the 'queue flush' metering rate.

The resulting number of unique scenarios for this experimental design is **24** [**2** (demand volume levels.) × **3** (directional splits) × **2** (truck percentages) × **2** (metering rates)]. A full enumeration of these scenarios is given in Table 4-6. Six replications were simulated for each scenario, which was sufficient to meet a 90% confidence level for target entry volumes.

Table 4-6. Experimental design traffic scenario enumeration

Scenario	Two-way Arterial Demand Volume (veh/h)	Arterial Directional Split	Truck %	Metering Rate Level
1	3600	65/35	4	Low
2	3600	65/35	4	Medium
3	3600	65/35	8	Low
4	3600	65/35	8	Medium
5	3600	55/45	4	Low
6	3600	55/45	4	Medium
7	3600	55/45	8	Low
8	3600	55/45	8	Medium
9	3600	50/50	4	Low
10	3600	50/50	4	Medium
11	3600	50/50	8	Low
12	3600	50/50	8	Medium
13	2800	65/35	4	Low
14	2800	65/35	4	Medium
15	2800	65/35	8	Low
16	2800	65/35	8	Medium
17	2800	55/45	4	Low
18	2800	55/45	4	Medium
19	2800	55/45	8	Low
20	2800	55/45	8	Medium
21	2800	50/50	4	Low
22	2800	50/50	4	Medium
23	2800	50/50	8	Low
24	2800	50/50	8	Medium

There are total of six timing plans for the arterial intersection(s). These six plans correspond to the different combinations of demand (2 levels) and directional split (3 levels). Timing plans were not varied due to heavy vehicle percentage. Green times used for the timing plans are shown in the following tables.

Table 4-7. Displayed green times for diamond interchange

		West Intersection						East Intersection				
		Timing Stage Green Time (s)						Timing Stage Green Time (s)				
Timing Plan ID	Exp. Design Scenarios	1	2	3	Cycle Length (s)	Crit. Mvmt. v/c		1	2	3	Cycle Length (s)	Crit. Mvmt. v/c
1	1-4	79	15	11	120	0.60		57	36	15	120	0.44
2	5-8	61	24	10	110	0.60		59	24	12	110	0.51
3	9-12	56	19	10	100	0.54		56	19	10	100	0.54
4	13-16	69	13	13	110	0.49		49	33	17	110	0.37
5	17-20	57	16	12	100	0.46		50	21	14	100	0.42
6	21-24	48	16	11	90	0.45		48	16	11	90	0.45

Table 4-8. Displayed green times for diverging diamond interchange

		West Intersection					East Intersection			
		Timing Stage Green Time (s)					Timing Stage Green Time (s)			
Timing Plan ID	Design Scenarios	1	2	Length (s)	Crit. Mvmt. v/c		1	2	Cycle Length (s)	Crit. Mvmt. v/c
1	1-4	41	19	90	0.47		38	22	90	0.44
2	5-8	32	23	85	0.47		29	26	85	0.46
3	9-12	27	23	80	0.47		23	27	80	0.47
4	13-16	28	12	70	0.40		26	14	70	0.38
5	17-20	20	15	65	0.41		19	16	65	0.40
6	21-24	16	14	60	0.42		14	16	60	0.42

Table 4-9. Displayed green times for single point urban interchange

		Timing Stage Green Time (s)				
Timing Plan ID	Exp. Design Scenarios	1	2	3	Cycle Length (s)	Crit. Mvmt. v/c
1	1-4	8	45	19	90	0.61
2	5-8	8	38	16	80	0.54
3	9-12	7	32	13	70	0.51
4	13-16	9	37	16	80	0.50
5	17-20	8	31	13	70	0.45
6	21-24	7	25	10	60	0.44

Table 4-10. Displayed green times for partial cloverleaf interchange

		West Intersection				East Intersection			
		Timing Stage Green Time (s)				Timing Stage Green Time (s)			
Timing Plan ID	Design Scenarios	1	2	Length (s)	Crit. Mvmt. v/c	1	2	Cycle Length (s)	Crit. Mvmt. v/c
1	1-4	57	13	80	0.55	56	14	80	0.52
2	5-8	47	13	70	0.49	47	13	70	0.47
3	9-12	39	11	60	0.47	39	11	60	0.47
4	13-16	46	14	70	0.46	46	14	70	0.45
5	17-20	41	14	65	0.41	40	15	65	0.41
6	21-24	36	14	60	0.39	36	14	60	0.39

Yellow and all-red times used for the timing plans are shown in Table 4-11.

Table 4-11. Change and clearance interval times

Interchange	**Yellow (s)**	**All-Red (s)**
Diamond	4	1
DDI[a]	4	11
SPUI	4	4
ParClo	3	2

[a] The DDI requires a long all-red interval due to the long intersection clearance distances.

The timing stage phases, and interval times get converted to the dual-ring structure for implementation in the signal controller. An example is illustrated in Figure 4-17.

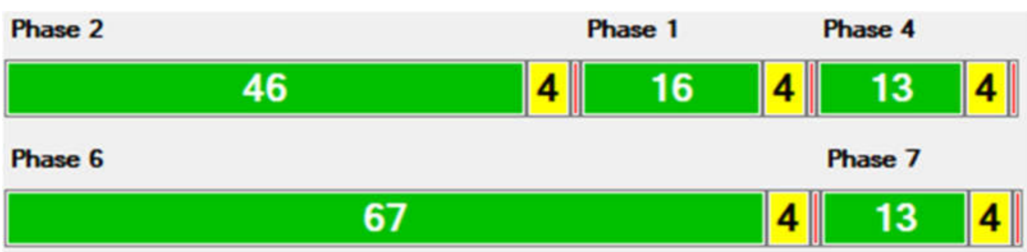


Figure 4-17. Signal controller dual-ring timing plan example.

Performance Measurement Methodology

Again, the performance measures considered in this study are:

- Interchange volume throughput
- Average speed for several travel paths through the arterial: through movements and right- and left-turn movements from the arterial to the on-ramp
- On-ramp measures
  - Average ramp meter queue discharge rate
  - Average % of time in three different metering modes: base, intermediate queue override, and advance queue override
  - Average on-ramp metering delay

This section provides an overview of the process used to calculate/determine these performance measures from the simulation results.

Interchange Volume Throughput

Each modeled interchange form is configured with detectors at multiple locations. This allows a comparison to be made between the input (demand) traffic volumes and the resulting observed traffic volumes. The reference volume measurement locations for each interchange are shown in Figure 4-18 through Figure 4-23. Also shown in Figure 4-24 through Figure 4-29 are the specified turning proportions (purple font) within the interchange area and resulting volumes based on scenario 1 traffic characteristics—heavy demand and a 65/35 directional split (EB is the peak direction).

The intent of these measurements is to identify bottleneck areas that are primarily a result of the ramp metering. Thus, for example, we expect to see significant differences in demand and observed volumes at the on-ramp just downstream of the ramp meter. While this is intentional, other locations within the interchange with significant differences between demand and observed volumes may indicate a situation where the on-ramp queue backup is restricting flow in unintentional or undesirable ways.

While there is a certain set of input volumes specified for each scenario, due to the nature of stochastic simulation, the actual input volumes will vary somewhat from replication to replication. This will lead to small percentage differences, but any differences < 0.1% are displayed as "—" so as to not draw attention to negligible differences. Additionally, for some of the relatively low demand volume movements, even small differences in absolute numbers can lead to percentage difference values that appear to be significant. To avoid this situation, the following check is applied:

$$\frac{|Observed\ Volume - Input\ Volume|}{NumLanes} \geq 15 \tag{4-1}$$

If this condition is met, the percentage difference is reported, otherwise it is displayed as "—".

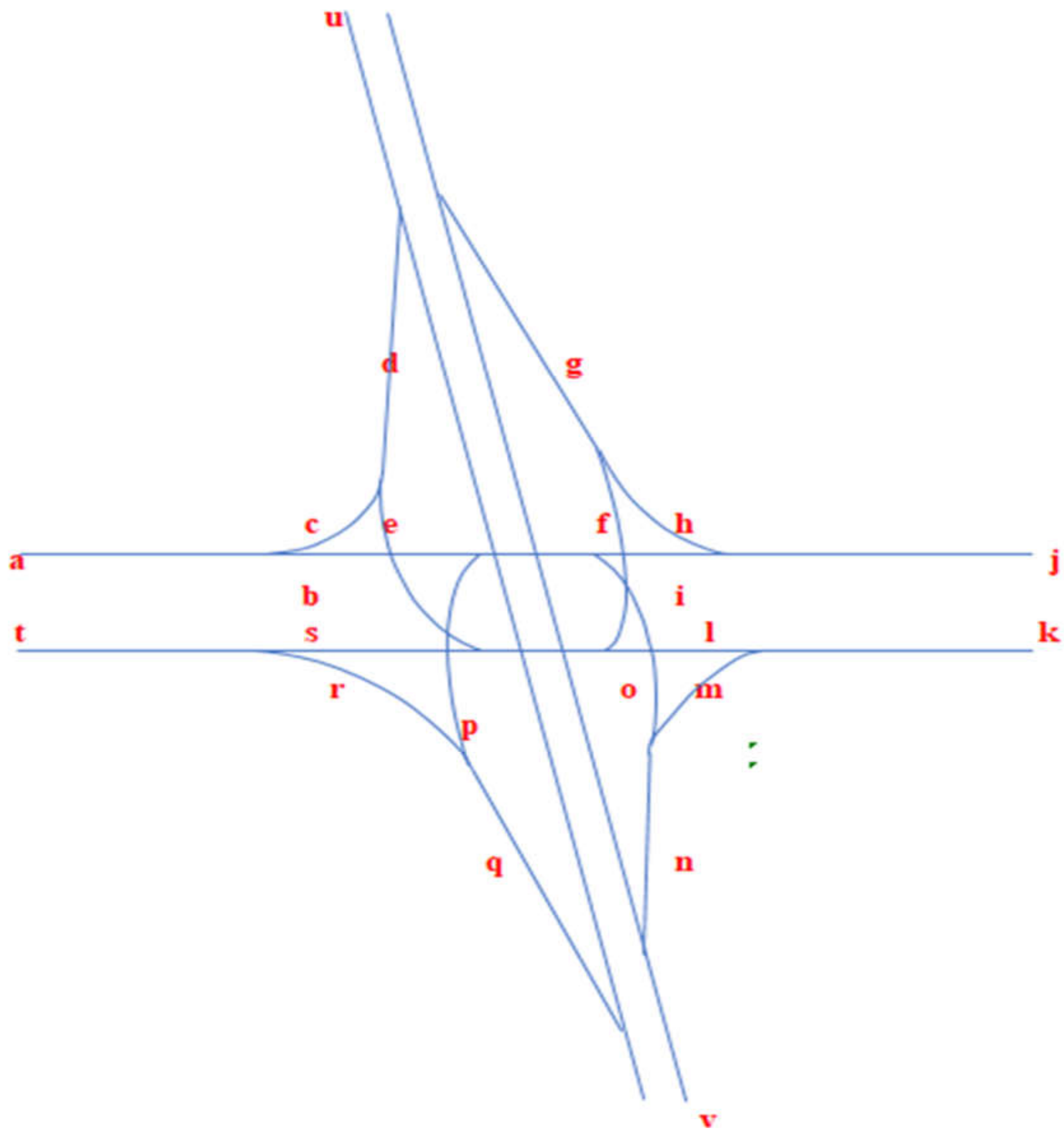


Figure 4-18. Diamond interchange detector location reference schematic.

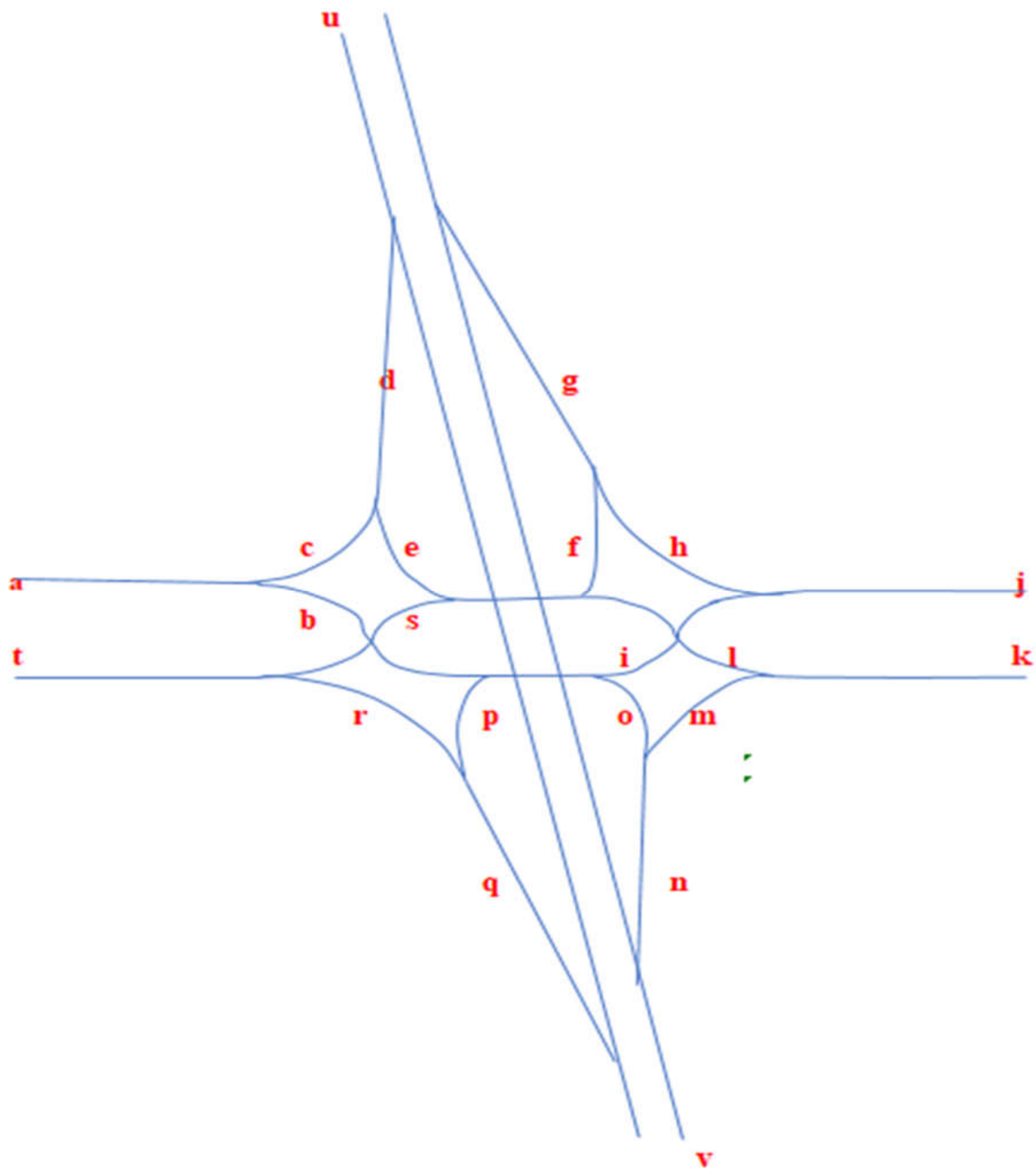


Figure 4-19. Diverging diamond interchange detector location reference schematic.

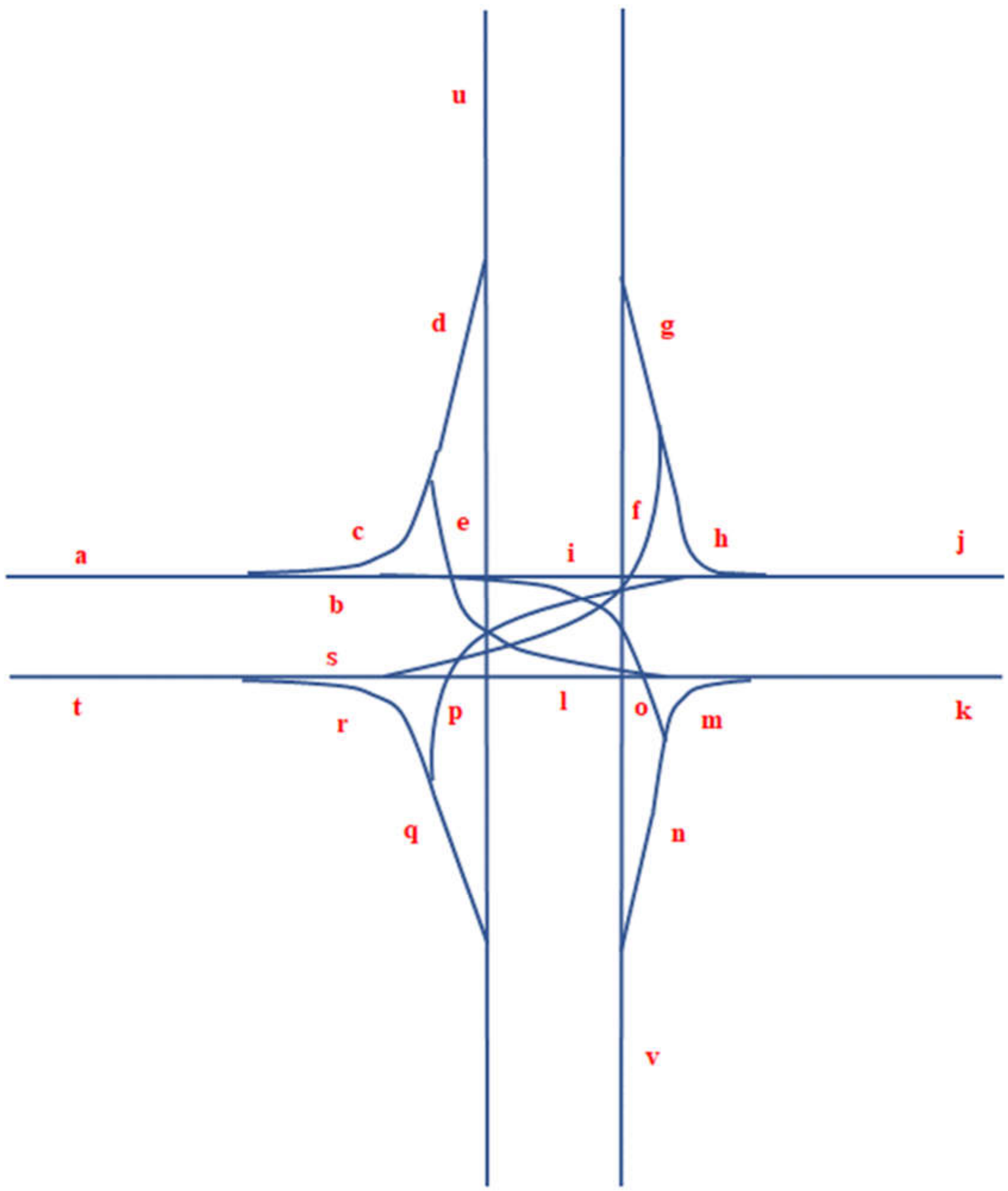


Figure 4-20. Single point urban interchange detector location reference schematic.

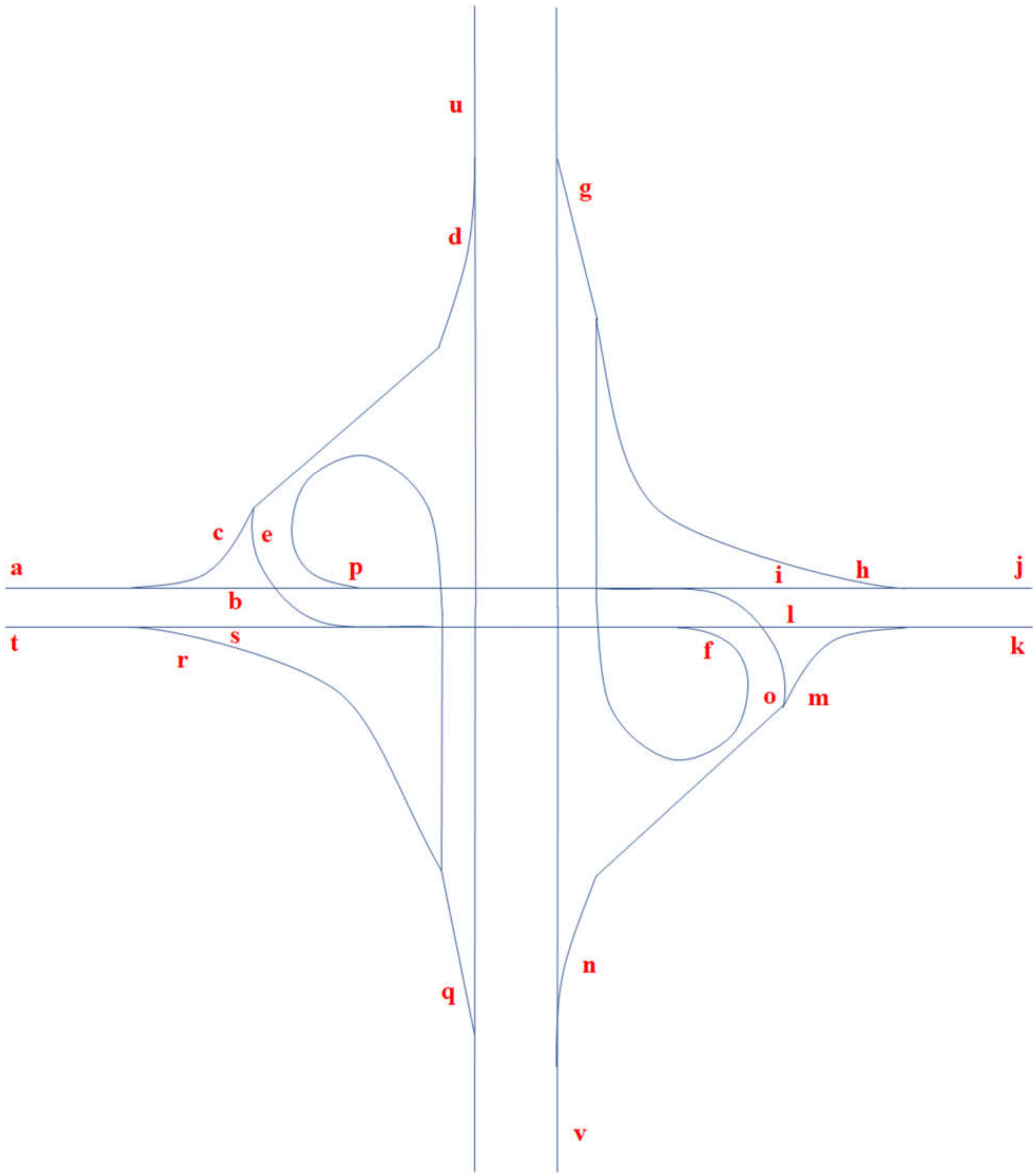


Figure 4-21. Partial cloverleaf interchange detector location reference schematic.

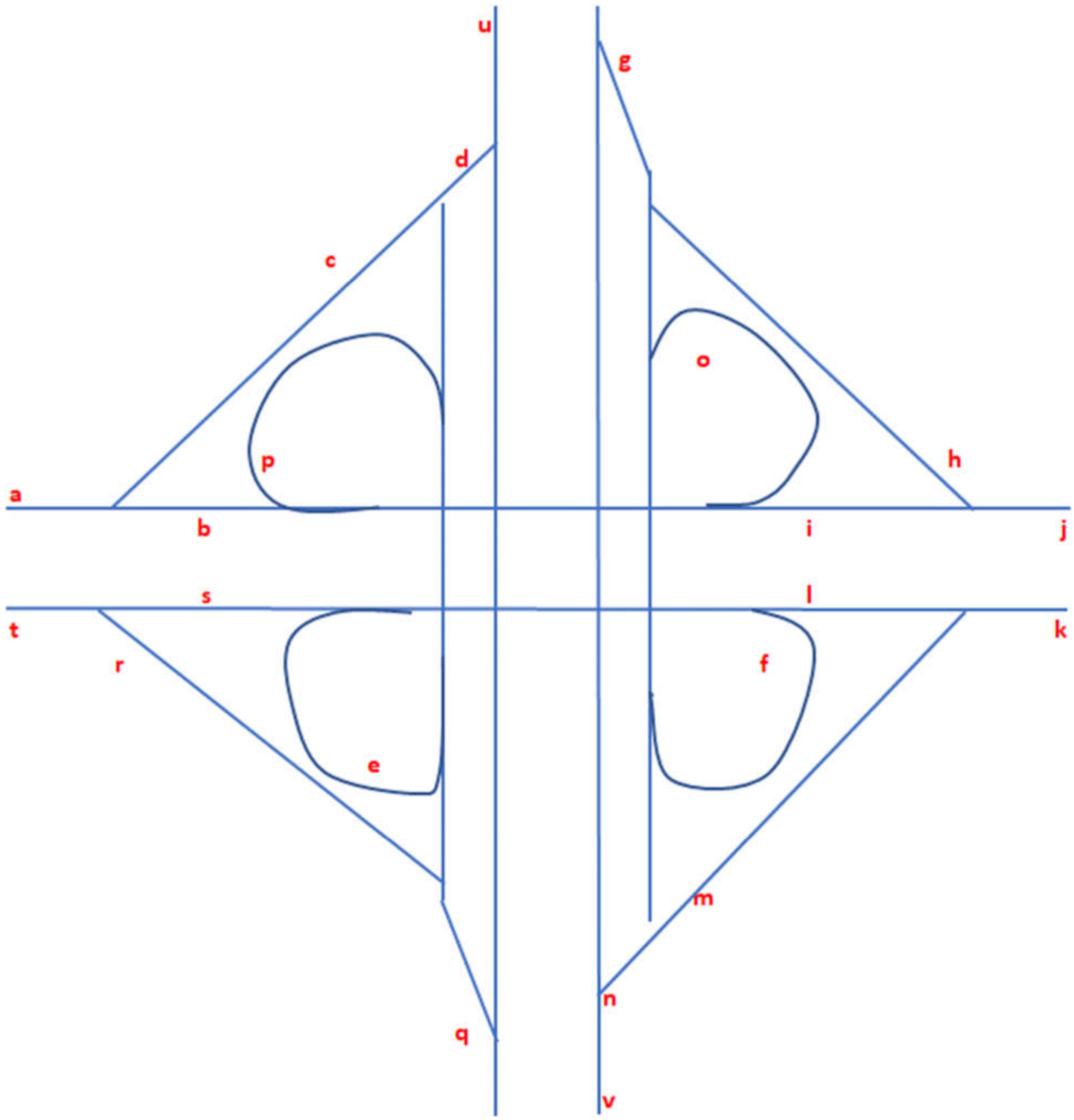


Figure 4-22. Full cloverleaf (with collector-distributor) interchange detector location reference schematic.

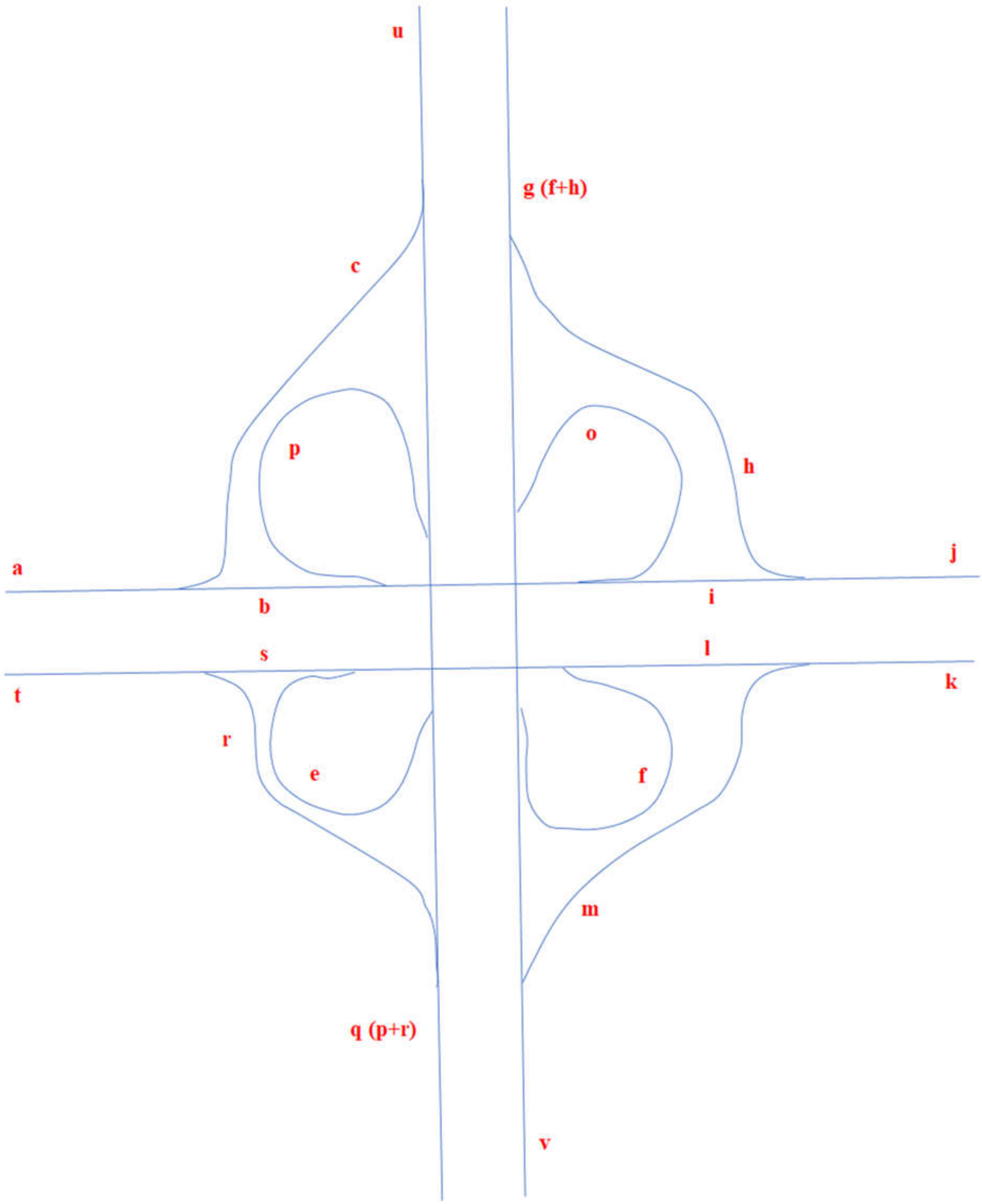


Figure 4-23. Full cloverleaf (without collector-distributor) interchange detector location reference schematic.

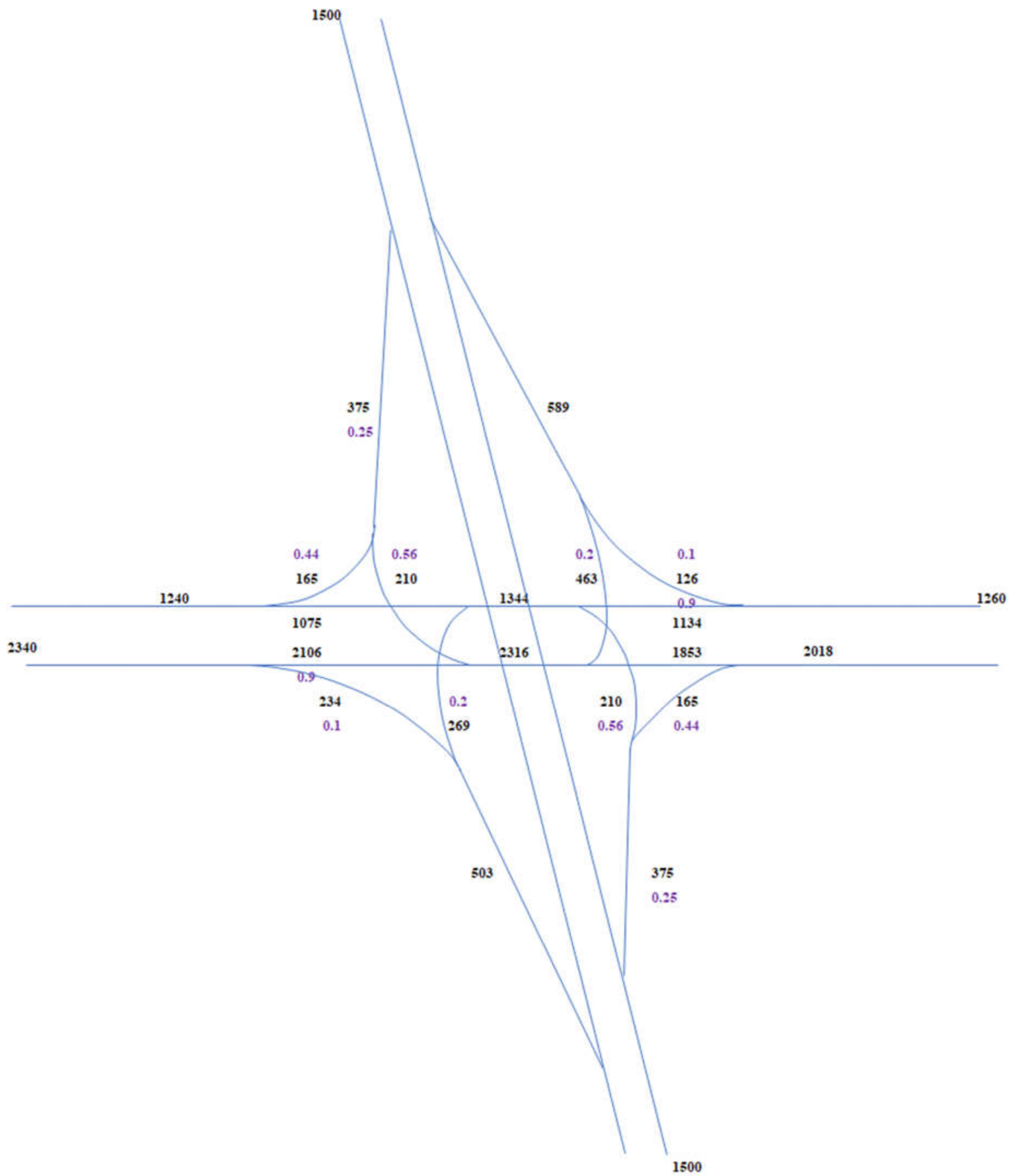


Figure 4-24. Demand pattern 1 for diamond interchange in veh/h (decimal values are turning percentages)

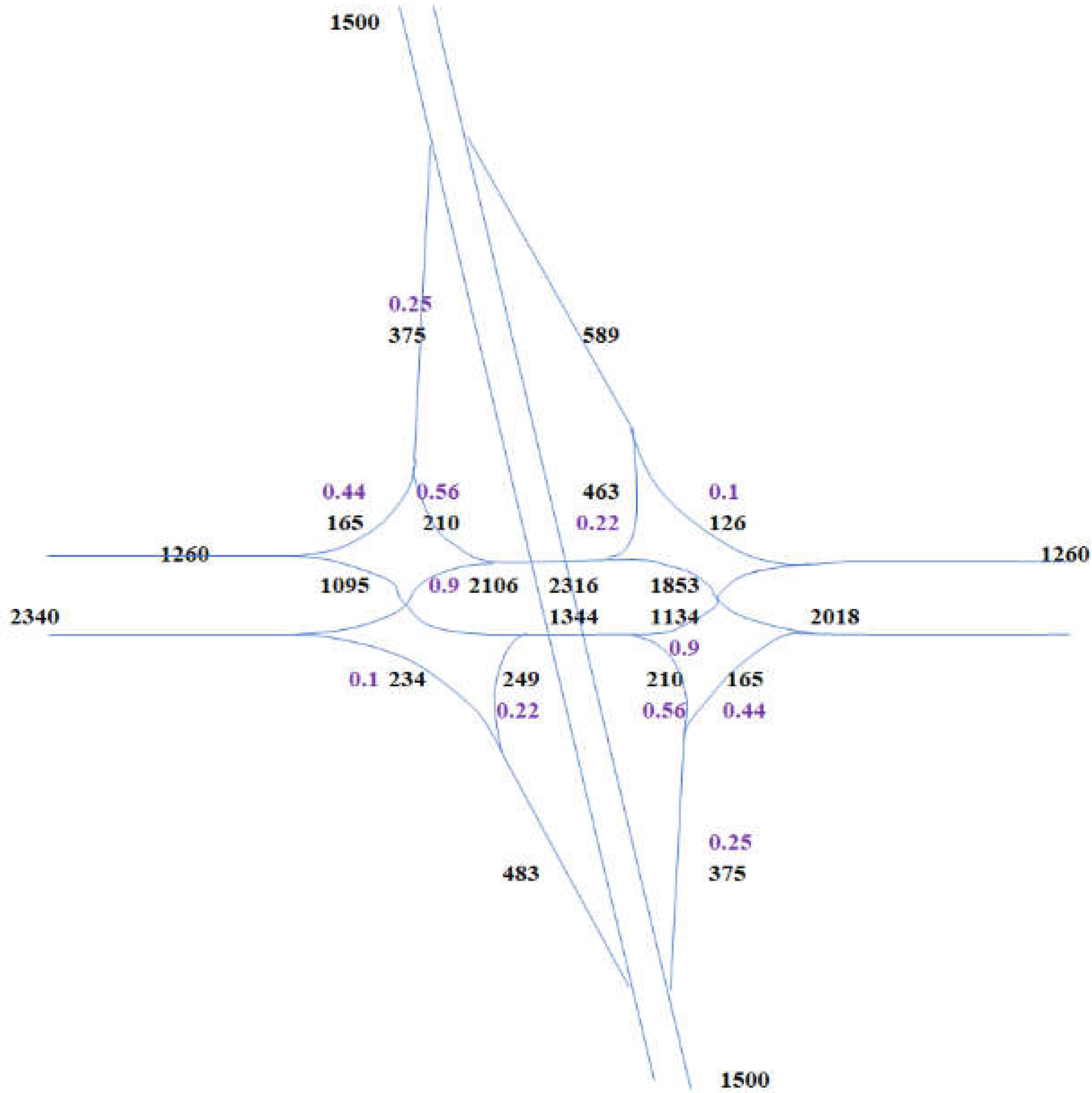


Figure 4-25. Demand pattern 1 for DDI in veh/h (decimal values are turning percentages)

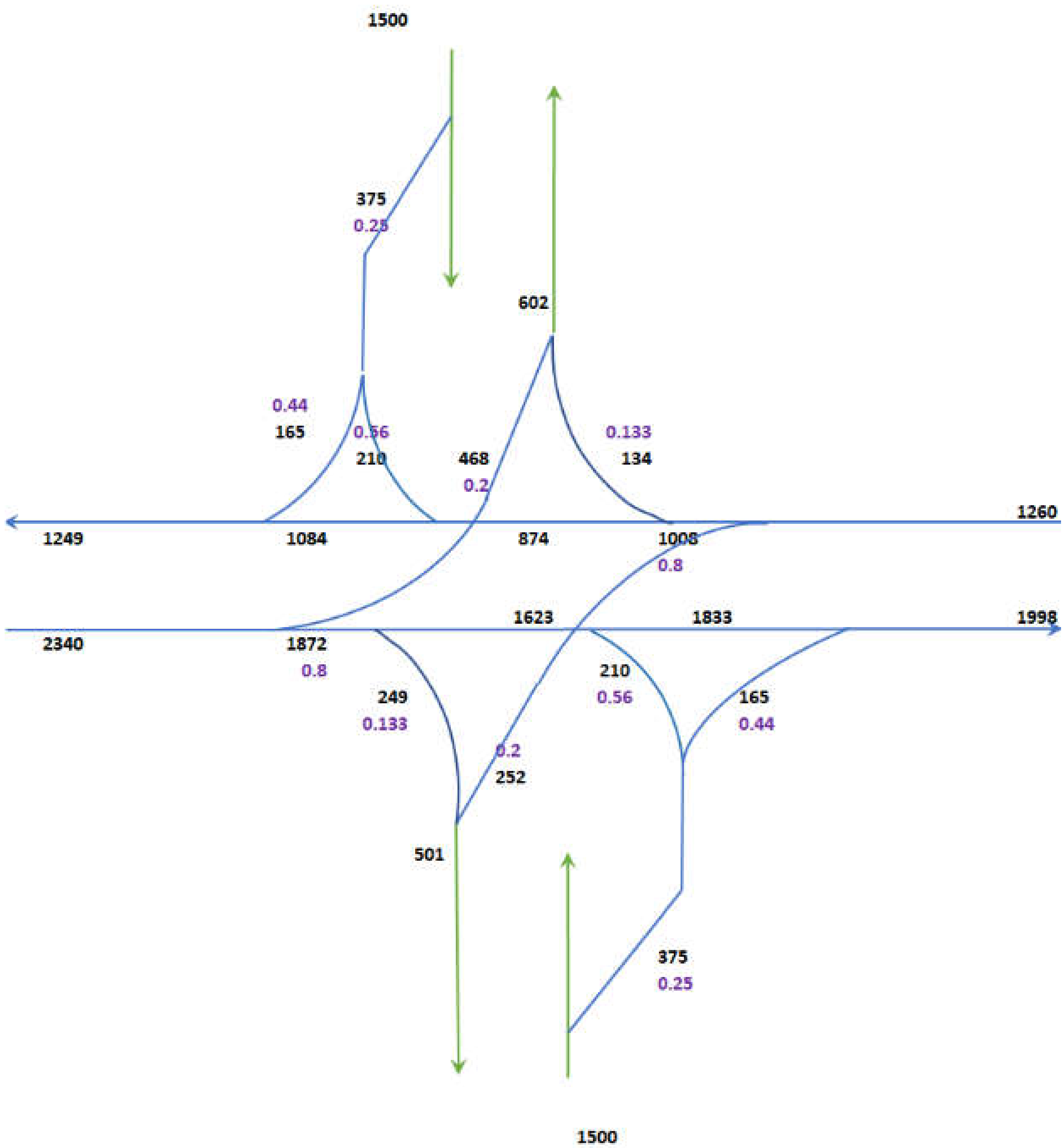


Figure 4-26. Demand pattern 1 for SPUI in veh/h (decimal values are turning percentages)

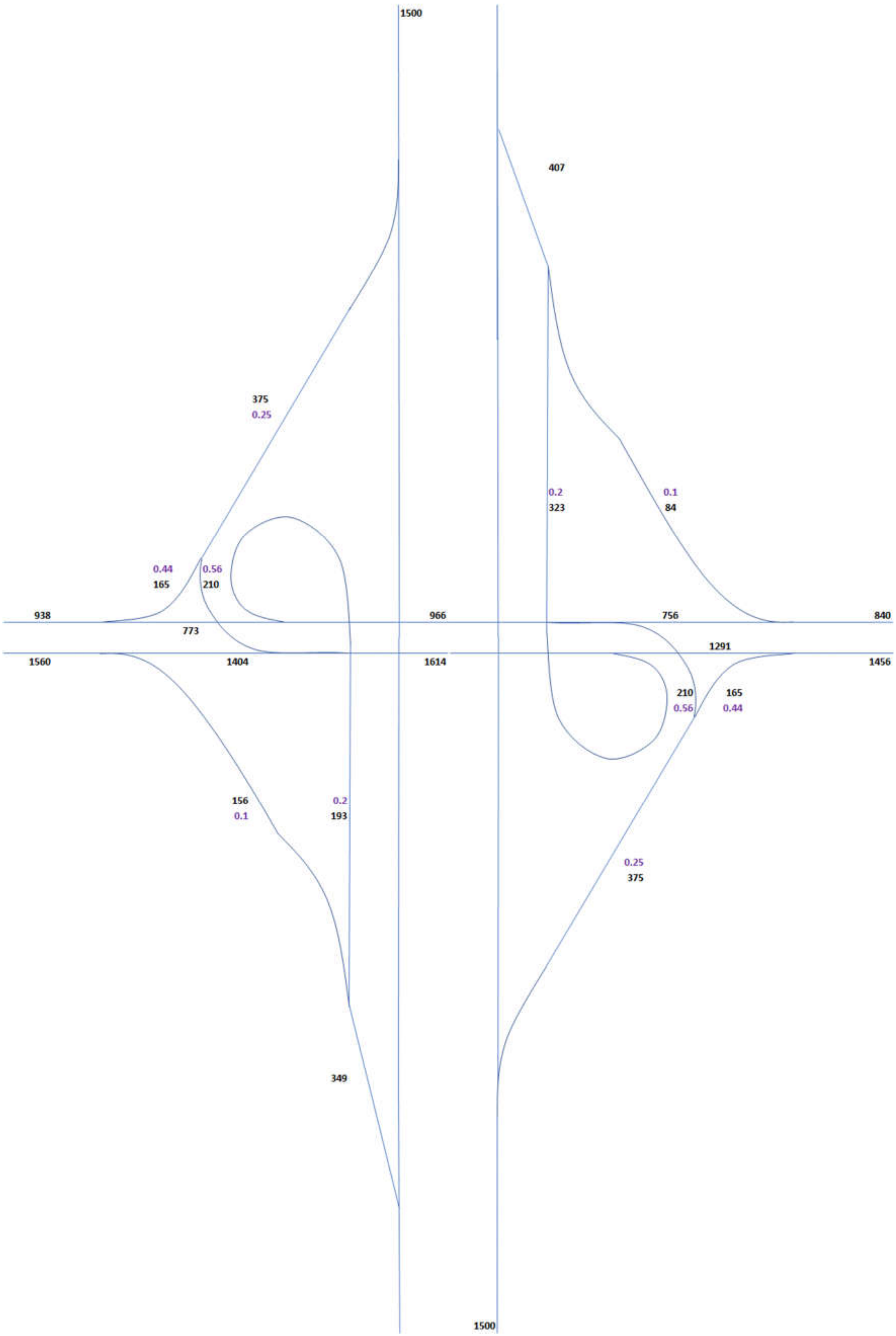


Figure 4-27. Demand pattern 1 for ParClo in veh/h (decimal values are turning percentages)

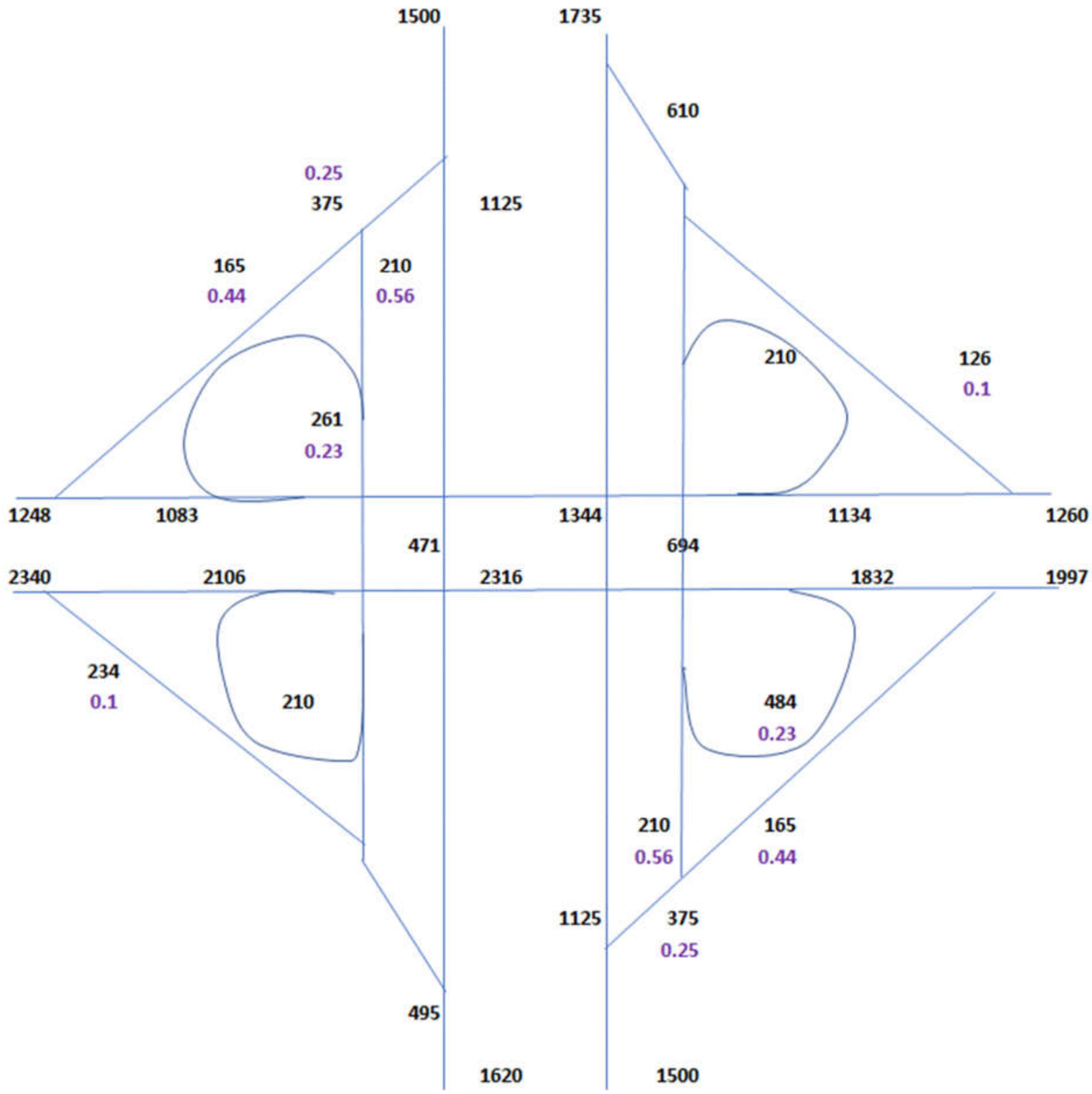


Figure 4-28. Demand pattern 1 for FullClo with C-D in veh/h (decimal values are turning percentages)

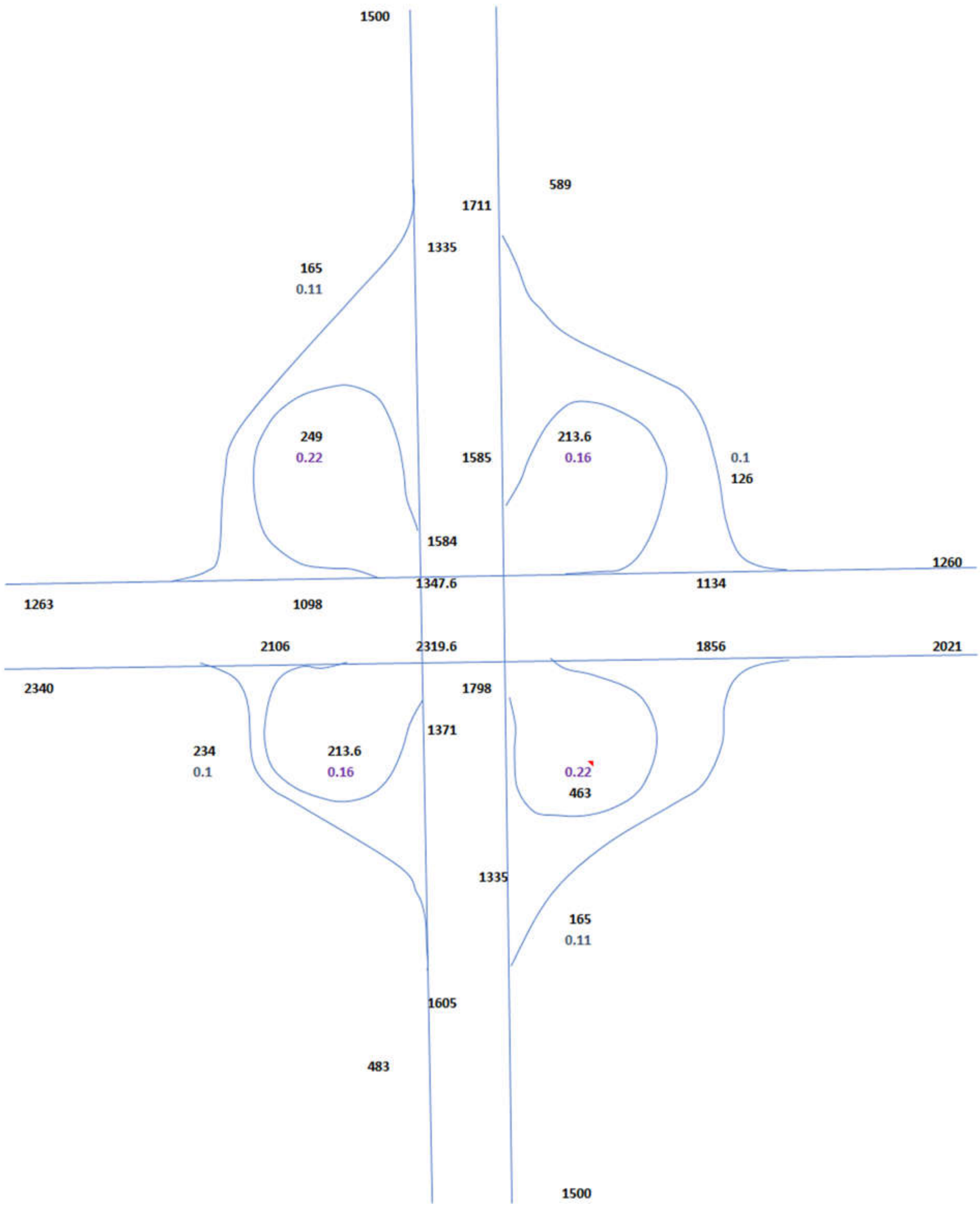


Figure 4-29. Demand pattern 1 for FullClo w/o C-D in veh/h
(decimal values are turning percentages)

<u>Signalized Intersection Capacity</u>

Intersection capacity values are provided, assuming operations are isolated from effects outside of the ramp terminal(s). Of course, in an interchange setting, intersection throughput may be constrained by downstream or upstream conditions. However, these capacity values provide a point of reference for the potential traffic movement capability of the interchange. The intersection capacity calculations utilize the given signal phasing/timing plan and Eq. 4-2

$$c = s \times g/C \tag{4-2}$$

Where:

c = capacity in veh/h,
s = saturation flow rate in veh/h, and
g/C = ratio of effective green time to cycle length.

Approximate saturation flow rates by movement, assuming level grade and 100% passenger cars, used in the calculations are as follows.

- Through only: 1800 veh/h/ln
- Through + Right Turn: 1700 veh/h/ln
- Left Turn: 1750 veh/h/ln

The effective green time is taken as the displayed green time plus 2 seconds.

Average Travel Speed

The measurement of average travel speed is calculated for three unique movements for each of two directions, for a total of six travel paths, as follows:

- Through: Vehicles that enter and exit the arterial mainline
- Right-turn: Vehicles that enter the arterial mainline and make a direct right-turn (i.e., not a loop ramp) onto the on-ramp
- Left turn: Vehicles that enter the arterial mainline and make a direct, or indirect (for a loop ramp), left turn

The average travel speed is calculated by dividing the distance between the start and end location of the respective travel path by the average travel time. The average travel time for a given path is calculated as follows:

- Measure the travel time for each vehicle that crosses the detectors at the start and end locations of each travel path.
- Sum the travel times for all vehicles that travel the subject path and divide by the total number of travel times for that path.

The simulation program outputs all of the detector actuation data to comma-separated value (CSV) files. See https://swashsim.miraheze.org/wiki/Simulation_Output_Data#Detector_Vehicle_Actuation_Data for more information about the data items output to these files. A custom data-processing tool is used to match vehicle IDs at the specified upstream and downstream detectors and calculate the corresponding travel time statistics.

The arterial mainline detectors are placed at a distance of about 2700 ft. from the interchange midpoint. Thus, the arterial through movement travel time is measured over a total distance of about 5400 ft., or just over 1 mi. The on-ramp detectors used are the ramp meter passage detectors. The total distances for each travel path for each interchange are given in Table 4-12. The travel time origin-destination pairs and corresponding detector stations are shown in Figure 4-30 through Figure 4-35.

Table 4-12. Distances, in feet, for travel time calculations

		Interchange Configuration					
Movement		**Diamond**	**DDI**	**SPUI**	**ParClo**	**FullClo with C-D**	**FullClo w/o C-D**
EB Entry to Exit	(1 → 4)	5401	5436	5405	5437	5398	5400
WB Entry to Exit	(8 ← 5)	5411	5436	5435	5424	5397	5401
EB Entry to SB On-ramp	(1 → 2)	3136	3147	4195	2992	3189	2985
EB Entry to NB On-ramp	(1 → 6)	4075	3892	4445	4883	5101	3654
WB Entry to NB On-ramp	(6 ← 5)	3130	3222	4300	2979	3189	2711
WB Entry to SB On-ramp	(2 ← 5)	4105	4032	4551	4870	5066	3324

The free-flow speed on the arterial mainline links is 35 mi/h. The free-flow speed for the ramp links changes incrementally from the arterial free-flow speed to the freeway mainline free-flow speed, with increments of 10 mi/h in between.

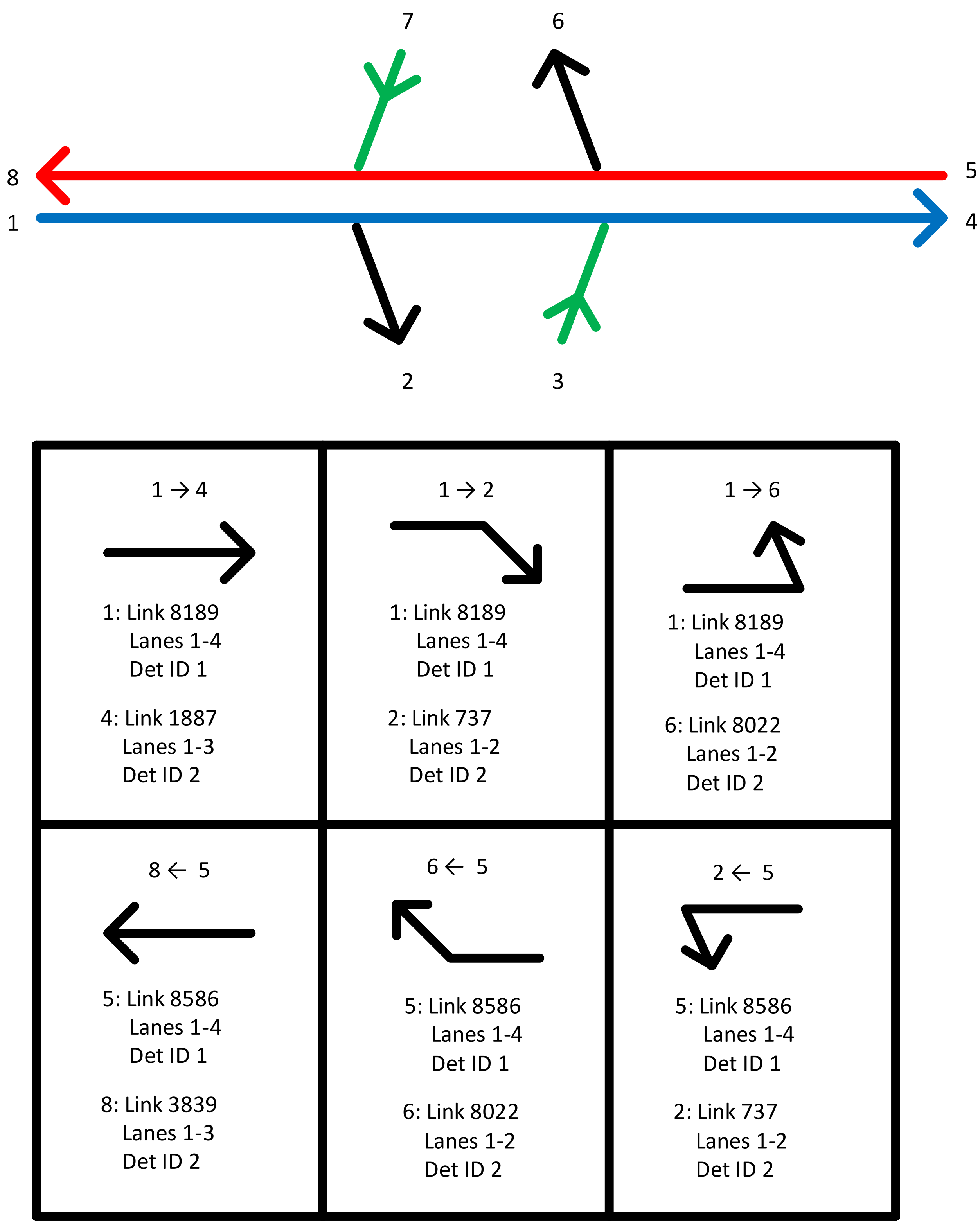


Figure 4-30. Diamond interchange travel time origin-destination pairs by movement.

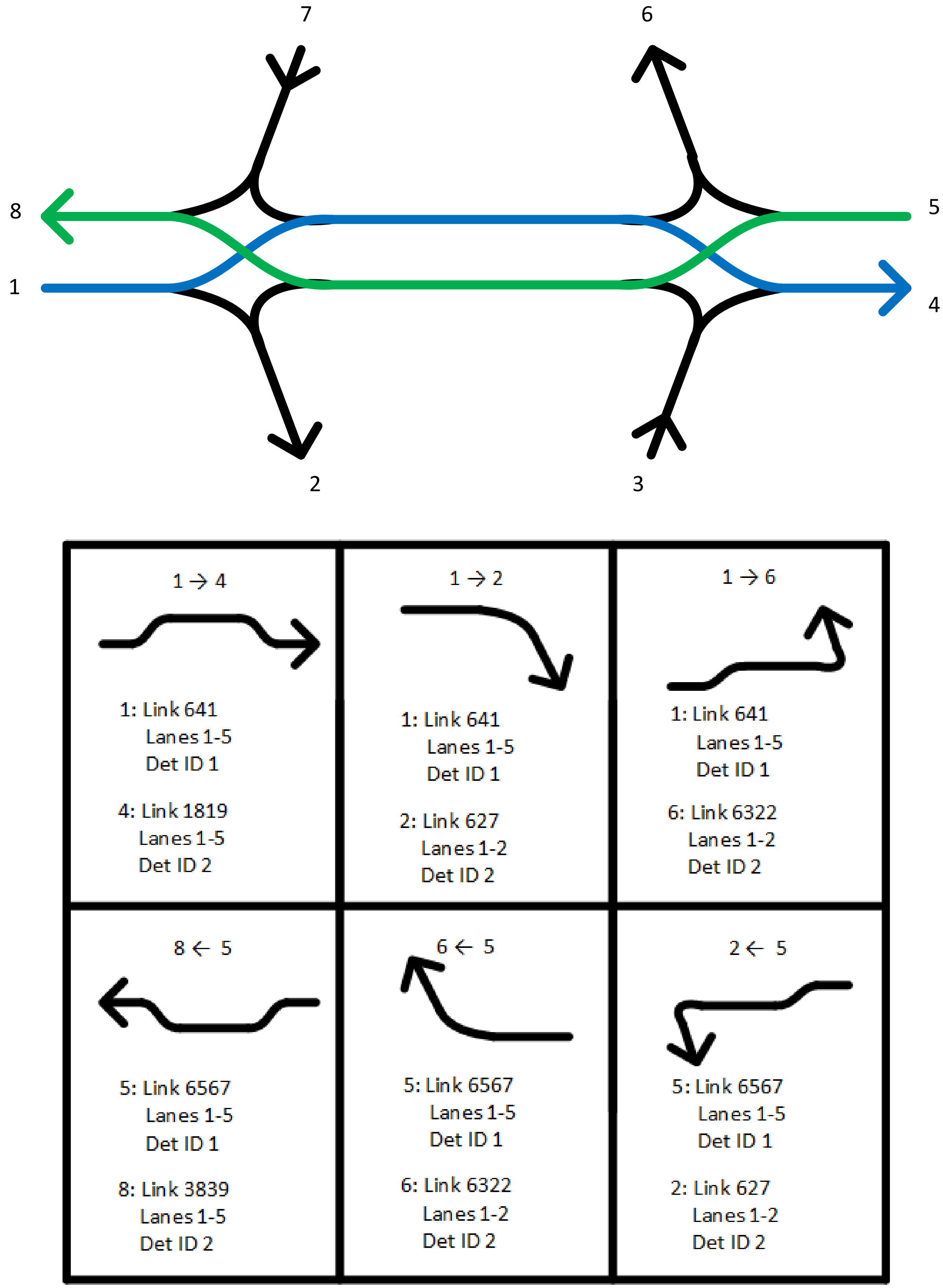


Figure 4-31. Diverging diamond interchange travel time origin-destination pairs by movement.

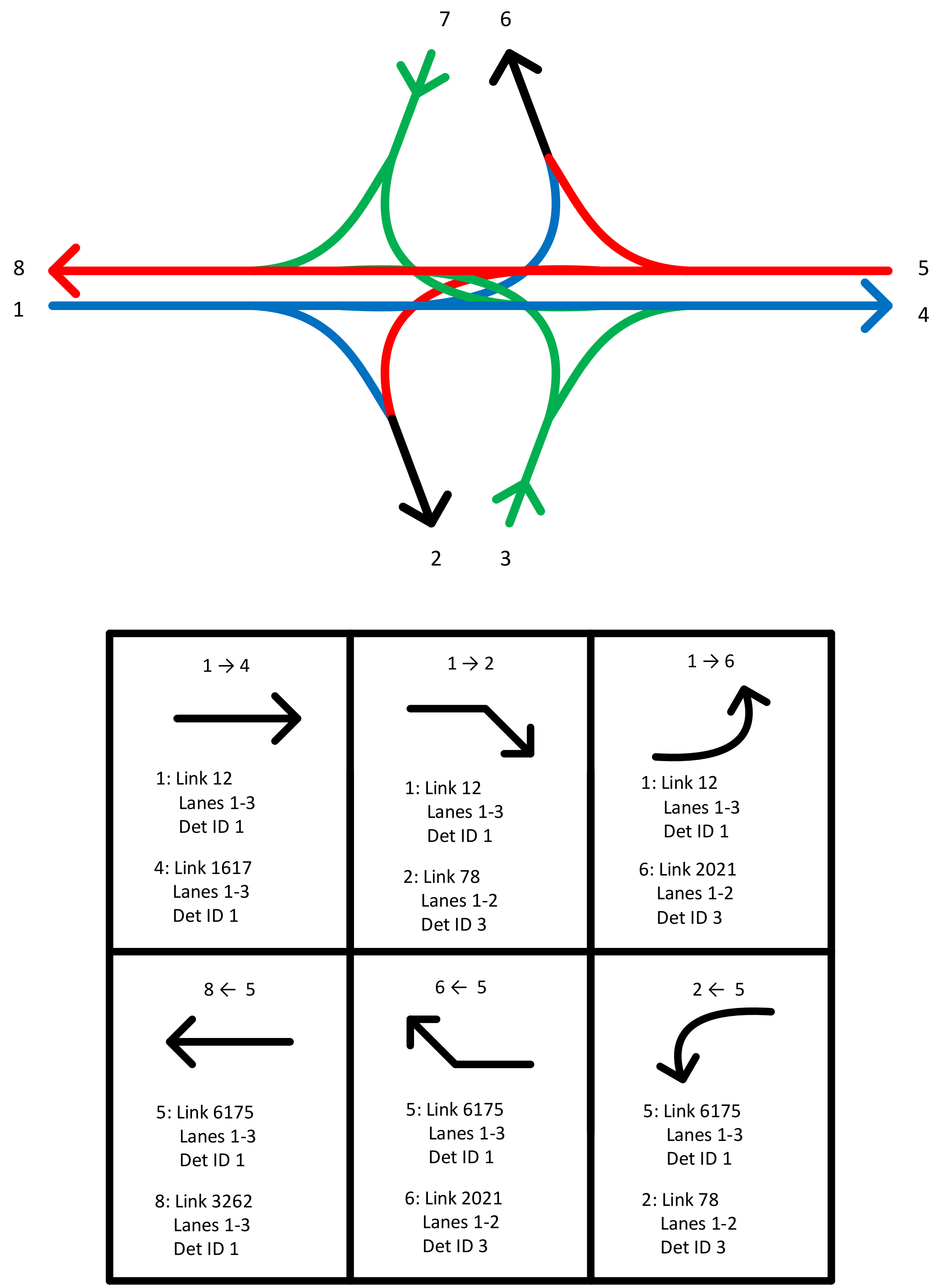


Figure 4-32. Single point urban interchange travel time origin-destination pairs by movement.

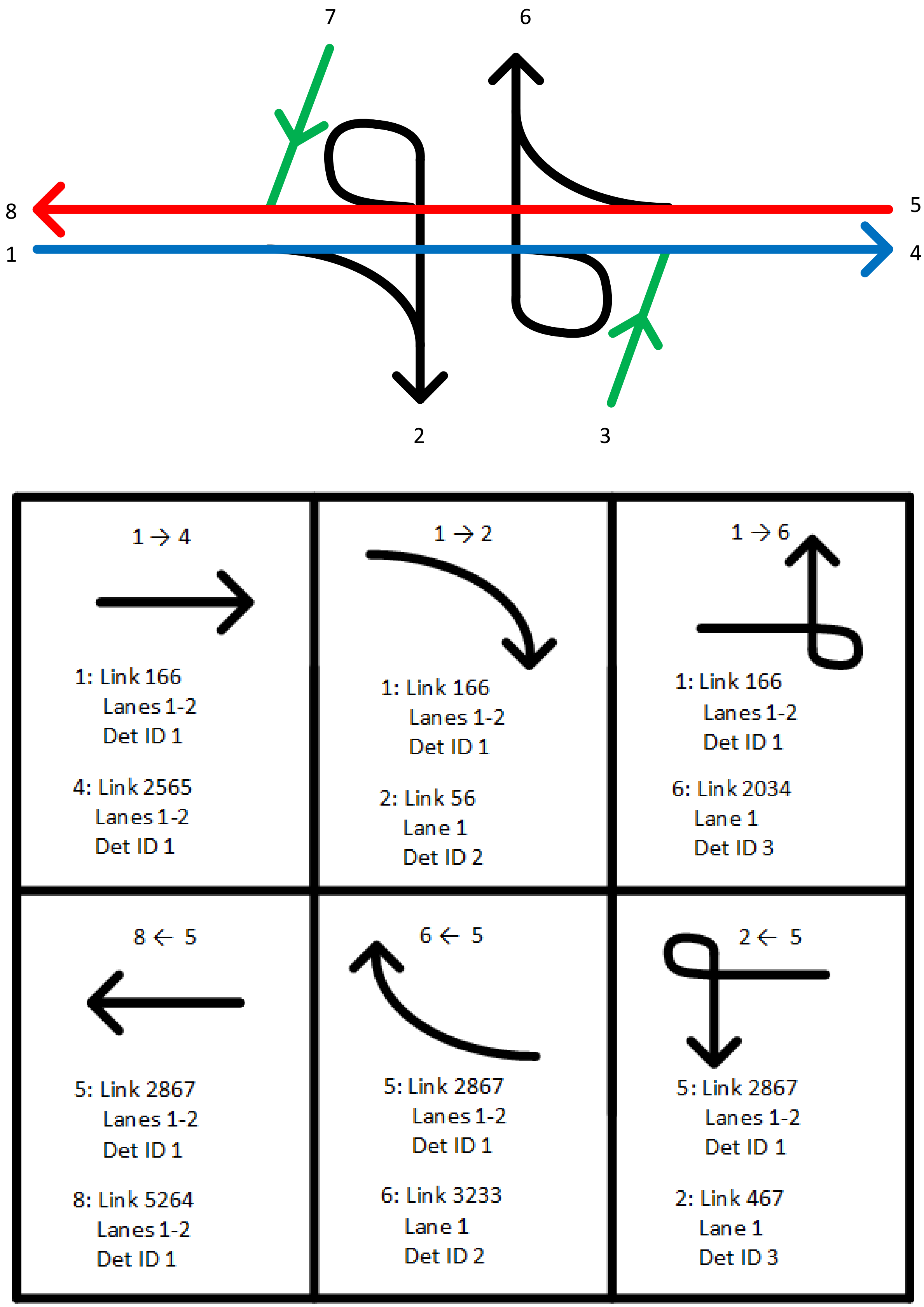
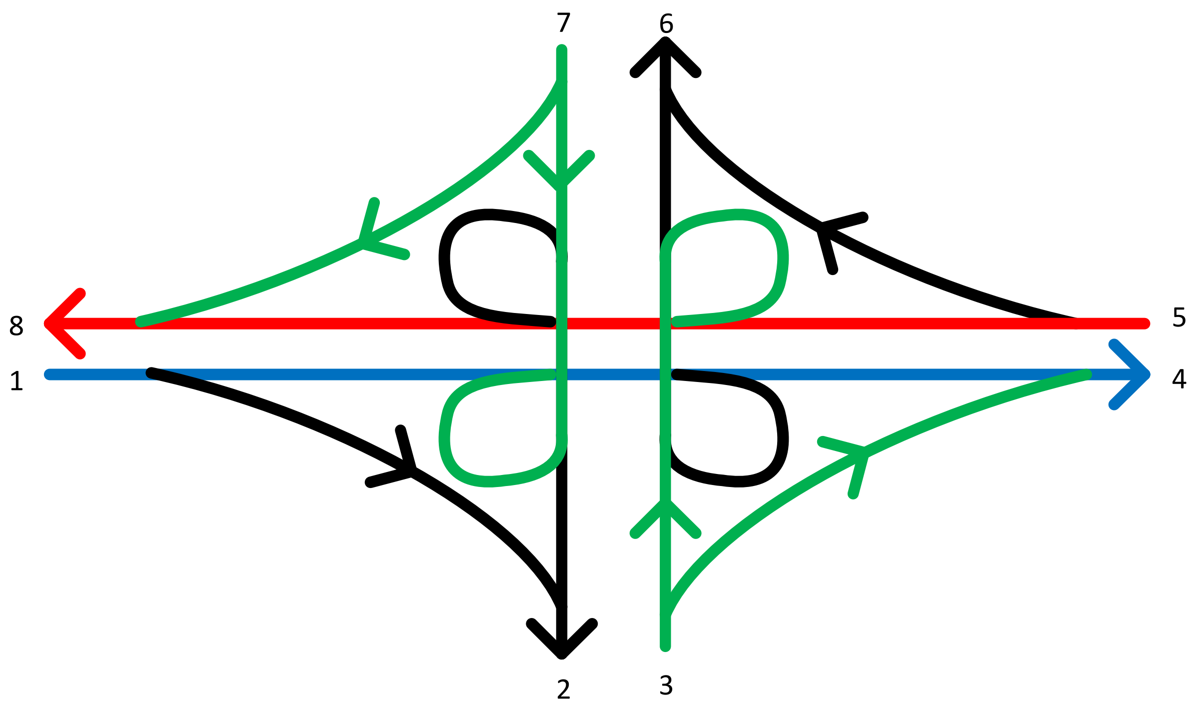


Figure 4-33. Partial cloverleaf travel time origin-destination pairs by movement.



1 → 4 1: Link 910 Lanes 1-3 Det ID 1 4: Link 1314 Lanes 1-3 Det ID 1	1 → 2 1: Link 910 Lanes 1-3 Det ID 1 2: Link 4243 Lane 1 Det ID 2	1 → 6 1: Link 910 Lanes 1-3 Det ID 1 6: Link 1718 Lane 1 Det ID 2
8 ← 5 5: Link 34 Lanes 1-3 Det ID 1 8: Link 78 Lanes 1-3 Det ID 1	6 ← 5 5: Link 34 Lanes 1-3 Det ID 1 6: Link 3738 Lane 1 Det ID 2	2 ← 5 5: Link 34 Lanes 1-3 Det ID 1 2: Link 2930 Lane 1 Det ID 2

Figure 4-34. Full cloverleaf, without C-D, travel time origin-destination pairs by movement.

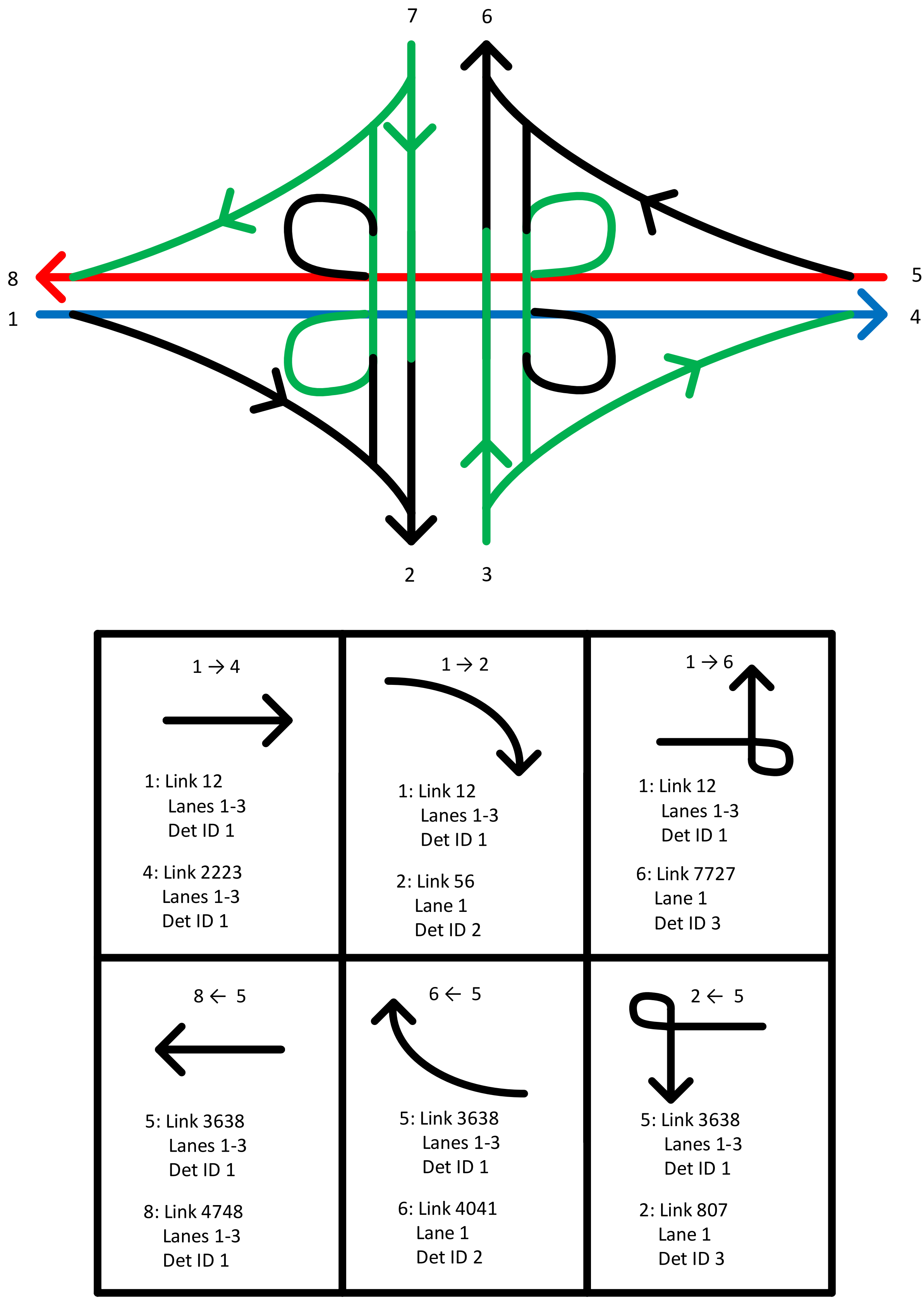


Figure 4-35. Full cloverleaf, with C-D, travel time origin-destination pairs by movement.

Average ramp meter queue discharge rate

The ramp meter queue discharge rate, in veh/h, is measured from passage detectors immediately downstream of the ramp meter stop bar. These values are an average across the replications within each scenario.

Average % of time with advance queue spillback

The ramp metering rate, for any given scenario, is set to one of three values:

- Base: the rate when on-ramp queue spillback adjustments are not necessary
  - For scenarios with the low metering rate, this value is 240 veh/h.
  - For scenarios with the medium metering rate, this value is 550 veh/h.
- Intermediate queue detector adjusted rate: when the on-ramp reaches the intermediate detector and this detector's occupancy value reaches the specified threshold, the metering rate is set to the base value (240/550 veh/h) plus 180 veh/h.
- Advance queue detector adjusted rate: when the on-ramp reaches the advance detector and this detector's occupancy value reaches the specified threshold, the metering rate is set to 900 veh/h.

The ramp metering rates and corresponding input detector occupancy values for every metering update interval (30 s) are output to CSV files. A summary file is also output, which provides the percentage of time over the simulation period that the metering rate was set to one of the above three values (base, intermediate queue, advance queue) for each ramp controller and each replication within a given scenario. See https://swashsim.miraheze.org/wiki/Simulation_Output_Data#Ramp_Metering_Data for more information about the data items output to these files. A macro is used to process the values in these values to determine average values across all replications within a given scenario, for each ramp controller.

For example:

- For a 3,600-s (1-h) simulation period, there would be 120 (3,600 s / 30 s) ramp metering calculation intervals
- Assume the following:
  - 90 of the intervals operated at the base metering rate
  - 18 of the intervals operated at the metering rate for intermediate queue override
  - 12 of the intervals operated at the metering rate for advance queue override
- The resulting percentages of time in which each metering rate mode was active would be as follows:
  - Base: 90/120 = 75%
  - Intermediate queue spillback override: 18/120 = 15%
  - Advance queue spillback override: 12/120 = 10%

Average on-ramp metering delay

Ideally, the delay due to ramp metering would be determined through the difference in simulation runs with ramp metering active versus ramp metering inactive. However, this approach is not used because it would require twice as many simulation runs. Instead, the delay due to ramp metering is estimated from the delay measured along the links that form a vehicle path immediately downstream of the signal/turning movement at the ramp terminal to the ramp meter passage detectors. This will include delay during the arrival to the ramp meter. This measure does not include the acceleration delay downstream of the ramp meter, but this represents a very small amount of delay compared to the delay upstream of the ramp meter.

This measure likely overestimates the delay due to ramp metering, as even without ramp metering, many vehicles will still experience travel speeds less than desired speed due to the general traffic conditions. Some vehicles, especially large trucks will still incur delay along the ramp roadway even without metering due to slow acceleration. It is estimated that the bias in the delay measurements is in the range of 5-10%.

The ramp meter delay is based on the delay values for the network links shown in Table 4-13.

Table 4-13. Links used for ramp metering delay calculation.

Interchange	NB on-ramp	SB on-ramp
Diamond Large	8022, 7280, 2072, 6920, 3021	737, 7173, 671, 706, 45
DDI Large	6322, 2063, 1520, 3221	627, 662, 366, 45
SPUI	2526, 1118, 1819, 1920, 2021	34, 275, 56, 67, 78
ParClo	**Direct on-ramp**–3537, 3435, 3233, 3132, 2931 **Loop on-ramp**–3537, 3435, 2034, 2720, 2627, 2226	**Direct on-ramp**–810, 78, 56, 45, 34 **Loop on-ramp**–810, 78, 467, 5546, 5455, 4854
FullClo with Collector/Distributor	**Direct on-ramp**–910, 89, 78, 56, 45, 34 **Loop on-ramp**–807, 7880, 5152, 5051, 4550	**Direct on-ramp**–2930, 2829, 2728, 4041, 3940, 3739 **Loop on-ramp**–7727, 7577, 2526, 2425, 1924
FullClo w/o Collector/Distributor	**Direct on-ramp**–4344, 4243, 4142, 4041 **Loop on-ramp**–2930, 2829, 2728, 56	**Direct on-ramp**–3839, 3738, 3637, 6436 **Loop on-ramp**–1718, 1617, 1516, 1112

This calculation uses the "LinkResultsAllScenarios.csv" output data file. Total delay values (veh-s) are output in column Z and total through volume is output in column AC. A macro identifies the appropriate links and divides the total delay by the total through volume for each link and then sums the resulting average delay values across the included links.

More information about the delay calculations in SwashSim can be found at
https://swashsim.miraheze.org/wiki/Performance_Measures#Delay

Results and Analysis

Arterial Volume Throughput

The percentage difference in input and observed traffic volumes are provided in the following tables.

Table 4-14. Percentage difference in demand versus observed volumes, for diamond interchange (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	-	-	-	-	-	-	-	-	-	-2.0%	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-4.0%	-6.8%	-8.8%	-6.2%	-3.2%	-3.4%	-6.9%	-3.1%	-4.9%	-6.3%	-7.6%	-7.0%
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-	-	-	-	-	-	-	-
G (NB On Tot)	-	-3.7%	-	-	-7.5%	-	-	-	-3.9%	-	-4.1%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-8.9%	-6.1%	-8.6%	-8.8%	-5.4%	-7.6%	-6.1%	-8.5%	-8.8%	-5.6%	-7.2%	-6.7%
O (NB LT Off)	-	-	-11.2%	-10.2%	-	-	-	-	-9.5%	-9.0%	-	-
P (SB LT On)	-7.4%	-7.8%	-8.4%	-	-	-6.8%	-	-	-	-	-	-
Q (SB On Tot)	-7.4%	-	-7.5%	-	-7.7%	-5.0%	-6.6%	-	-	-4.8%	-4.7%	-
R (SB RT On)	-	-	-	-	-	-	-5.6%	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-15. Percentage difference in demand versus observed volumes, for diamond interchange (Scenarios 13-24)

Scenario	13	14	15	16	17	18	19	20	21	22	23	24
A (WB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-6.0%	-5.6%	-4.9%	-3.2%	-6.0%	-5.6%	-6.0%	-3.3%	-3.9%	-6.2%	-6.3%	-8.7%
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-	-	-6.7%	-	-	-	-	-
G (NB On Tot)	-	-	-	-	-7.6%	-4.9%	-10.9%	-5.7%	-8.3%	-6.6%	-4.9%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-13.9%	-	-	-	-	-	-	-	-
N (NB Off Tot)	-8.1%	-7.7%	-8.9%	-10.0%	-8.3%	-5.7%	-6.7%	-6.8%	-5.5%	-6.7%	-9.6%	-8.0%
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB LT On)	-9.2%	-14.2%	-10.7%	-13.2%	-9.8%	-8.6%	-8.4%	-9.1%	-	-8.9%	-	-8.4%
Q (SB On Tot)	-11.5%	-9.7%	-10.4%	-	-10.4%	-6.8%	-9.8%	-4.8%	-6.4%	-7.0%	-6.6%	-5.7%
R (SB RT On)	-	-	-	5.5%	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

As expected, there is a significant difference in on-ramp volumes due to queuing at ramp meter. The off-ramp movements, particularly left turns, frequently were not fully served each cycle. This was more prevalent with the higher truck percentage scenarios, and for the northbound left-turn movement, which feeds into the non-coordinated westbound arterial direction. A positive number is generally indicative of a movement that received higher demand in a majority of replications than the specified demand volume. This can occasionally happen as a result of a combination of the stochastic process for vehicle entry and turning movement assignment.

Table 4-16. Percentage difference in demand versus observed volumes, for DDI (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	-	-	-	-	3.7%	3.9%	-	-	-	-	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-	-	-	-	-	-	-	-
G (NB On Tot)	-4.0%	-	-4.2%	-	-8.1%	-	-3.9%	-	-8.5%	-	-7.0%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB LT On)	-	-	-7.9%	-	-	-	-	-	-	-	-	-
Q (SB On Tot)	-7.1%	-4.2%	-7.8%	-	-7.6%	-4.9%	-7.0%	-	-5.8%	-	-	-4.1%
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-17. Percentage difference in demand versus observed volumes, for DDI (Scenarios 13-24)

Scenario	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
A (WB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-	-	-	-	-	-6.9%	-	-
G (NB On Tot)	-8.4%	-	-11.5%	-	-9.3%	-	-7.4%	-	-11.5%	-5.5%	-10.6%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB LT On)	-10.7%	-13.3%	-14.7%	-13.0%	-	-14.0%	-7.7%	-9.9%	-7.4%	-	-	-8.5%
Q (SB On Tot)	-12.3%	-5.1%	-12.4%	-6.6%	-7.0%	-9.8%	-9.6%	-7.9%	-11.9%	-6.9%	-10.1%	-6.6%
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

For the DDI, only the on-ramps experienced a lower volume served than the demand (again, due to the metering restriction). Additionally, for the medium metering rate scenarios (even numbers), northbound on-ramp queueing was minimal.

Table 4-18. Percentage difference in demand versus observed volumes, for SPUI (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-7.1%	-
D (SB Off Tot)	3.0%	-	-4.3%	-	-	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-	-	-	-	-	-	-	-
G (NB On Tot)	-4.5%	-	-3.8%	-	-	-	-	-	-5.3%	-	-	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-6.8%	-	-	-	-	-	-6.6%	-	-6.5%	-
N (NB Off Tot)	-	-	-	-	-	-	-	-	-	-	-3.6%	-
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB LT On)	-	-	-	-8.9%	-	-	-	-	-7.0%	-	-	-
Q (SB On Tot)	-8.5%	-	-7.6%	-5.9%	-7.0%	-	-7.8%	-	-10.9%	-	-8.9%	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-19. Percentage difference in demand versus observed volumes, for SPUI (Scenarios 13-24)

Scenario	13	14	15	16	17	18	19	20	21	22	23	24
A (WB Art Exit)	-	-	-	3.0%	-	-	-	-	-	-	-	-
B (WB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB LT On)	-	-	-	-	-7.4%	-	-	-	-9.8%	-	-	-
G (NB On Tot)	-6.8%	-	-10.6%	-	-7.4%	-	-8.1%	-	-8.8%	-	-8.4%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	-	2.5%	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	6.7%	-	-
N (NB Off Tot)	-	-	-	-	-	-	-	-	-	4.7%	-	-2.7%
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB LT On)	-11.9%	-	-10.6%	-	-	-	-	-	-8.4%	-	-	-
Q (SB On Tot)	-11.0%	-	-7.5%	-	-5.8%	-	-7.4%	-	-8.8%	-	-7.6%	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

The results are comparable to those for the DDI.

Table 4-20. Percentage difference in demand versus observed volumes, for ParClo (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	2.8%	-	-	-	-	-	-	2.2%	1.5%	-	-	-
B (WB Art Thru)	3.0%	-	2.0%	-	-	-	-	1.8%	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-	-	-	-	-	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB Loop On)	-6.3%	-	-3.9%	-2.7%	-4.7%	-	-5.1%	-4.0%	-5.6%	-	-5.0%	-3.9%
G1 (NB On-Ramp Loop)	-8.2%	-8.6%	-6.9%	-9.2%	-10.6%	-6.9%	-10.7%	-6.6%	-12.5%	-4.5%	-10.1%	-7.5%
G2 (NB On-Ramp Direct)	-	-	-	-	-	-	-6.6%	-	-	-	-	-
H (NB RT On)	-	-	-	-	-	-	-6.6%	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	1.4%	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	1.4%	1.3%	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	1.7%	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-	-3.1%	-	-	-	-	-	-	-	-	-	-
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB Loop On)	-6.7%	-6.4%	-5.7%	-5.1%	-3.3%	-	-5.5%	-4.3%	-3.6%	-	-4.0%	-3.1%
Q1 (SB On-Ramp Loop)	-18.6%	-7.1%	-16.6%	-6.5%	-12.8%	-4.7%	-13.8%	-5.8%	-10.6%	-4.6%	-10.0%	-4.7%
Q2 (SB On-Ramp Direct)	-	-	-	-	-6.2%	-	-	-	-9.1%	-	-	-
R (SB RT On)	-	-	-	-	-6.2%	-	-	-	-9.1%	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-21. Percentage difference in demand versus observed volumes, for ParClo (Scenarios 13-24)

Scenario	13	14	15	16	17	18	19	20	21	22	23	24
A (WB Art Exit)	3.3%	2.6%	3.6%	2.5%	-	2.0%	-	-	2.2%	-	-	-
B (WB Art Thru)	4.3%	3.5%	3.7%	3.9%	-	1.9%	-	-	2.5%	-	1.9%	2.0%
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB Off Tot)	-	-	3.5%	-2.8%	-2.8%	-	-	-	-	-	-	-
E (SB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
F (NB Loop On)	-5.0%	-	-4.4%	-5.5%	-	-6.3%	-3.5%	-4.9%	-6.7%	-6.6%	-8.4%	-
G1 (NB On-Ramp Loop)	-12.4%	-6.0%	-10.1%	-8.3%	-9.8%	-7.7%	-12.4%	-5.0%	-19.0%	-7.7%	-19.6%	-
G2 (NB On-Ramp Direct)	-	-	-	-	-	-	-	-	-7.7%	-	-	-
H (NB RT On)	-	-	-	-	-	-	-	-	-7.7%	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	1.9%
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	1.5%	-	-	-	1.6%	1.9%	-
L (EB Art Thru)	1.6%	-	2.1%	-	-	2.0%	-	1.9%	-	2.3%	2.6%	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-	-	-	-	-	-	-	-	-	-3.0%	-	-
O (NB LT Off)	-	-	-	-	-	-	-	-	-	-	-	-
P (SB Loop On)	-11.1%	-17.0%	-10.7%	-10.8%	-6.0%	-8.8%	-4.3%	-7.0%	-4.7%	-	-5.6%	-5.7%
Q1 (SB On-Ramp Loop)	-16.0%	-17.6%	-16.3%	-11.1%	-16.3%	-9.7%	-15.3%	-6.0%	-15.4%	-3.9%	-15.3%	-6.4%
Q2 (SB On-Ramp Direct)	-6.9%	-	-6.3%	-	-8.2%	-	-	-	-	-	-	-
R (SB RT On)	-6.9%	-	-6.3%	-	-8.2%	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

The differences for the loop ramps are consistently significant. The right-turn (direct) ramps occasionally have significant differences. This illustrates a challenge with the 'split-ramp' design of the ParClo. The experimental design traffic characteristics were set up with higher left-turn demands than the right-turn movement. The right-turn movement often had excess queue storage capacity, but the left-turn traffic cannot access it in this design. It is also notable that in some cases the arterial through volume had positive difference values. This is usually caused by vehicles not being able to join the loop-ramp queue and thus get reassigned to a through movement. While this is a simulation 'artifact', a similar scenario can happen in reality—drivers bypass a long queue for one ramp and then do a U-turn further down the arterial to access the other on-ramp.

Table 4-22. Percentage difference in demand versus observed volumes, for FullClo with C-D (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	5.3%	3.9%	5.3%	-	-	-	-	-	-	-	-	-
B (WB Art Thru)	6.1%	4.5%	6.5%	3.4%	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-6.8%	-	-	-6.1%	-7.5%	-	-
D (SB RT Off)	-3.5%	-	-3.0%	-	-	-3.4%	-	-	-	-	-	-
E (EB Loop Off)	-	-5.2%	-	-	7.8%	-	11.6%	-	11.0%	-	10.3%	-
F (NB Loop On)	-	-	5.0%	-	-	-	-	-	-	-	-	-
G1 (NB On-Ramp Loop)	-9.5%	-4.9%	-10.5%	-7.1%	-6.5%	-	-5.5%	-	-	-3.6%	-5.7%	-4.1%
G2 (NB On-Ramp Direct)	-	-	-	-	-8.9%	-	-	-	-	-5.9%	-	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-2.0%	-	-	-	-	-	-	-	-	-
L (EB Art Thru)	-	-	-1.9%	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
N (NB Off Tot)	-	-	-	-	-3.3%	-	-	-	-	-	-	-
O (WB Loop Off)	31.4%	14.8%	34.3%	14.4%	7.9%	5.4%	18.6%	5.5%	8.4%	-	7.0%	-
P (SB Loop On)	-	-	-	-	-	-	-	-	-	-	-	-
Q1 (SB On-Ramp Loop)	-7.4%	-4.2%	-4.3%	-	-3.6%	-	-	-	-5.8%	-5.5%	-7.7%	-
Q2 (SB On-Ramp Direct)	-7.0%	-	-5.9%	-	-7.1%	-	-6.5%	-	-6.5%	-	-8.4%	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-23. Percentage difference in demand versus observed volumes, for FullClo with C-D (Scenarios 13-24)

Scenario	13	14	15	16	17	18	19	20	21	22	23	24
A (WB Art Exit)	4.3%	-	2.9%	-	-	-	-	-	-	-	-	-
B (WB Art Thru)	4.0%	-	3.5%	-	-	-	-	-	-	-	-	-
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
D (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
E (EB Loop Off)	-	-	-	-	-	-	-	-	5.1%	-	-	-
F (NB Loop On)	-	-	-	-	-	-	-	-	-	-	-	-
G1 (NB On-Ramp Loop)	-5.6%	-	-3.9%	-4.2%	-6.9%	-	-	-	-7.9%	-5.0%	-6.0%	-5.1%
G2 (NB On-Ramp Direct)	-	-	-	-	-	-	-	-	-7.5%	-	-	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	-	-	-	-	-	-	-	-	3.0%	-	-	-
L (EB Art Thru)	-	-	-	-	-	-	-	-	-	-	-	-
M (NB RT Off)	-	-	-	-	-	-	-	-	6.3%	-	-8.0%	-
N (NB Off Tot)	-	-	-3.6%	-	-	-	-	-	4.1%	-	-	-
O (WB Loop Off)	9.1%	-	11.0%	-	5.6%	-5.2%	-	-	5.0%	-	8.7%	-
P (SB Loop On)	-	-	-	-	-	-	-	-	-	-	-	-
Q1 (SB On-Ramp Loop)	-11.8%	-8.3%	-8.5%	-6.3%	-5.8%	-	-4.2%	-	-	-	-6.2%	-3.9%
Q2 (SB On-Ramp Direct)	-6.3%	-	-8.1%	-	-	-	-	-	-7.7%	-	-	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

There are similarities to the ParClo results. However, of note with these results is the positive exit loop ramp values in some cases. This is a result of a queue backup into the weaving area of the C-D lanes for the loop on-ramp, such that some vehicles originally destined for the on-ramp queue bypass it and proceed to the exit loop ramp. Again, largely a simulation 'artifact', but an indicator of a undesirable situation that can lead drivers to make undesired maneuvers to try to "beat the system".

Table 4-24. Percentage difference in demand versus observed volumes, for FullClo without C-D (Scenarios 1-12)

Scenario	1	2	3	4	5	6	7	8	9	10	11	12
A (WB Art Exit)	3.6%	-	4.0%	3.4%	3.6%	2.4%	4.7%	2.7%	5.4%	2.3%	6.5%	2.4%
B (WB Art Thru)	4.3%	3.2%	5.1%	4.2%	4.2%	3.1%	4.7%	3.4%	5.8%	2.5%	7.4%	2.1%
C (SB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
[redacted]												
E (EB Loop Off)	-	-	-5.0%	-	-	-9.7%	-10.4%	-6.0%	-7.9%	-5.5%	-7.0%	-13.4%
F (NB Loop On)	-35.8%	-23.7%	-34.9%	-23.1%	-30.2%	-15.5%	-30.6%	-18.1%	-29.4%	-17.2%	-27.4%	-14.8%
[redacted]												
G2 (NB On-Ramp Direct)	-	-	-	-	-	-	-	-	-	-	-9.2%	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-7.6%	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	8.8%	5.4%	9.2%	5.3%	7.6%	2.7%	6.7%	3.5%	5.4%	2.8%	6.5%	-
L (EB Art Thru)	9.4%	5.9%	9.3%	5.8%	8.0%	2.8%	7.6%	3.4%	6.4%	3.3%	7.1%	-
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
[redacted]												
O (WB Loop Off)	-4.7%	-6.4%	-	-	-6.7%	-5.2%	-9.3%	-8.1%	-8.4%	-5.4%	-	-5.9%
P (SB Loop On)	-23.3%	-15.2%	-22.6%	-17.2%	-24.4%	-16.0%	-24.7%	-16.9%	-27.0%	-12.1%	-27.1%	-13.4%
[redacted]												
Q2 (SB On-Ramp Direct)	-	-	-	-	-	-	-5.4%	-	-	-	-	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

Table 4-25. Percentage difference in demand versus observed volumes, for FullClo without C-D (Scenarios 13-24)

Scenario	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
A (WB Art Exit)	-	-	-	-	-	2.7%	3.4%	-	4.0%	2.3%	3.2%	-
B (WB Art Thru)	-	-	-	-	3.4%	3.3%	3.6%	-	4.6%	2.7%	3.6%	-
C (SB RT Off)	-	-	-	-	-7.2%	-	-	-	-	-	-	-
E (EB Loop Off)	-6.1%	-	-	-7.2%	-8.7%	-5.3%	-6.8%	-9.6%	-8.3%	-9.0%	-7.9%	-
F (NB Loop On)	-29.8%	-13.8%	-29.6%	-14.5%	-25.3%	-17.4%	-27.0%	-12.0%	-23.9%	-14.8%	-23.8%	-14.7%
G2 (NB On-Ramp Direct)	-	-	-	-	-	-	-	-	-	-	-	-
H (NB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
I (WB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
J (WB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
K (EB Art Exit)	6.5%	3.2%	7.1%	3.0%	4.1%	2.4%	5.5%	-	3.3%	-	4.3%	3.0%
L (EB Art Thru)	7.1%	3.3%	7.7%	2.5%	5.4%	3.3%	6.0%	-	4.4%	2.7%	4.7%	3.6%
M (NB RT Off)	-	-	-	-	-	-	-	-	-	-	-	-
O (WB Loop Off)	-6.4%	-9.4%	-6.2%	-6.6%	-7.4%	-	-6.1%	-7.5%	-7.2%	-5.3%	-9.2%	-9.1%
P (SB Loop On)	-19.8%	-21.3%	-20.5%	-15.5%	-20.2%	-17.7%	-19.8%	-16.4%	-23.5%	-17.2%	-22.2%	-16.0%
Q2 (SB On-Ramp Direct)	-	-	-	-	-	-	-	-	-	-	-	-
R (SB RT On)	-	-	-	-	-	-	-	-	-	-	-	-
S (EB Art L+T)	-	-	-	-	-	-	-	-	-	-	-	-
T (EB Art Entr)	-	-	-	-	-	-	-	-	-	-	-	-
U (SB Fwy Entr)	-	-	-	-	-	-	-	-	-	-	-	-

This design avoids the complication of the FullClo with C-D design because of the lack of a C-D weaving segment, but experiences more significant differences than the ParClo design due to less queuing storage for the loop ramp and thus more backup onto the arterial (the ramp meter is on the loop ramp itself).

Summary

Generally, for the medium-demand scenarios, there were very few volume throughput issues, other than downstream of the ramp meter, as intended. For the high-demand scenarios and a 65/35 directional split, a common issue for the FullClo with C-D was the backup of on-ramp queuing into the C-D weaving area. Similarly, for the FullClo w/o C-D was the backup of on-ramp queuing onto the arterial weaving area. Figure 4-36 illustrates the situation where the loop on-ramp queue is about to block the entry to the weaving section. Eventually, this will lead to some on-ramp vehicles being re-assigned to the exit loop ramp. This is partly a simulation issue, as many drivers would continue to queue up on the entry loop ramp, but it does lead to some other practical concerns that are discussed later.

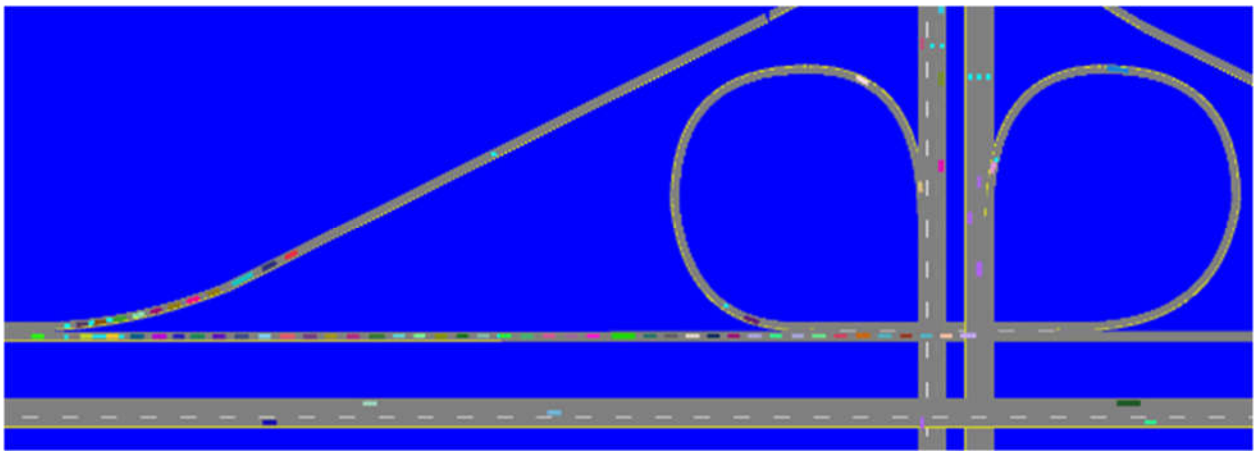


Figure 4-36. Loop ramp queue backup at FullClo with C-D

Figure 4-37 illustrates an on-ramp queue backup to the connecting arterial weaving area for a FullClo without C-D. In this case, again, some vehicles initially assigned to the on-ramp may be reassigned to an arterial through movement.

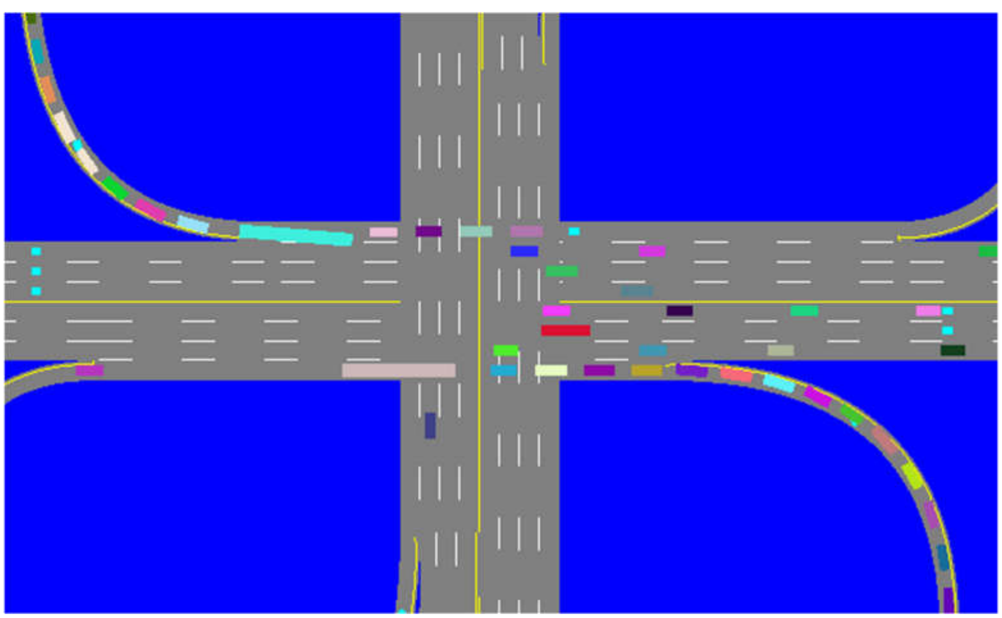


Figure 4-37. Loop ramp queue backup at FullClo without C-D

Signalized intersection capacity

Diamond Interchange

For this demand scenario, the approximate total intersection movement capacity is provided in Table 4-26.

Table 4-26. Diamond intersection capacity (timing plan for Traffic Scenario 1)

Timing Stage	**1**	**2**	**3**
Phases	2, 6	1, 6	4, 7
Lanes	3, 3	2, 3	2, 2
s	1800 × 6 = 10,800	1750 × 2 + 1800 × 3 = 8,900	1800 × 2 + 1750 × 2 = 7,100
g/C	60/115 = 0.522	29/115 = 0.252	20/115 = 0.174
c	10,800 × 0.522 = 5,638	8,900 × 0.252 = 2,243	7,100 × 0.174 = 1,235
Total Capacity (veh/h)	5,638 + 2,243 + 1,235 = 9,116		

DDI

For this demand scenario, the approximate total intersection movement capacity is provided in Table 4-27.

Table 4-27. DDI intersection capacity (timing plan for Traffic Scenario 1)

Timing Stage	**1**	**2**
Phases	2, 5	1, 6
Lanes	5, 3	2, 5
s	1800 × 8 = 14,400	1800 × 7 = 12,600
g/C	37/80 = 0.463	19/80 = 0.238
c	14,400 × 0.463 = 6,667	12,600 × 0.238 = 2,999
Total Capacity (veh/h)	6,667 + 2,999 = 9,666	

SPUI

For this demand scenario, the approximate total intersection movement capacity is provided in Table 4-28.

Table 4-28. SPUI intersection capacity (timing plan for Traffic Scenario 1)

Timing Stage	**1**	**2**	**3**
Phases	1, 5	4, 8	3, 7
Lanes	2, 2	3, 3	2, 2
s	1750 × 4 = 7,000	1800 × 6 = 10,800	1750 × 4 = 7,000
g/C	9/90 = 0.1	46/90 = 0.511	20/90 = 0.222
c	7,000 × 0.1 = 700	10,800 × 0.511 = 5,519	7,000 × 0.222 = 1,554
Total Capacity (veh/h)	700 + 5,519 + 1,554 = 7,773		

ParClo

For this demand scenario, the approximate total intersection movement capacity is provided in Table 4-29.

Table 4-29. ParClo intersection capacity (timing plan for Traffic Scenario 1)

Timing Stage	**1**	**2**
Phases	4, 8	2, 5
Lanes	3, 3	1, 2
s	1800 × 6 = 10,800	1750 × 3 = 5,250
g/*C*	58/80 = 0.725	14/80 = 0.175
c	10,800 × 0.725 = 7,830	5,250 × 0.175 = 919
Total Capacity (veh/h)	7,830 + 919 = 8,749	

These intersection capacity values are provided for informational purposes rather than direct comparison. While the DDI is shown to have the highest capacity value, the number of lanes by movement is not identical across all interchange forms. Nonetheless, the DDI generally benefits from a more efficient combination of geometry and timing plan, which is discussed further at the end of this section.

Average Travel Speed

Figure 4-38 through Figure 4-43 present the average travel speed results, across all scenarios, for a given travel path through the interchange.

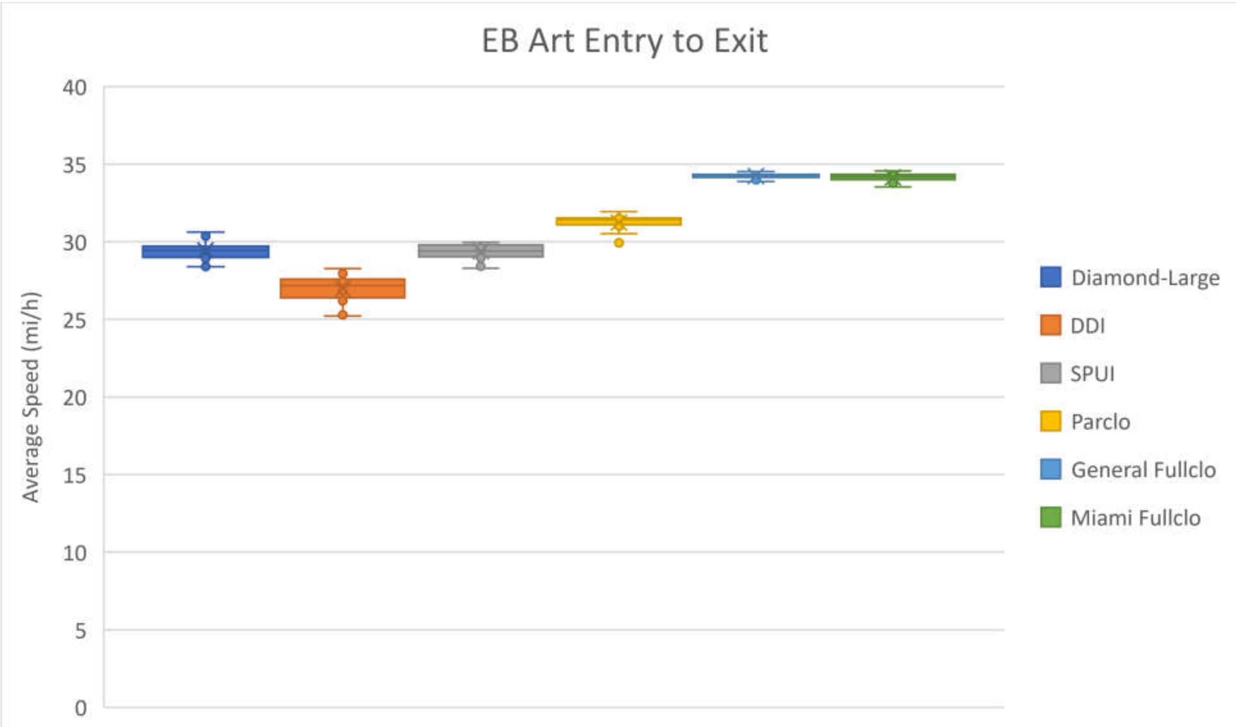


Figure 4-38. Average travel speed for path EB arterial entry to exit

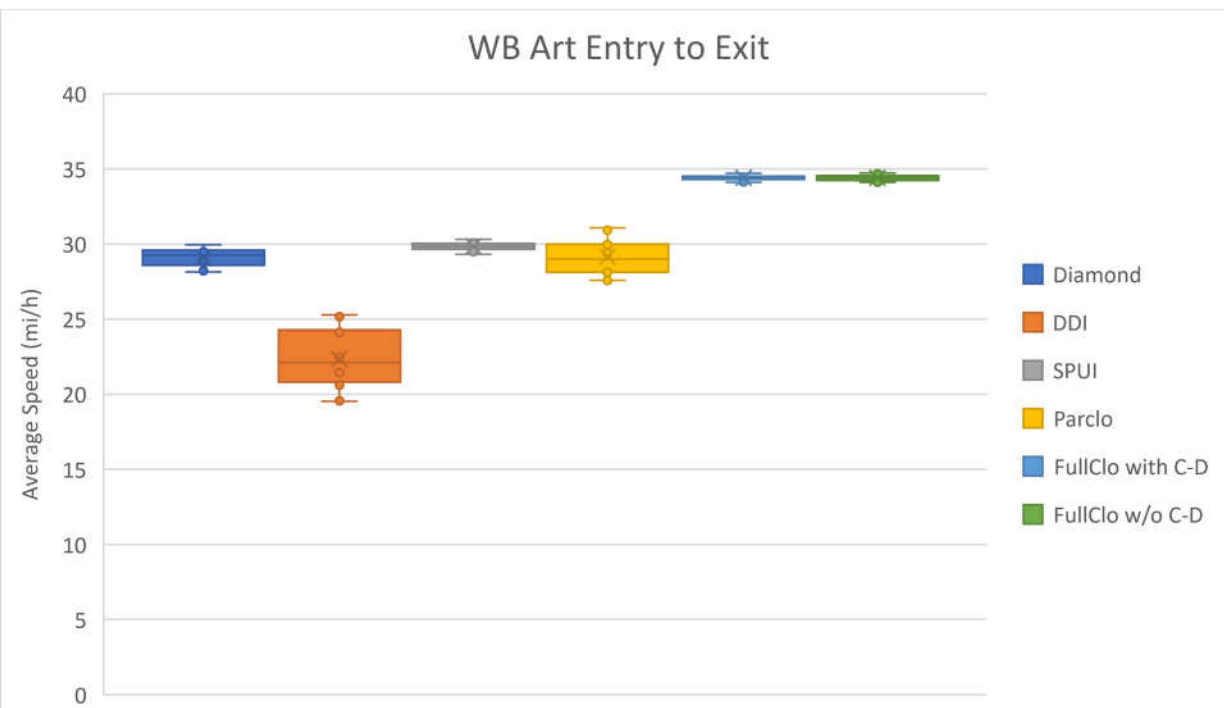


Figure 4-39. Average travel speed for path WB arterial entry to exit

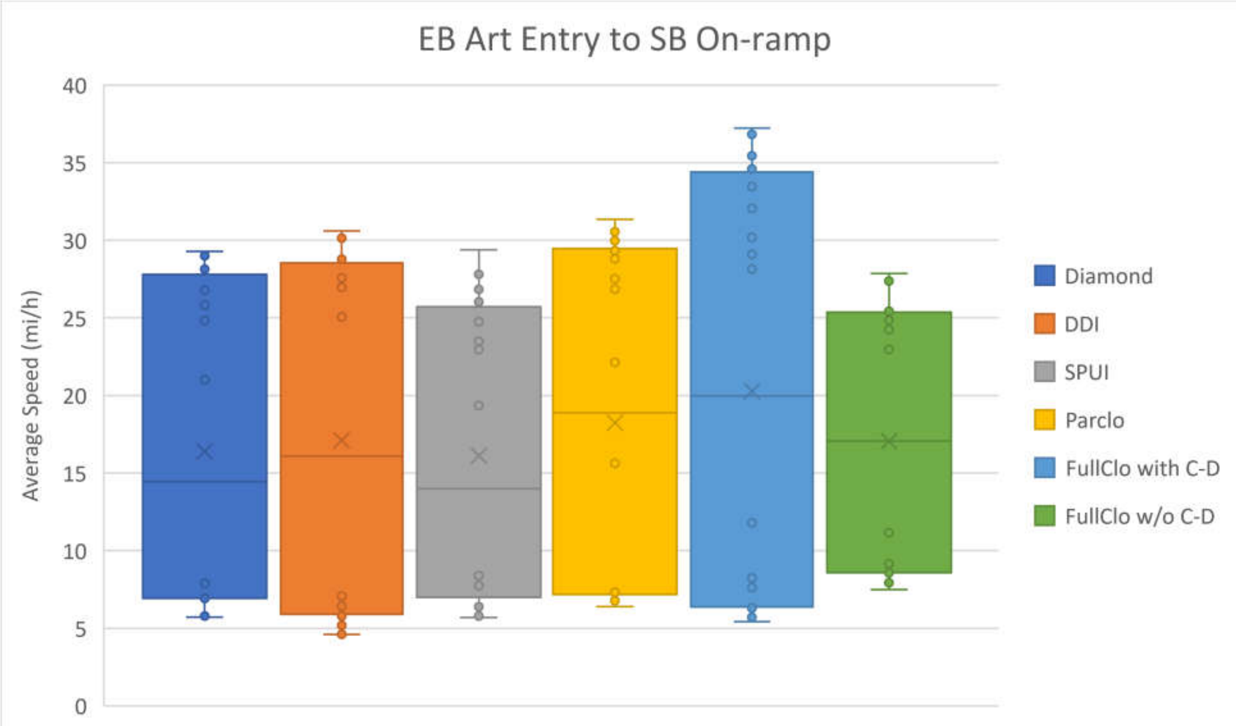


Figure 4-40. Average travel speed for path EB arterial entry to SB on-ramp

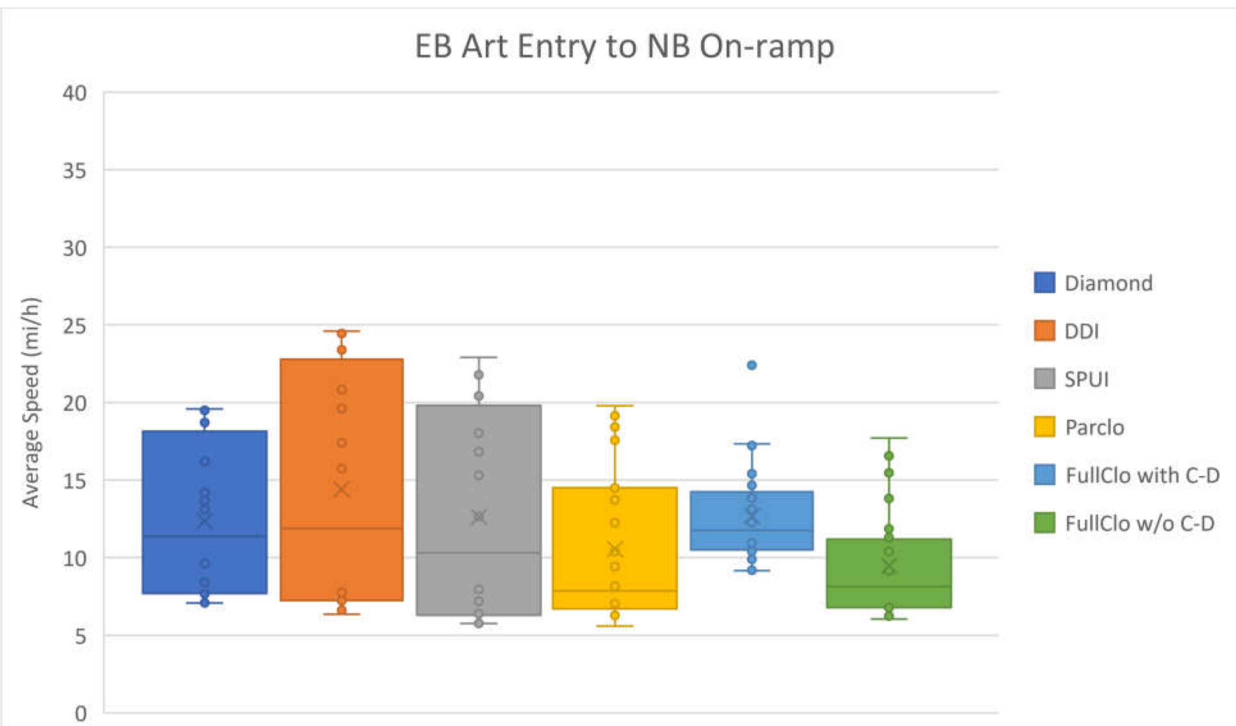


Figure 4-41. Average travel speed for path EB arterial entry to NB on-ramp

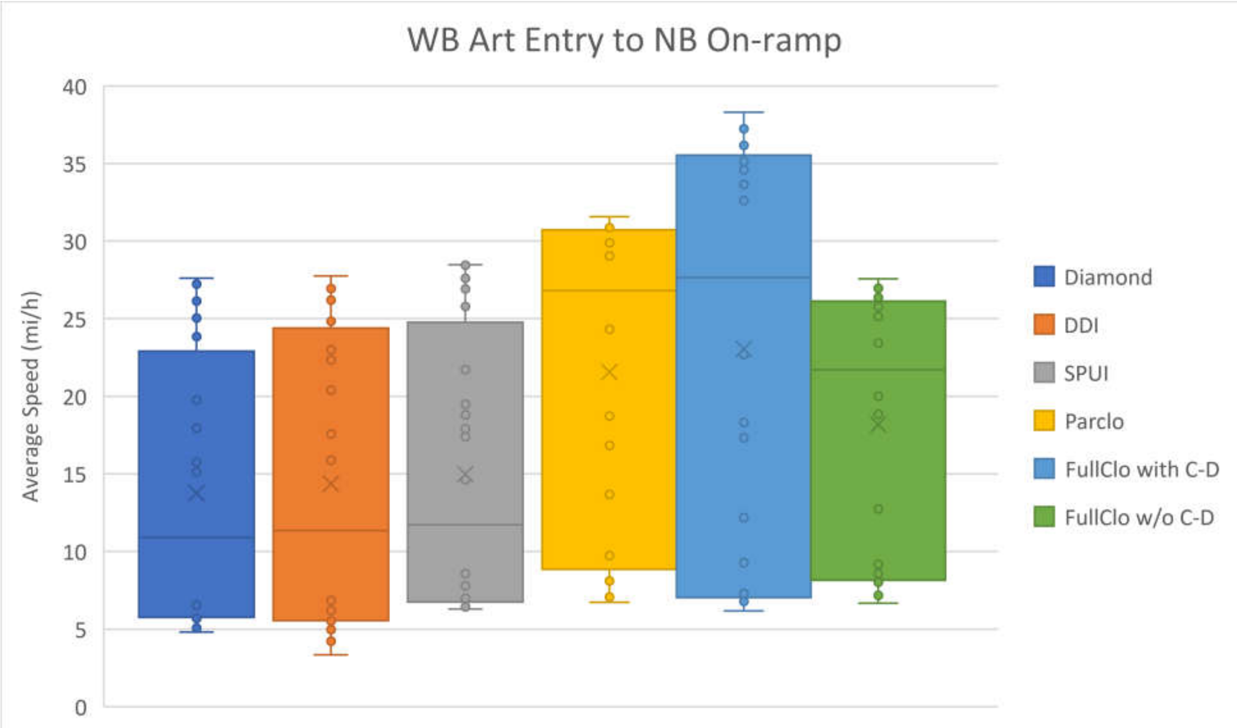


Figure 4-42. Average travel speed for path WB arterial entry to NB on-ramp

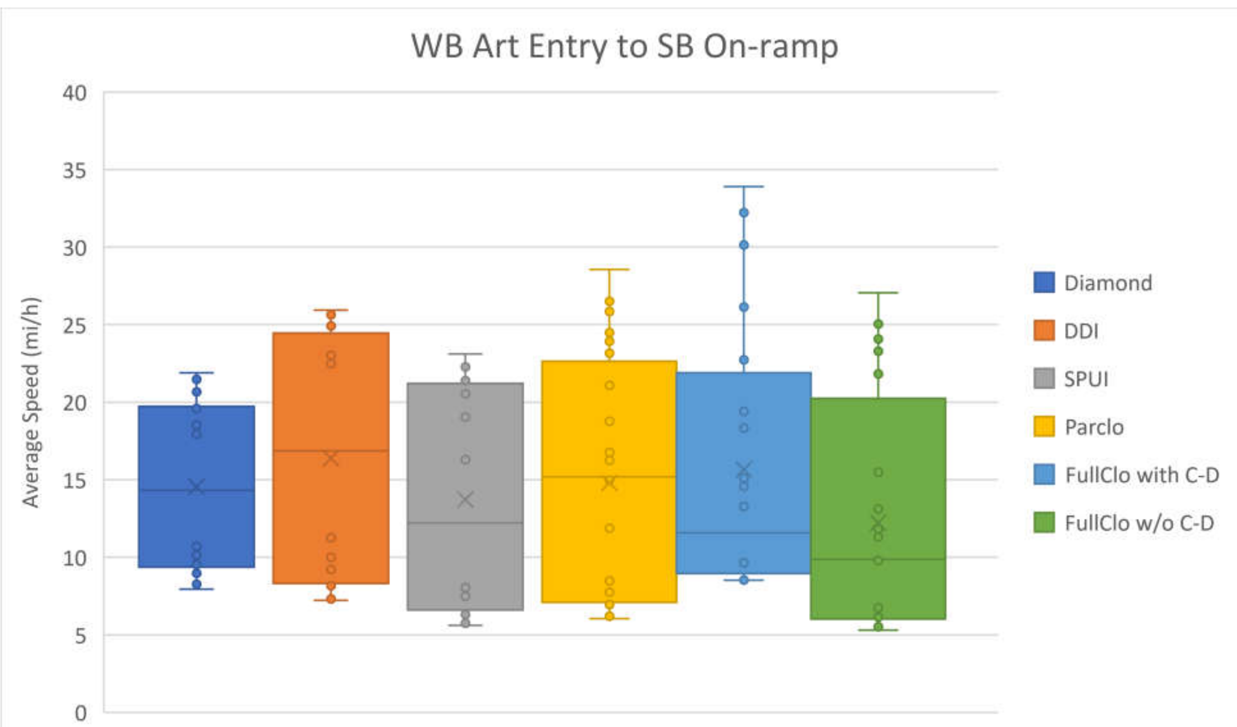
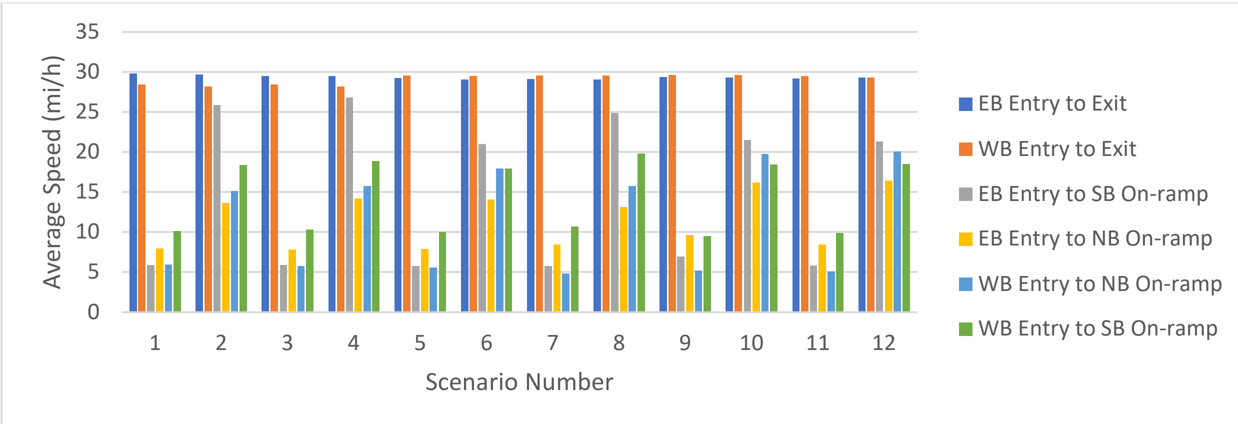
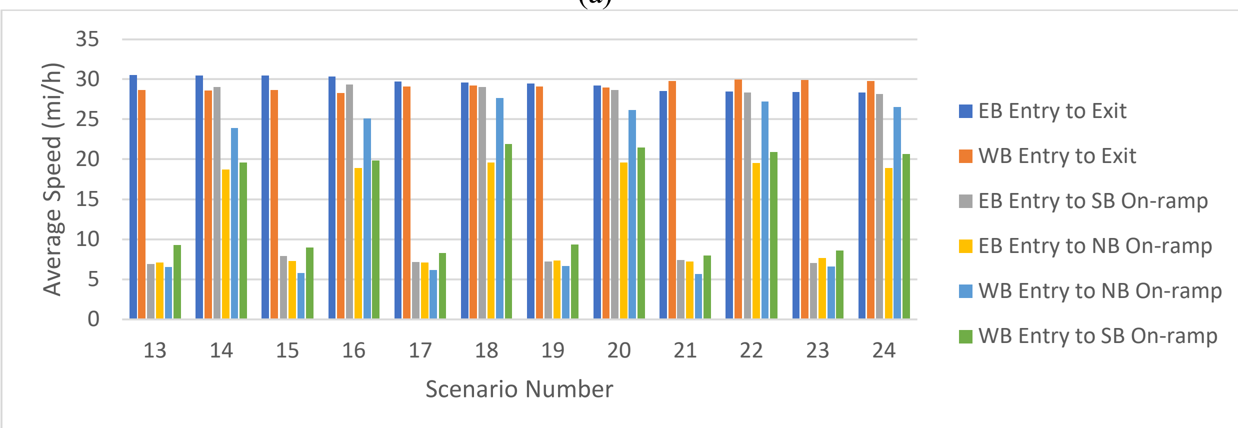


Figure 4-43. Average travel speed for path WB arterial entry to SB on-ramp

Figure 4-44 through Figure 4-49 present the average travel speed for all travel paths for a given interchange, by scenario. Scenarios 1-12 are the high-demand scenarios and scenarios 13-24 are the medium-demand scenarios.



(a)



(b)

Figure 4-44. Average travel speed for all paths for Diamond interchange: (a) high- (b) medium-demand scenarios

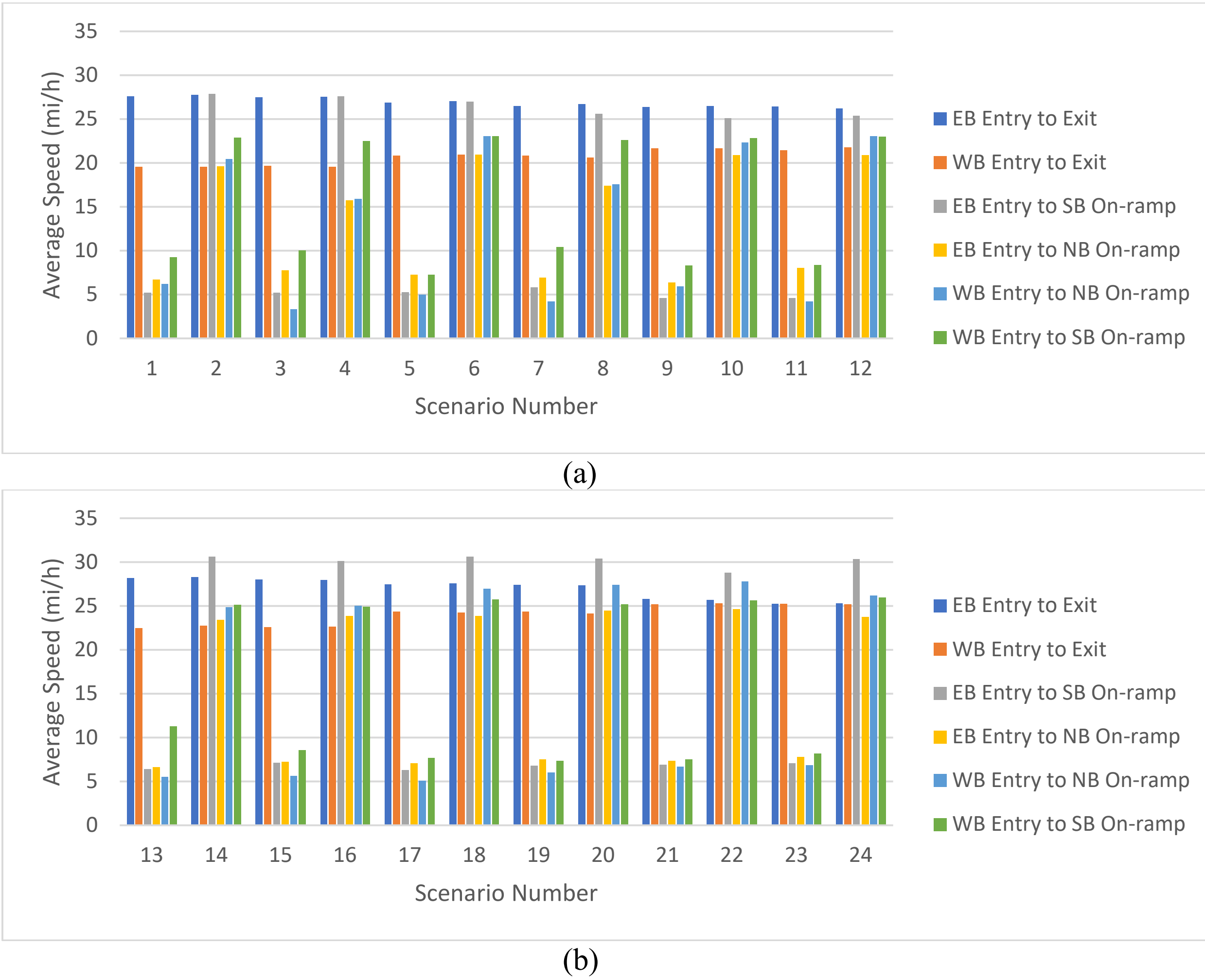
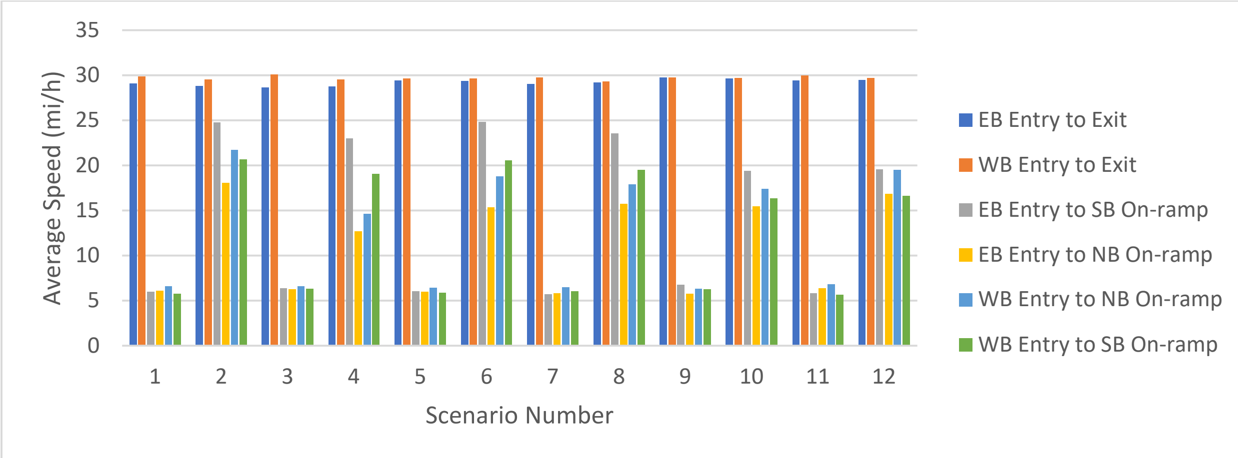
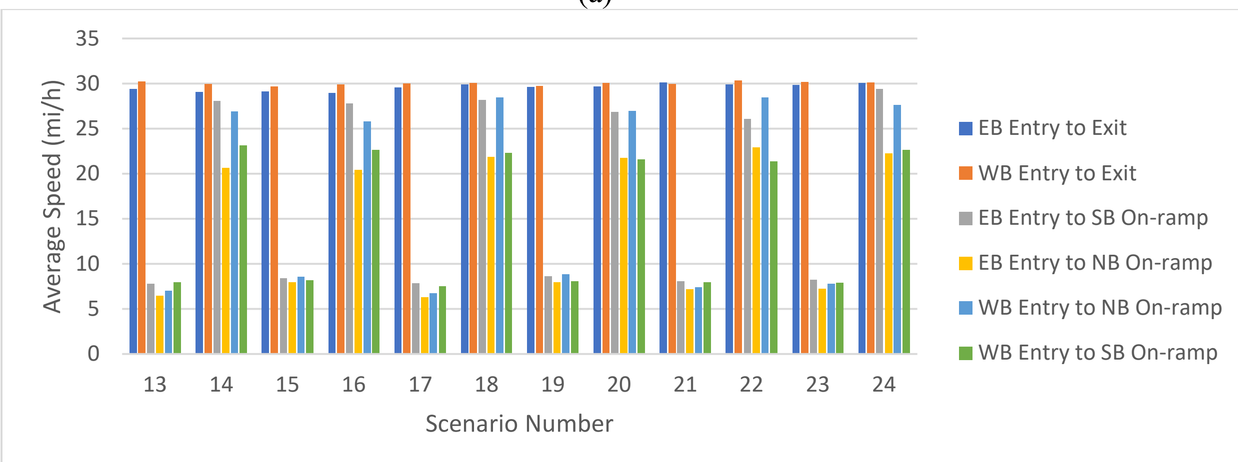


Figure 4-45. Average travel speed for all paths for DDI: (a) high- (b) medium-demand scenarios



(a)



(b)

Figure 4-46. Average travel speed for all paths for SPUI: (a) high- (b) medium-demand scenarios

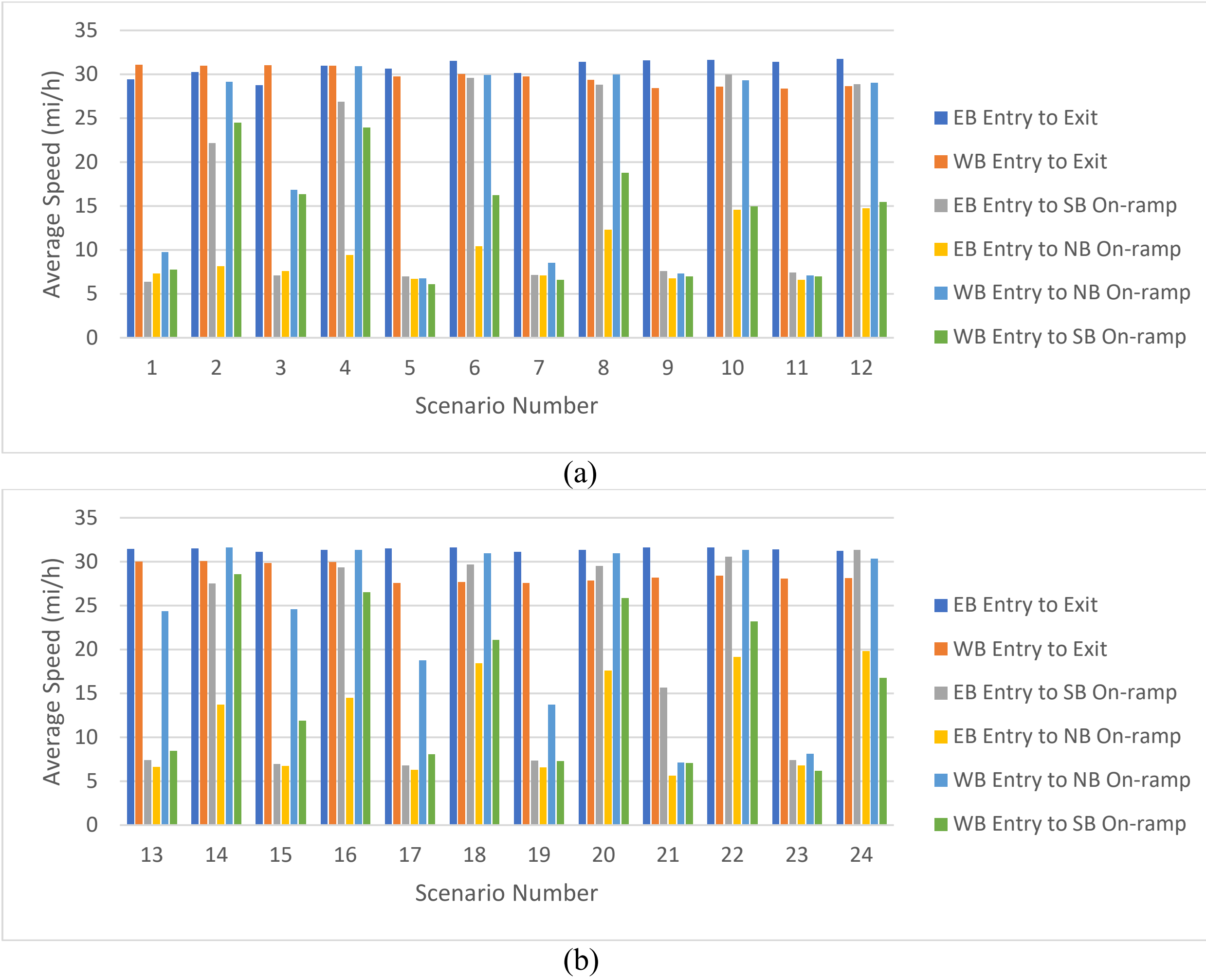
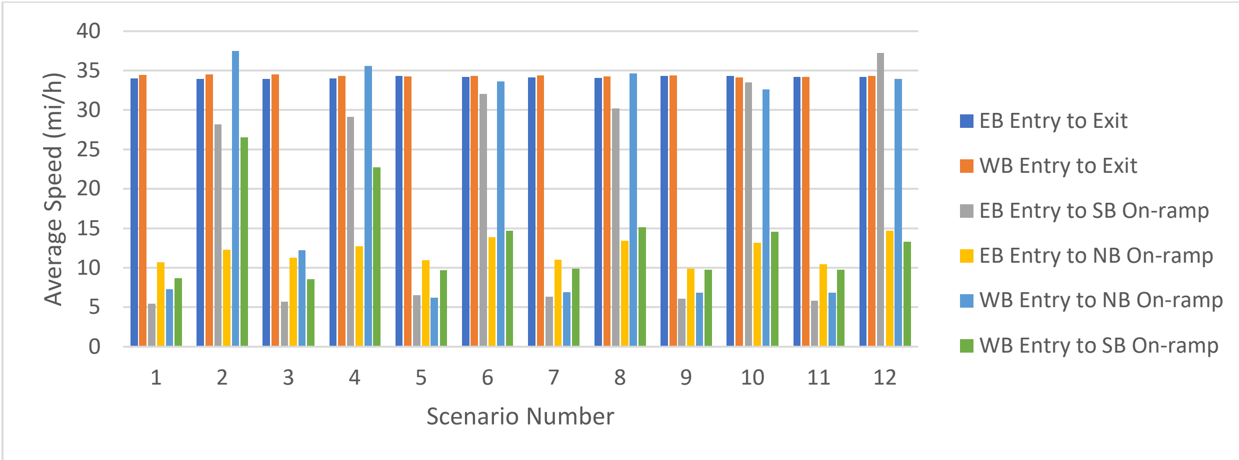
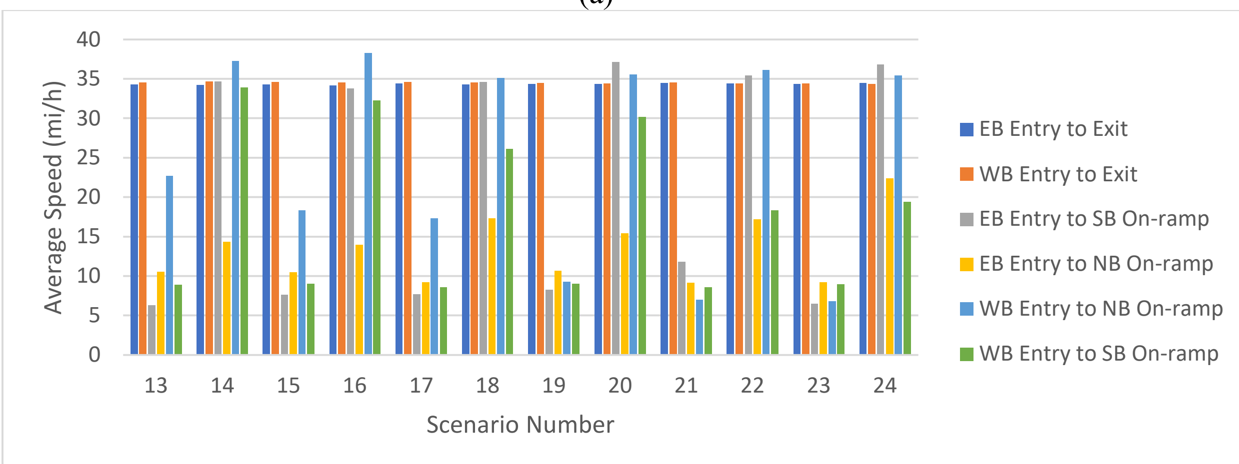


Figure 4-47. Average travel speed for all paths for ParClo: (a) high- (b) medium-demand scenarios

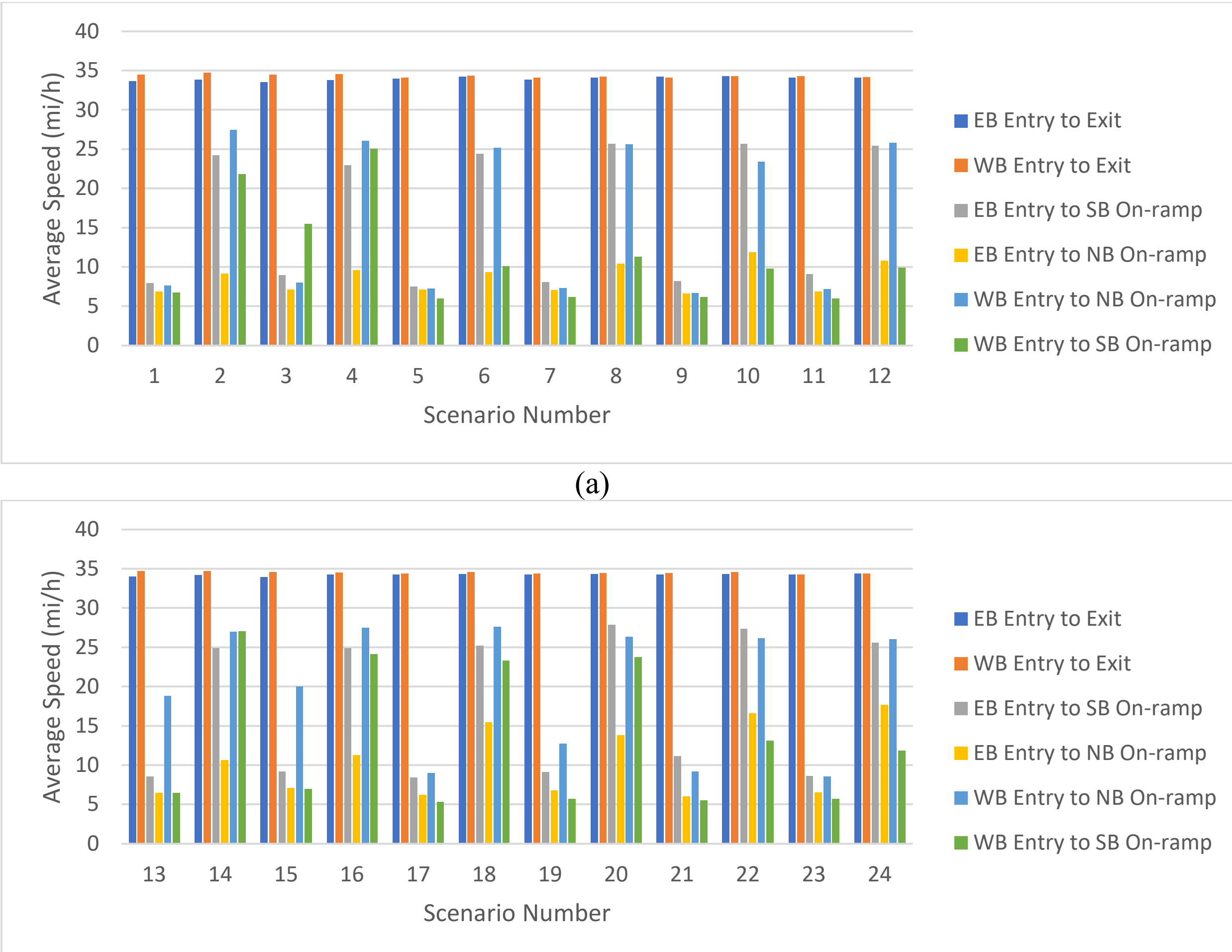


(a)



(b)

Figure 4-48. Average travel speed for all paths for FullClo with C-D: (a) high- (b) medium-demand scenarios



(a)

(b)

Figure 4-49. Average travel speed for all paths for FullClo without C-D: (a) high- (b) medium-demand scenarios

The difference in average travel speed for the arterial through movements (entry to exit) across all interchange forms is minimal. As expected, the average speed for the FullClo designs is higher than the others due to no arterial signals being present within the interchange area. The FullClo without C-D speeds are slightly lower than those for the FullClo with C-D design because of less queue storage and consequently more backup onto the arterial. However, for full cloverleaf designs in urban areas (where ramp metering would be considered), it is likely that signalized intersections would be within a short distance on either side of the interchange area. The DDI travel speeds for the arterial through movements are slightly lower and more variable than the other interchanges with arterial signals due to more friction in the "weaving" area (area between off-ramp and on-ramp termini) than the other configurations and due to less capability for good signal coordination in both arterial directions because of the major traffic streams crossing.

The arterial right-turn movements onto the on-ramp (EB entry to SB; WB entry to NB) for the loop ramp designs (ParClo, FullClo) generally have a higher average speed than for the non-loop ramp designs (diamond, DDI, SPUI) for two reasons.

1. The loop ramp designs have independent lanes for the direct-turning traffic and loop traffic; thus, there is no merging friction with a competing traffic stream.
2. These movements overall had lower demand across all the scenarios than the left-turn movements, and because this lane was controlled separately, it incurred less ramp meter delay.

The lower demand has a much larger impact on the travel speed than the merging friction. This issue is discussed further in the conclusions chapter.

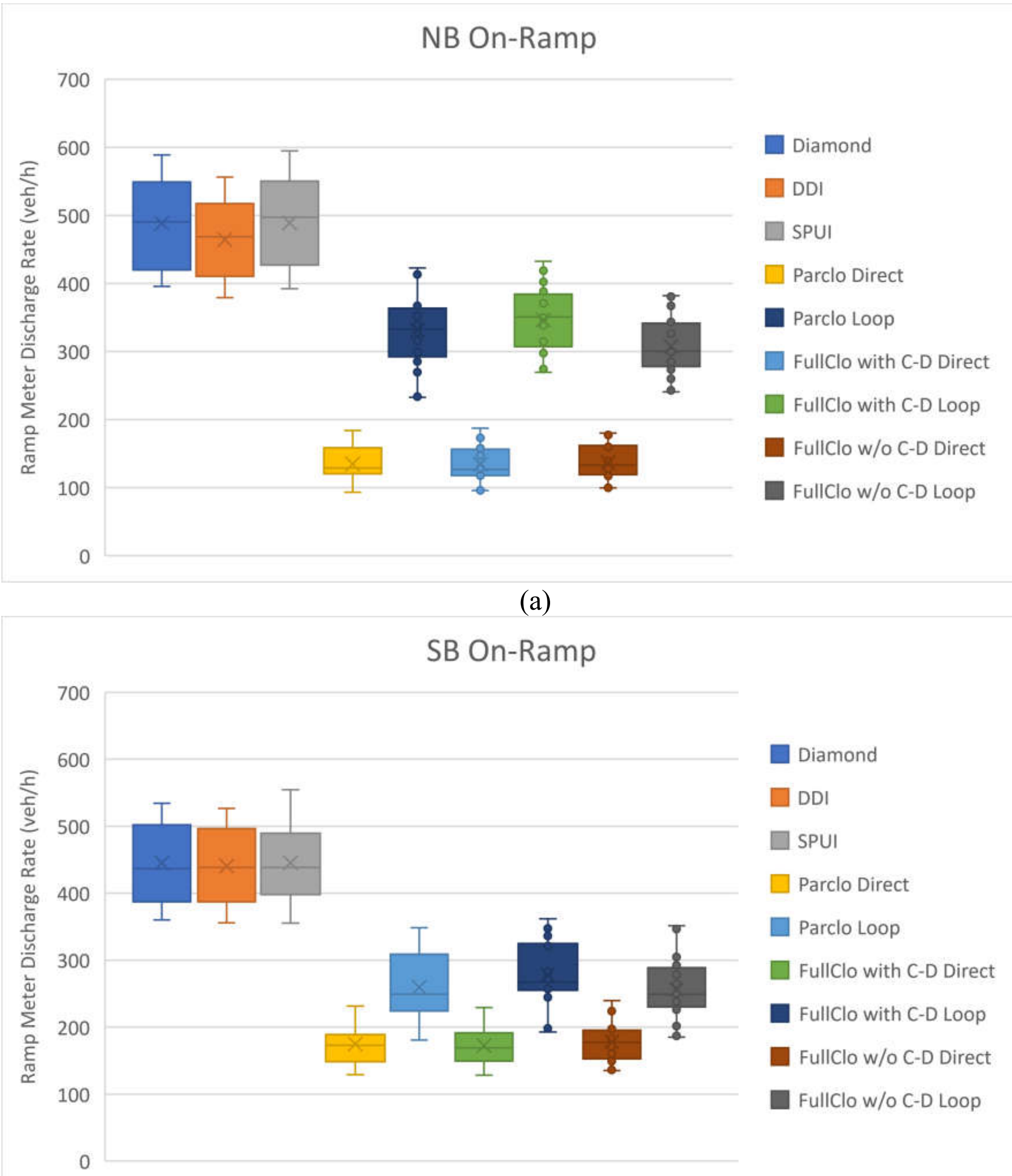
The arterial left-turn movements onto the on-ramp (EB entry to NB; WB entry to SB) for the loop ramp designs (ParClo, FullClo) also benefit from reduced friction due to a dedicated lane. However, the average speeds are generally lower than for the non-loop ramp designs (diamond, DDI, SPUI) because of the higher demand for this movement, relative to the right-turn, not being able to utilize additional queue capacity of two shared ramp lanes. That is, for the non-loop ramp designs, left- and right-turning traffic share much of the queue storage of two lanes on the ramp, whereas this is not possible for the loop-ramp designs because the ramp lanes for left- and right-turning traffic are geometrically separated. Again, this demand balance issue is discussed further in the conclusions chapter.

Generally speaking, the best performing loop ramp design is the ParClo interchange. Similarly, out of the three non-loop ramp designs (diamond, DDI, SPUI), the DDI generally performs best. An advantage of the DDI and SPUI relative to the diamond is that the left-turning (to the on-ramp) movement only must pass through one signalized intersection within the interchange area. Furthermore, the DDI has an additional advantage over the SPUI because it has only two signal timing stages instead of three.

On-ramp queuing

Average ramp meter queue discharge rate

The ramp meter queue discharge rates across all scenarios, as measured by passage detectors downstream of the ramp meter stop bar, are given in Figure 4-50.



(b)

Figure 4-50. Ramp meter discharge rates: (a) NB on-ramp (b) SB on-ramp

The NB ramp values are higher overall than the SB ramp values because the NB ramp received higher demand for the 65/35 and 55/45 directional split scenarios (2/3 of the total scenarios). For the ParClo and FullClo designs, the loop ramps experienced higher discharge rates than the direct right-turn ramps because the loop ramps served the higher demand. For a given ramp, a lower discharge rate generally means that the meter spent less percentage of time in the queue override modes (intermediate and/or advance). This is most common for the medium demand and medium metering rate scenarios. The percent time spent in each metering mode results are provided in the following section. The specific values for average metering rate for each scenario for each interchange are provided in Table C-1 and Table C-2 of Appendix C.

Average % of time ramp meter spends in each operation mode

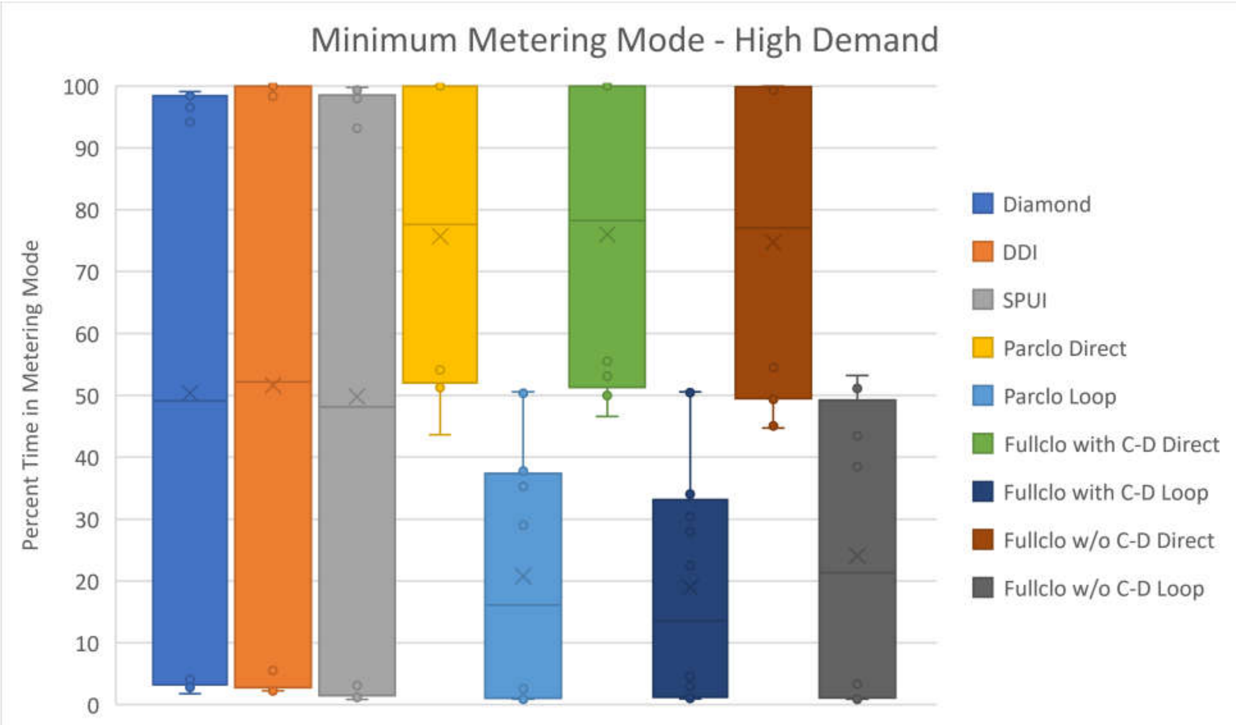


Figure 4-51. Percent time spent in base metering mode, high-demand scenarios

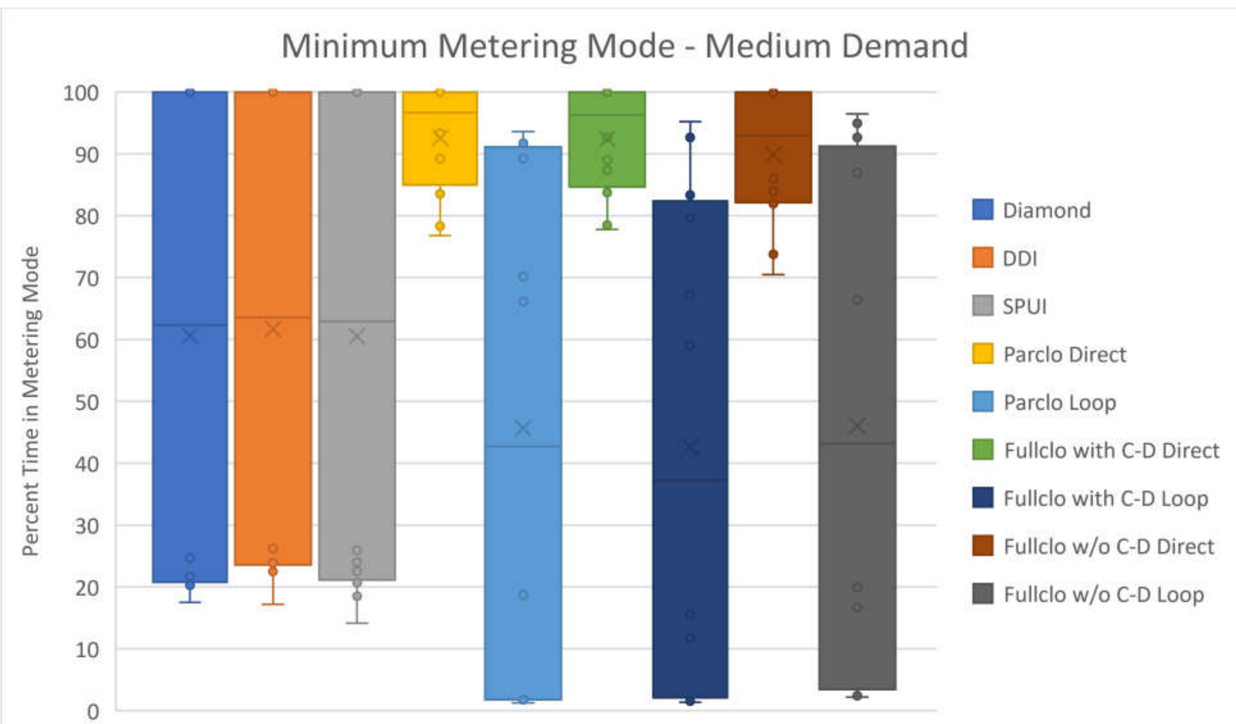


Figure 4-52. Percent time spent in base metering mode, medium-demand scenarios

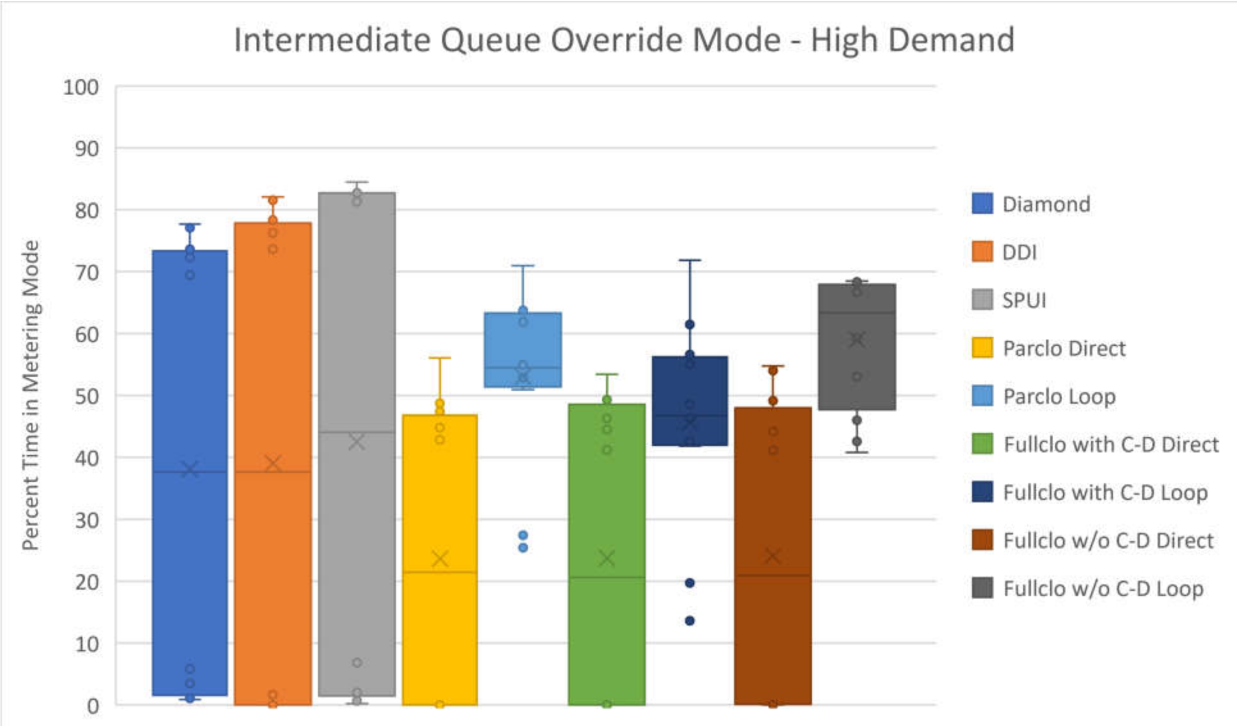


Figure 4-53. Percent time spent in intermediate queue override metering mode, high-demand scenarios

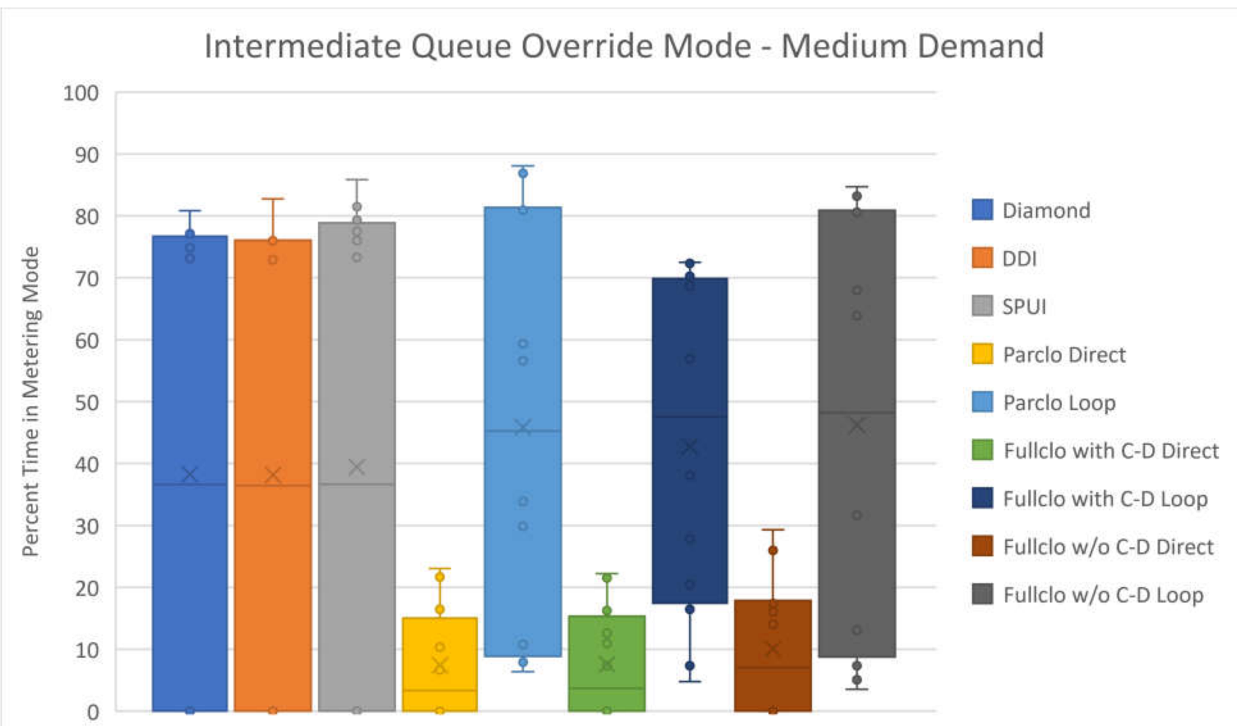


Figure 4-54. Percent time spent in intermediate queue override metering mode, medium-demand scenarios

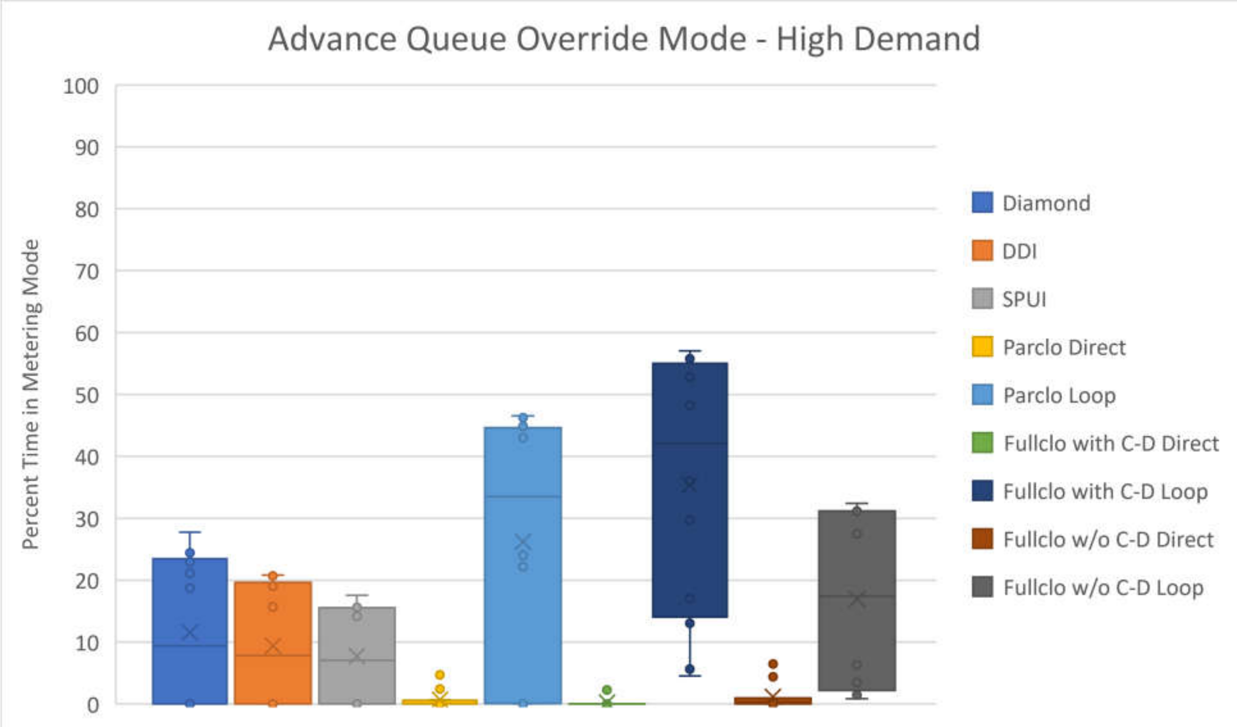


Figure 4-55. Percent time spent in advance queue override metering mode, high-demand scenario

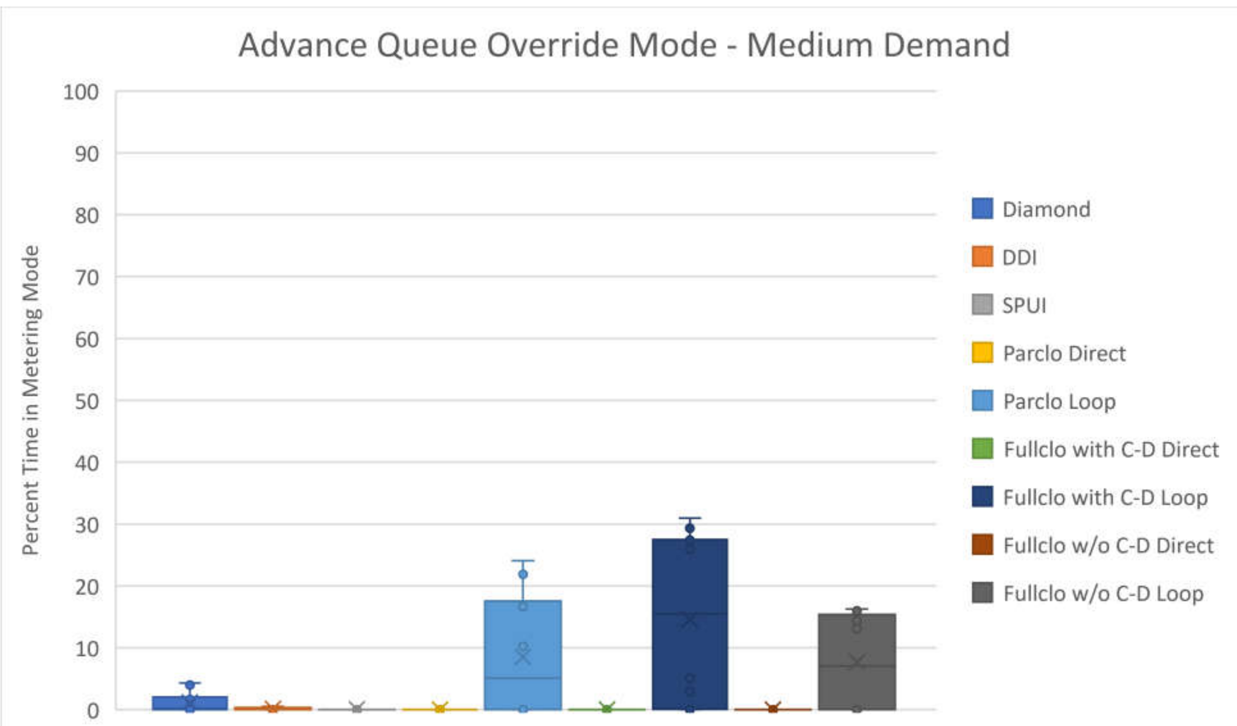
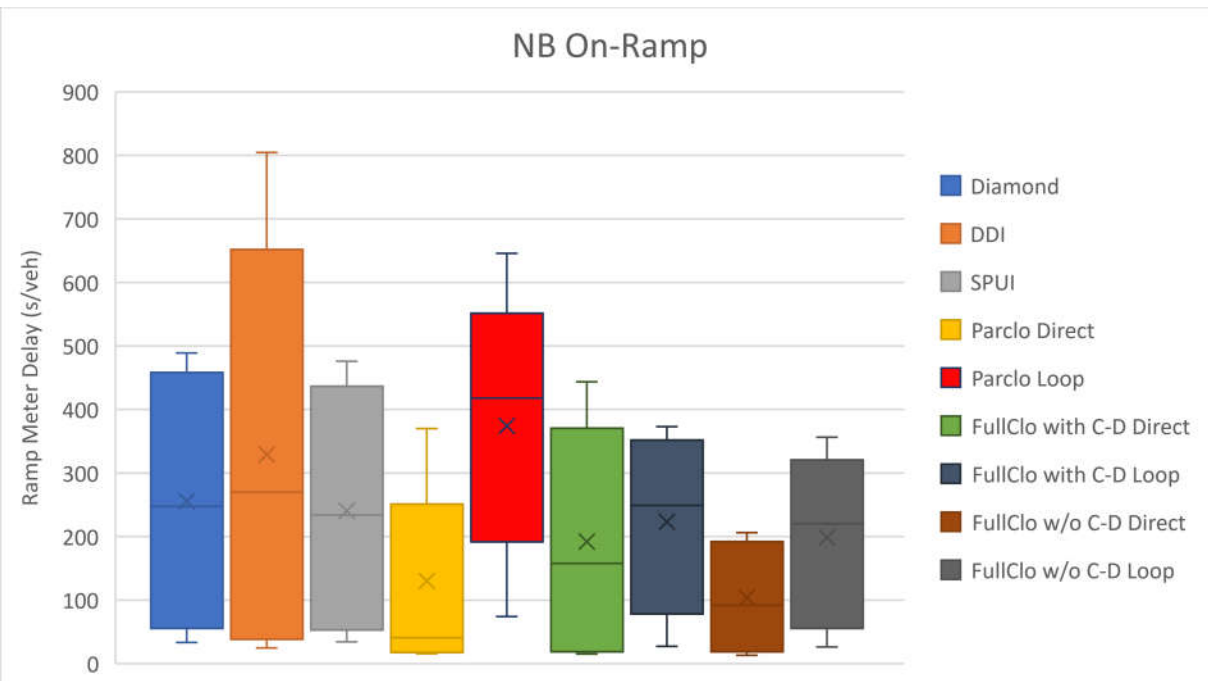


Figure 4-56. Percent time spent in advance queue override metering mode, medium-demand scenario

Because of overall similarities in the on-ramp configurations for the Diamond, DDI, and SPUI interchange, the median values and total variability are comparable. As expected, higher percentages spent in intermediate and advance queue override modes correspond to the lower metering rate and high demand scenarios. The DDI has a small advantage over the Diamond and SPUI designs in that the arrival pattern to the on-ramp is more uniform because the left-turn movement is not controlled by a signal at the ramp entrance point. Surges in arrivals from signal-controlled movements can produce more frequent activations of the queue detectors. This can be counteracted to some extent through the queue detector occupancy threshold and timer settings.

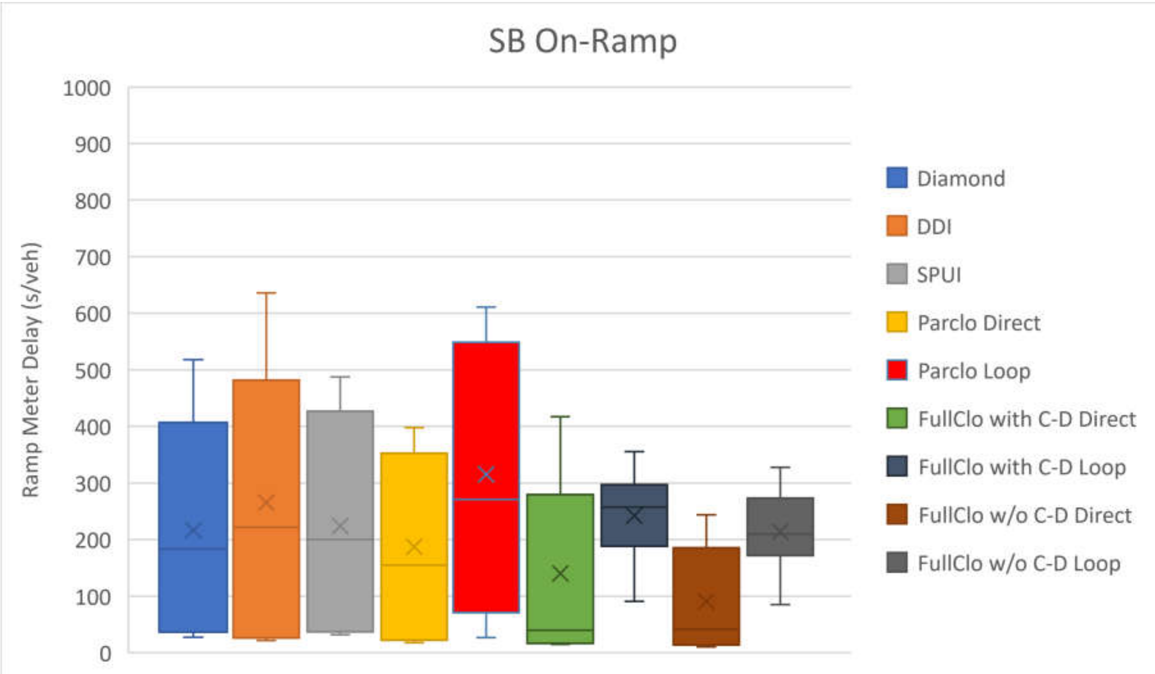
For the ParClo and FullClo designs, there is a significant difference between the loop and direct ramps. Again, this is due to the higher demand volumes for the loop ramps. The lower demands for the direct right-turn ramp lanes results in a consistently high percentage of time in the minimum metering mode. This imbalance in metering modes between the loop and direct ramp lanes is not necessarily desirable, which is discussed further in the conclusions chapter. A more detailed breakdown of the ramp metering mode results is provided in Appendix C.

Average on-ramp metering delay



(a)

Figure 4-57. Ramp meter average delay: (a) NB on-ramp



(b)

Figure 4-57. Ramp meter average delay: (a) NB on-ramp (b) SB on-ramp

Overall, the relationships between the results for the different interchange forms are similar for the ramp meter delay as for percent time spent in a given metering mode, as expected. For the non-loop ramp designs, the Diamond and SPUI results are very similar. The DDI has more scenarios with higher delay than the Diamond and SPUI because it has a few hundred more total lane-feet of queue storage. The ParClo and FullClo designs have significant differences between the loop and direct ramp lanes, again due to the imbalance in ramp demand/queuing. The specific values for average metering delay for each scenario for each interchange are provided in Table C-3 and Table C-4 of Appendix C.

5. CONCLUSIONS AND RECOMMENDATIONS

The material in this chapter is intended to largely be incorporated into the Interchange Access Request User's Guide (FDOT, 2018).

On-Ramp Configuration

Location of Ramp Meter Stop Bar

A key consideration in the placement of the ramp meter stop bar is the distance needed by a vehicle to accelerate from a stop to the desired freeway merging speed. Based on the review of the literature, it was found that most recommended acceleration distances are based on passenger-car performance or outdated commercial-truck characteristics. The research team examined acceleration distances for modern truck characteristics within two simulation tools and found the results from the TruckSim program (Mechanical Simulation, 2020) to be most realistic. The developed acceleration distances are based on a FHWA Class 9 vehicle (tractor + semi-trailer), and for two different total weights: 53,000 lb and 80,000 lb. The 53,000-lb weight was chosen based on an analysis of Florida weigh-in-motion data in previous studies (Washburn. and Ozkul (2013), and Ozkul (2014)). The 80,000-lb weight is the maximum legal load without a permit. For a conservative design, it is recommended to use the acceleration distances corresponding to the 80,000-lb truck. The recommended acceleration distance values are provided in Table 5-1 and Table 5-2.

Table 5-1. Minimum acceleration lengths (ft) for 53,000-lb trucks

Merging speed (mi/h)	**Percent Grade**			
	–2	0	+2	+4
23	205	233	271	327
27	298	355	430	558
31	396	468	585	803
35	506	612	790	1163
39	691	863	1187	2116
43	862	1105	1597	3472
47	1065	1400	2140	7321
50	1305	1772	2972	-
53	1517	2114	3868	-
55	1671	2369	4615	-

Table 5-2. Minimum acceleration lengths (ft) for 80,000-lb trucks

Merging speed (mi/h)	Percent Grade			
	−2	0	+2	+4
23	242	291	372	534
27	355	452	617	1069
31	482	617	894	1925
35	627	827	1278	4370
39	851	1175	2064	-
43	1073	1537	3036	-
47	1335	1982	4548	-
50	1619	2509	9355	-
53	1888	3041	-	-
55	2082	3440	-	-

Multilane Metering

Geometric design and operation of multilane metered on-ramps is not discussed comprehensively in any of the existing ramp-metering design manuals. Overall, very few states have guidance on multilane metering. Chapter 2 (Literature Review) provided some guidance on multilane metering, primarily for two lane on-ramps. Appendix D provides some supplemental information regarding metering for three on-ramp lanes. Overall, Caltrans has the most comprehensive guidance on warrants and operation of multilane metering. The Caltrans guidance can be applied in Florida as appropriate. In general, the key issues to consider for multilane metering applications are geometric design of the metered lanes and/or bypass lanes, signage and signaling configurations, and vehicle release patterns.

Interchange Performance

From a conceptual perspective, the interchange area can be considered a simple input-output system. Inputs consist of arterial entry traffic and freeway exiting traffic (off-ramps). Outputs consist of arterial exiting traffic and freeway entry traffic (on-ramps).

The challenge is to provide enough capacity for the respective movement demands in order to avoid significant flow interruptions and delays. The presence of signalized intersections along the arterial increases this challenge relative to interchange configurations without arterial signals, such as a full cloverleaf design. Introducing ramp metering to an interchange significantly increases the complexity, because arterial operations must now be considered alongside the freeway's metering operations.

During peak travel periods, when ramp metering is activated, the metering rate (on-ramp output) is typically lower than on-ramp demand. This results in queuing on the on-ramp that has the potential to back up to the adjacent arterial roadway. Queue backup onto the arterial is a very undesirable situation for both operational and safety reasons. To avoid these issues, modern ramp metering systems usually employ an advance, and sometimes an intermediate, queue detector. These queue detectors essentially act as pressure-relief values for the on-ramp—increasing the ramp output when the queue backup approaches the arterial. Of course, increasing the on-ramp output can have negative implications for the freeway operations, creating a cycle of self-aggravating consequences for both the arterial and freeway operations.

The queue buildup and dissipation process is a function of the rate of traffic flow entering the on-ramp, the base metering rate, and adjustments to the metering rate based on intermediate or advance queue occupancy levels. The arrival pattern of vehicles at the on-ramp can also influence the queuing process. The arrival pattern is influenced by the demand from the feeding movements (e.g., left turn, right turn) and the type of control for the movement (e.g., signalized, free). Generally, for a given on-ramp demand level, decreasing the queue storage will lead to an increased percentage of time the ramp meter operates in queue override modes, which—by increasing the ramp metering rate—decreases the ramp meter control delay and increases the average travel speed and throughput of the ramp, with potentially adverse effects to the freeway mainline.

The simulation results generally conform to these expectations. However, not everything was perfectly equal in the setup of the various interchange forms, as the configuration was significantly influenced by existing design in the field. Thus, direct comparison of results across interchange forms must also be considered in the context of specific configuration values (e.g., queue storage differences). Nonetheless, there is not necessarily a one-size-fits-all interchange configuration when ramp metering is included. Depending on the local traffic characteristics, some designs may work better than others. Some designs may lend themselves to more easily avoid problems created by on-ramp queuing backups, such as conflicting movement blockages. Issues such as demand directional balance, queue storage allocation relative to left/right turn on-ramp demand, travel path for left-turn movements, and presence of weaving areas factor into a given interchange configuration's ability to mitigate or exacerbate on-ramp queue spillback issues. These issues are discussed relative to each of the interchange configurations considered in this study in the remainder of this section.

The six interchange forms considered in this study—diamond, DDI, SPUI, ParClo, FullClo with C-D, and FullClo without C-D—can broadly be grouped into one of two categories: (1) designs with loop ramps and (2) designs without loop ramps. Thus, the discussion will first compare and contrast designs within each of these categories.

Designs with loop ramps

In a review of interchanges with ramp metering throughout the U.S., while not exhaustive, a relatively small percentage of them were partial or full cloverleaf interchanges. One of the primary reasons for this is that cloverleaf interchange designs are generally less prevalent along urban freeway corridors, where ramp metering would be deployed. These designs, particularly full cloverleaf designs, are less desirable for urban applications due to their relatively large right-of-way footprint. Nonetheless, these interchange forms still occur in urban areas because they may have been built within freeway sections that originally passed through less developed areas.

Full Cloverleaf

A significant benefit to the full cloverleaf (FullClo) design is the lack of signalized intersections within the interchange area, especially if the interchange is located in a rural area and other signalized intersections are not in close proximity. However, when such an interchange configuration is located in an urban area, it is likely that there will be signalized intersections within a short distance on either side of the interchange area. Furthermore, the delay at an on-ramp signal during peak conditions may be considerably higher than delay experienced at a signalized intersection along the arterial mainline. In some situations, not having signals within the interchange area may reduce the options to achieve certain traffic arrival patterns to the on-ramps.

When metering is implemented in a FullClo design, the weaving areas, along both the freeway and the arterial, complicate the accommodation of queuing. The design with a collector-distributor (C-D) lane potentially provides more queuing storage for the loop-ramp movement, as the meter can be located downstream on the C-D, near the merge point with the direct (right-turn) ramp lane. However, care must still be taken to prevent a situation where the queue blocks the loop off-ramp movement (see Figure 5-1). Note that the advance queue detector is located about 25 feet upstream of the gore point between the C-D lane and off-ramp loop. The queue may still extend beyond this point, depending on the arrival rate, and the occupancy threshold for the detector. As seen in Figure 5-1, an extensive queue on the C-D lane reduces the weaving area between the on and off ramp movements, creating both operational and safety issues.

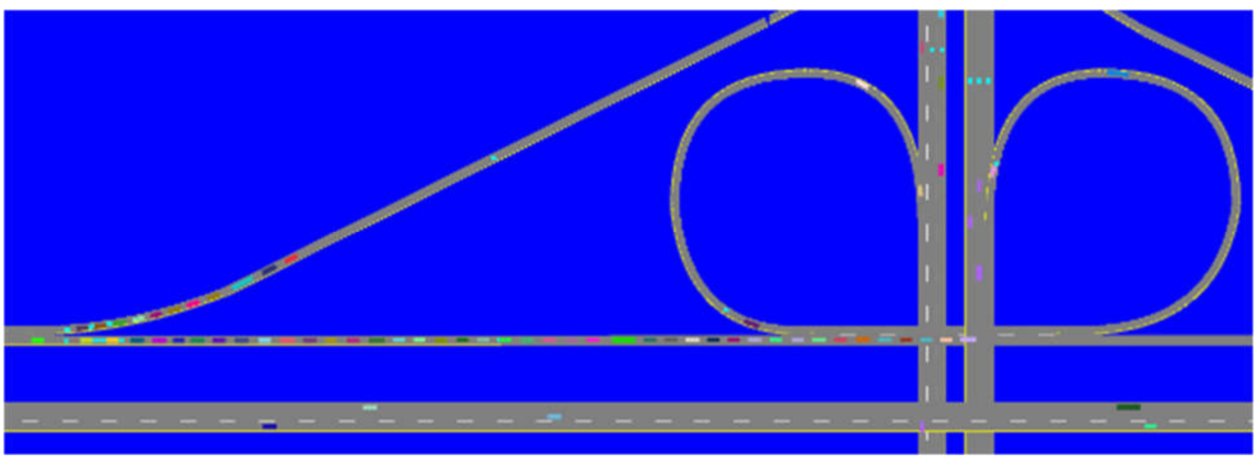


Figure 5-1. Loop ramp queue backup at FullClo with C-D. The advance detector is located on the C-D slightly upstream from the loop off-ramp.

In the FullClo design without a C-D, the loop ramp meter location must be placed on the loop-ramp itself. This avoids the potential complication of the on-ramp queue blocking the loop off-ramp movement but will also provide less queue storage. In this situation, care must be taken to prevent the on-ramp queue from blocking traffic exiting from the loop ramp (for the opposite direction) onto the arterial (Figure 5-2).

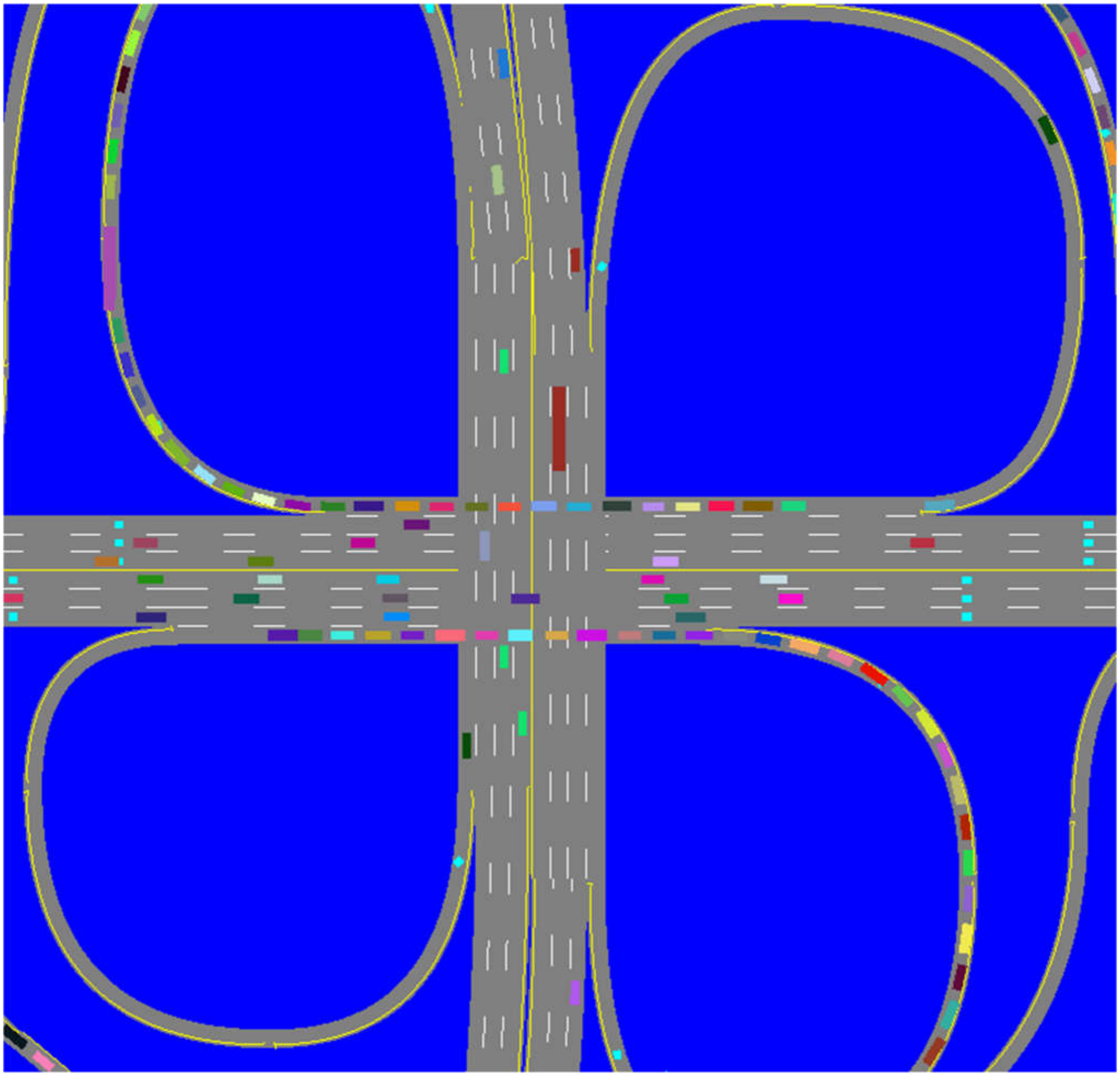


Figure 5-2. Potential off-ramp exit blockage due to loop on-ramp queue at FullClo without C-D (freeway runs north-south, arterial east-west)

Partial Cloverleaf

The partial cloverleaf (ParClo) design has the potential disadvantage relative to the full cloverleaf in that it has two signalized intersections within the interchange area. Those signals, however, operate with only two timing stages and their contribution to vehicle delay will typically be much lower than the delay created by the ramp meter. The advantage of the ParClo over the FullClo in a ramp-metering situation is the potential for more queue storage along the loop ramp due to the lack of weaving areas.

Since ramp metering is only implemented in urban freeway interchange situations, potential benefits from the lack of internal arterial signals with the FullClo design are largely offset by nearby signals and ramp metering delay. Thus, the ParClo, even with its internal signals, is a better option for ramp metering applications than FullClo designs due to the lack of weaving areas within the interchange area.

An advantage of all loop ramp designs is that traffic on the far side of the arterial accesses the on-ramp via an uncontrolled right turn, rather than a signal-controlled left turn. A significant disadvantage to the loop-ramp designs relative to the non-loop ramp designs is the added complexity of trying to coordinate queuing operations between the separate loop and direct ramp lanes for each on-ramp. An example of unbalanced queue operations is also illustrated in Figure 5-1. This situation can lead to equity perception issues. The metering algorithm can be tailored beyond the queue override settings to set metering rates at respective ramps to balance meter queue lengths/delays, but drivers may not understand that a lane has a faster metering rate because of higher demand and more queuing. Such settings to account for ramp demands at a fine-grained level adds complexity to the detection setup and algorithm. However, if drivers perceive significant inequity among individual lanes of an on-ramp, some will choose to make a U-turn to access the ‘other’ on-ramp lane. This may also occur in situations where the on-ramp queue blocks the loop off-ramp.

Some existing ParClo interchanges are configured like the one shown in Figure 5-3, located in Sarasota, FL (27°20'17.96"N, 82°26'48.67"W). Unless the loop ramp demands are expected to be only moderate, however, the loop ramp should be extended to tie into the right-turn ramp lane, in order to provide sufficient queue storage, such as shown in Figure 5-4.

Figure 5-3. Existing ParClo interchange along I-75 in Sarasota, FL

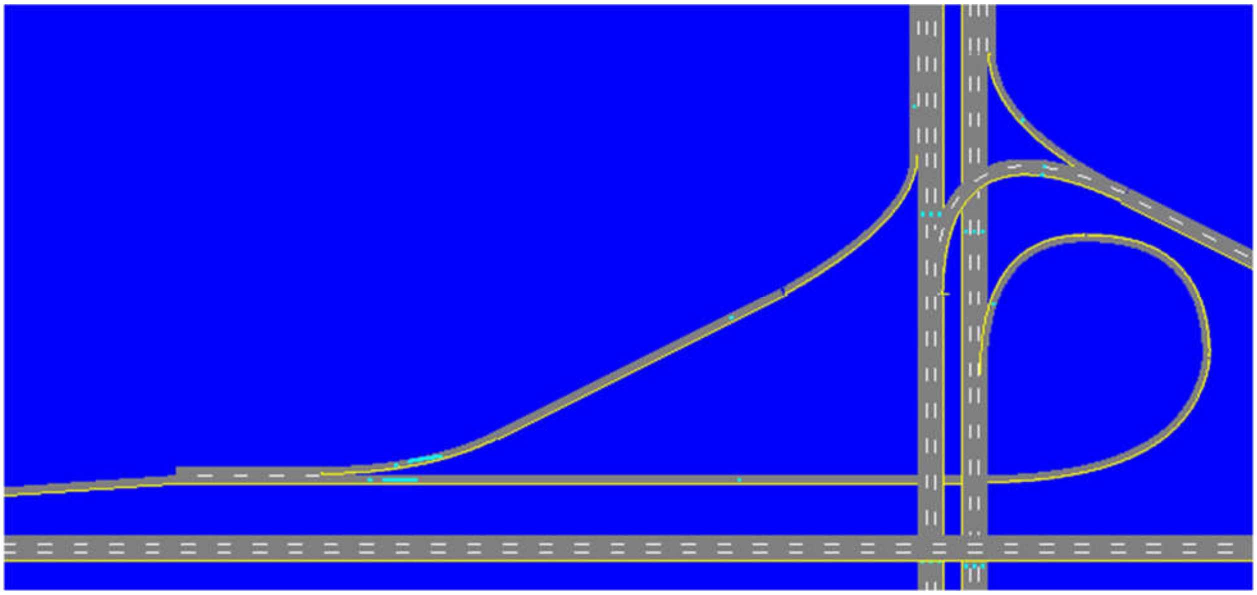


Figure 5-4. ParClo loop ramp design to accommodate ramp metering

Designs without loop ramps

A potential advantage of the Diamond, DDI, and SPUI designs relative to the loop-ramp designs is that the right- and left-turning movements use the same on-ramp lanes. This provides for driver-balancing of the queue length and eliminates queue balance challenges discussed previously. These designs also use a more compact geometric footprint.

Diamond

With the diamond configuration, a minimum of three timing stages is needed at each intersection, one of which is used to serve the left turn movement from the arterial to the on-ramp (see Table 4-1). This configuration leads to a surging (or ebb and flow) pattern with the left-turn arrivals to the on-ramp. The setting of queue detection parameters to avoid oscillation of ramp metering rates, as illustrated in Figure 5-5 and Figure 5-6, can be more challenging. Note that updates to the metering rate values are based on the previous 1-min aggregated detector measurements.

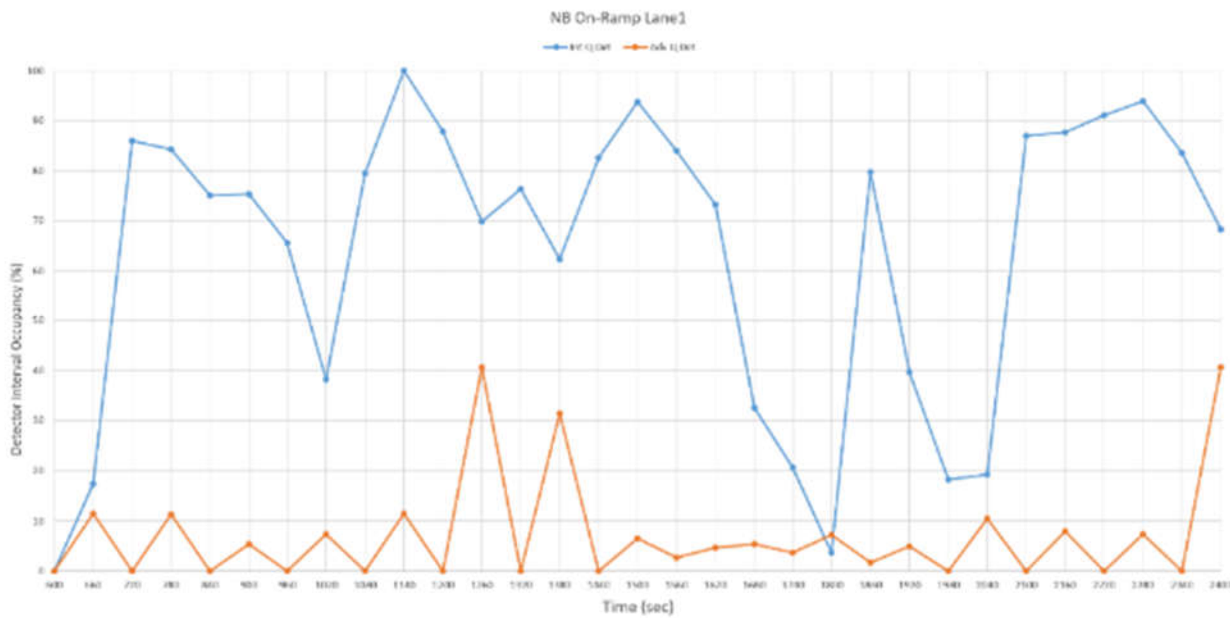


Figure 5-5. Detector interval occupancy (%) vs. time (seconds) [Diamond] (based on 1-min aggregation detector measurements)

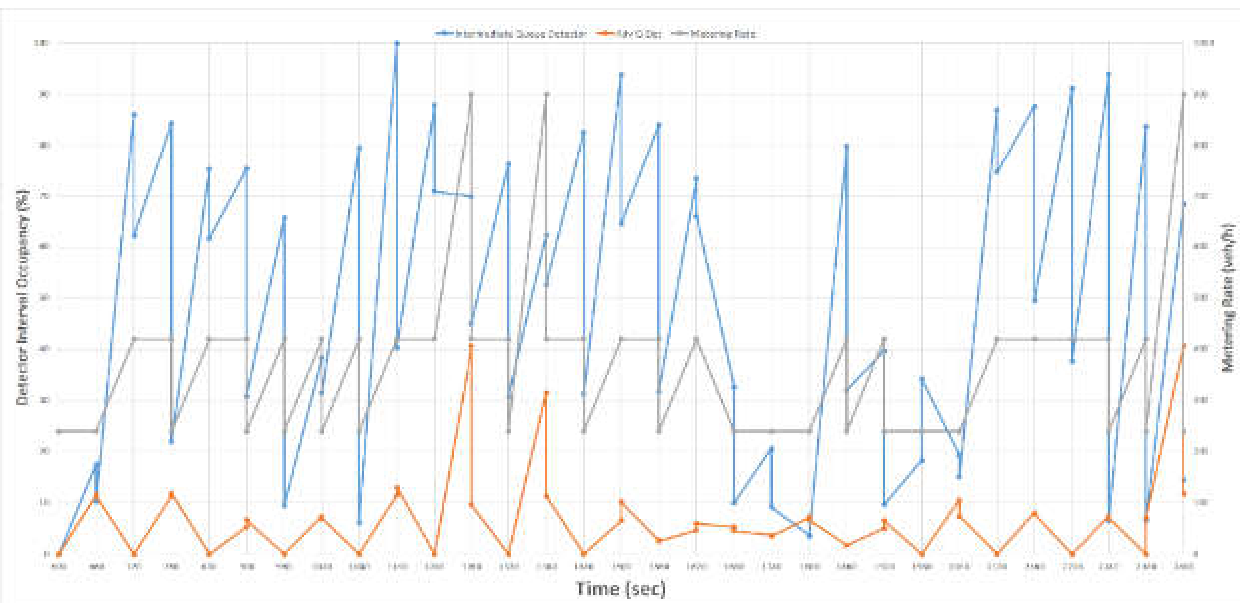


Figure 5-6. Metering rate/detector interval occupancy vs. time [Diamond] based on 1-min aggregation detector measurements)

Another potential issue for the Diamond configuration is that on-ramp queue backup for the left-turn movement not only may reduce the capacity of the left-turn movement (and cause complications upstream on the arterial), but may also block the conflicting through and off-ramp left-turn movements, creating potential safety issues in addition to operations issues.

<u>SPUI</u>

The SPUI has some similarities to the Diamond, such as the path routing for left-turn access to the on-ramps. Unlike the Diamond, the two ramp terminals are consolidated into one. Thus, there is only one signalized intersection in the interchange area, which also runs three timing stages. This may reduce arterial signal delay for the through movements and simplify the storage of left-turn vehicles.

A characteristic of the SPUI design is that the setback of the left-turn stop bar from on-ramp entrance point is typically larger than in the case of the Diamond design. With ramp metering, this larger setback is somewhat undesirable. When the on-ramp queue is at or near the on-ramp/arterial junction, it may be more difficult for drivers at the front of the left-turn queue to realize that there is no queue storage available and end up temporarily trapped in the intersection area. Just as for the Diamond configuration, this could result in a blocking situation for the conflicting through and off-ramp left-turn movements. Additional logic can be built into the arterial signal timing plan that holds the left-turn movement if the advance queue detector is occupied, but this adds more complexity to the system. This could also lead to some drivers traveling through the intersection, doing a U-turn and then making a right-turn movement onto the on-ramp.

<u>DDI</u>

The DDI's need for only two timing stages for signalized intersection operation generally provides greater vehicle throughput relative to timing plans with three or more timing stages. This timing plan efficiency is offset somewhat, however, by two issues. Because of the long distance that must be covered by vehicles through the intersection on the crossover movement, the all-red time must be much longer than for a traditional intersection. The all-red time translates directly into lost time. Two-way signal coordination is also not possible with the DDI since the two major arterial streams cross one another. As long as the peak direction is coordinated, however, lack of coordination for the off-peak direction is not likely to have a significant adverse impact.

With respect to how the interchange design and signal phasing/timing plan affects the on-ramps and metering operations, it has some similarity to cloverleaf designs. The left-turn traffic volume is not served through a dedicated left-turn timing stage at the downstream intersection of the two intersections. The left-turn traffic is instead served in the same timing stage as the through traffic at the upstream intersection.

Combined with a relatively short cycle length, the arrival pattern of the left-turn traffic to the on-ramp is relatively consistent. This can simplify the ramp-metering operations. For interchange configurations that lead to more of a surging pattern with the left-turn arrivals, such as the Diamond, the setting of queue detection parameters to avoid oscillation of ramp metering rates is more challenging. Figure 5-7 shows less oscillation of the metering rates, compared to Figure 5-5.

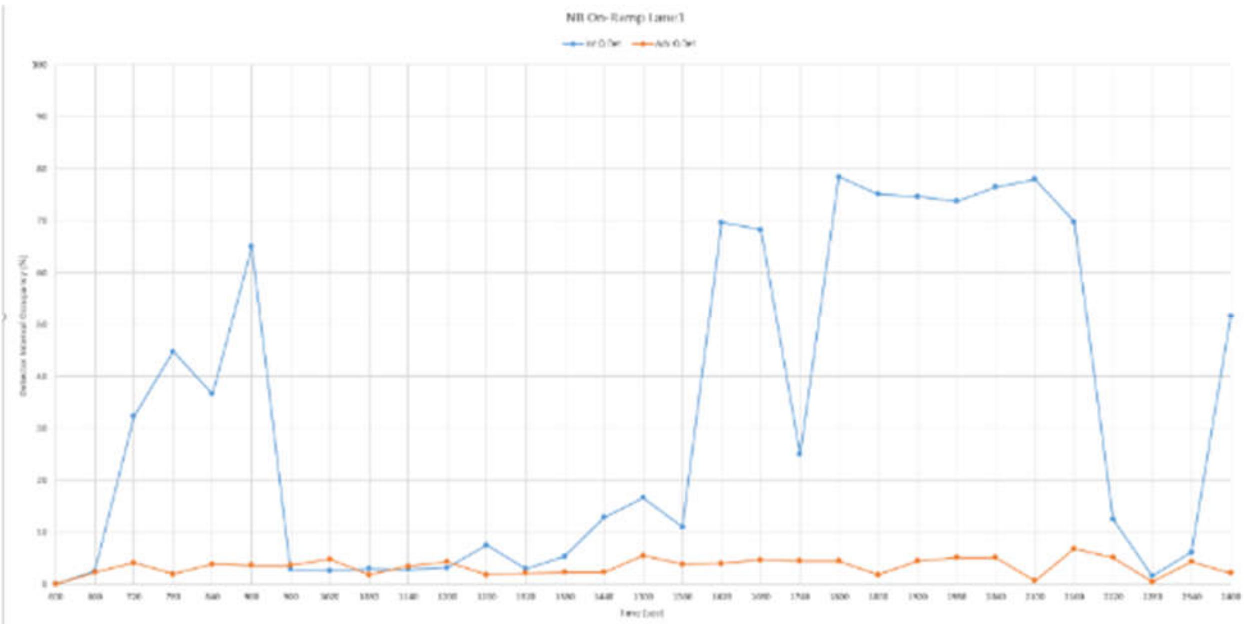


Figure 5-7. Detector interval occupancy (%) vs. time (seconds) [DDI] (time steps are aggregated to one minute)

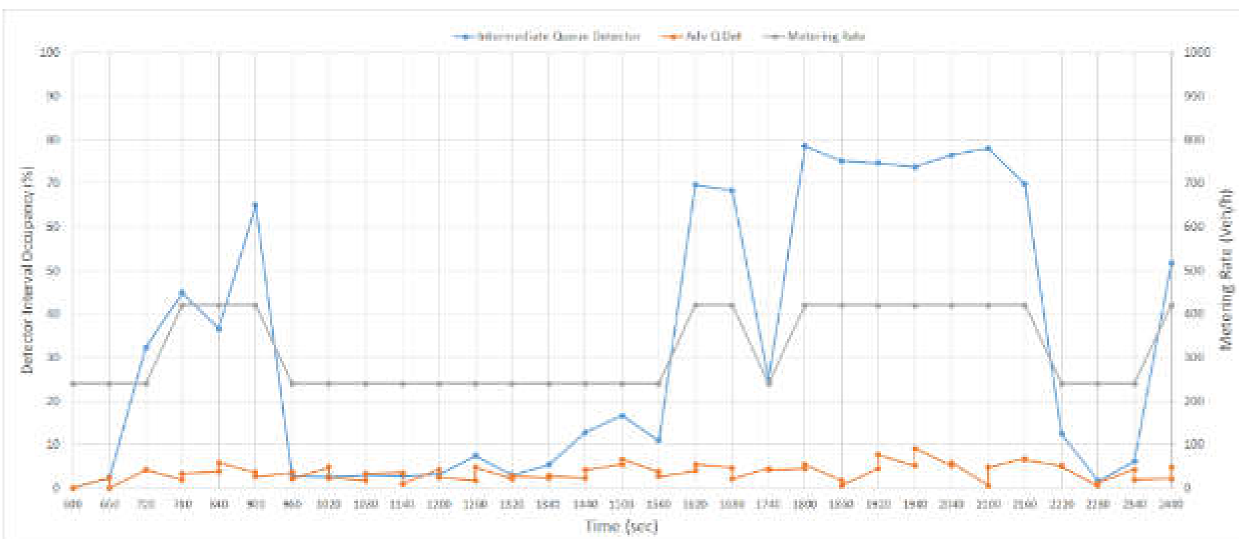


Figure 5-8. Metering rate/detector interval occupancy vs. time for DDI.

The DDI configuration under this traffic characteristics scenario had regular queue backup to the intermediate detector, but infrequent backup to the advance queue detector.

The crossover design feature of the DDI and the resulting left-turn positioning, which results in the left-turn traffic feeding the on-ramp from the near side (same as for right turn) of the interchange, creates some advantages in the ramp metering situation. Firstly, on-ramp queue backup of the left turns onto the arterial will not create a conflict with the through and off-ramp left-turn movements (for the same end of the interchange). Excessive queue backup could create conflicts with the opposing through and off-ramp left turn movements on the upstream end of the interchange, but these situations should be rare if the queue override detectors and logic are setup properly.

Ramp entrance design

Another issue to consider, for ramp configurations without loop ramps, is the merging of the right-turning and left-turning traffic segments of roadway into the combined queue storage portion of the on-ramp. This is illustrated in Figure 5-9 through Figure 5-11.

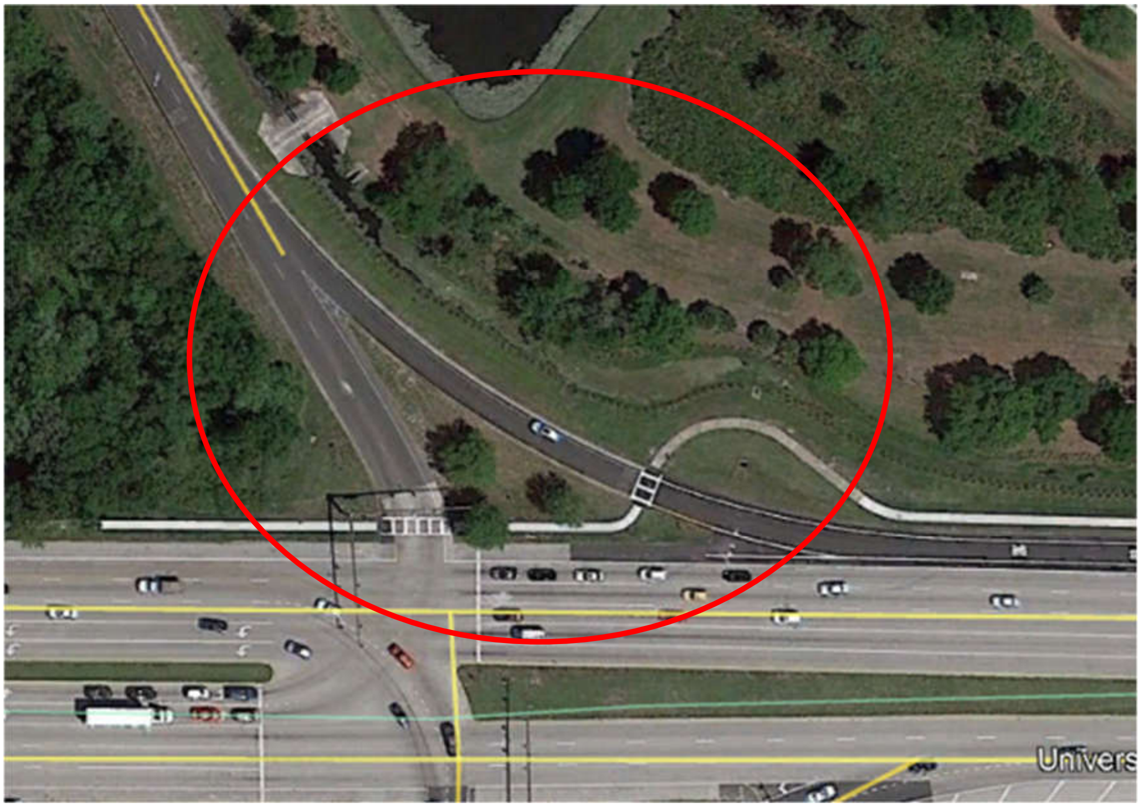


Figure 5-9. Left-turning and right-turning on-ramp segments for Diamond.
Source: Google Earth

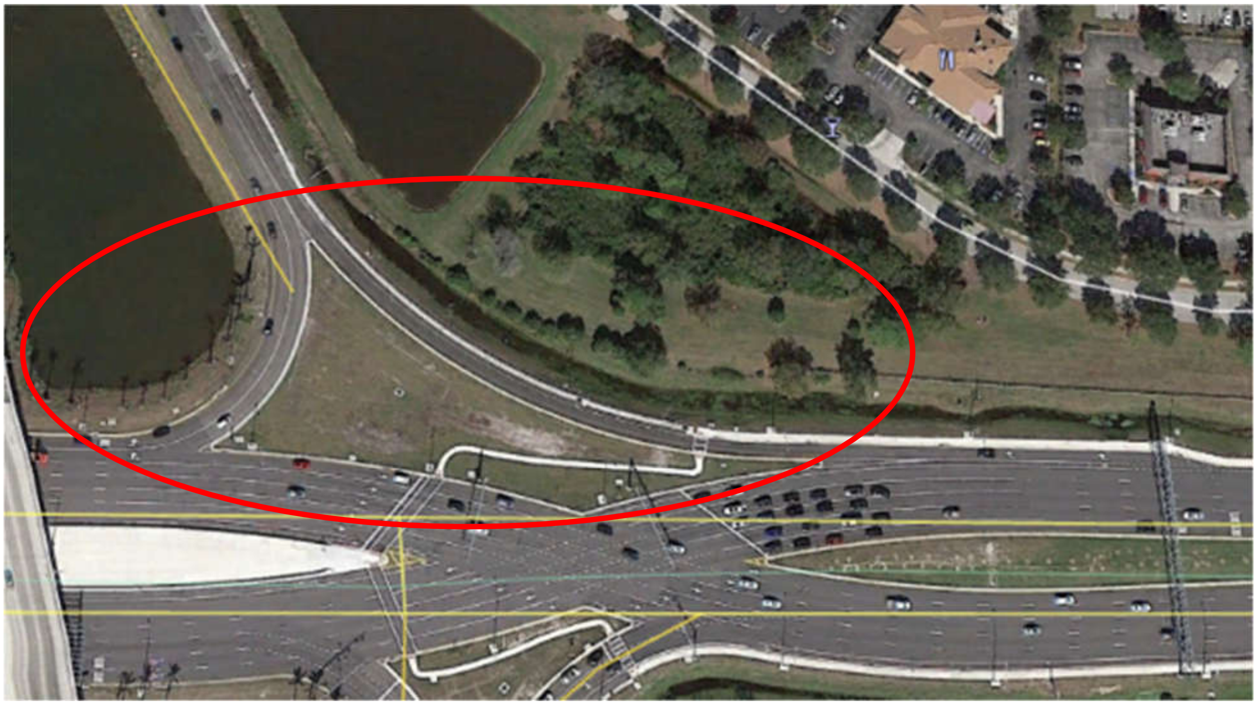


Figure 5-10. Left-turning and right-turning on-ramp segments for DDI.
Source: Google Earth

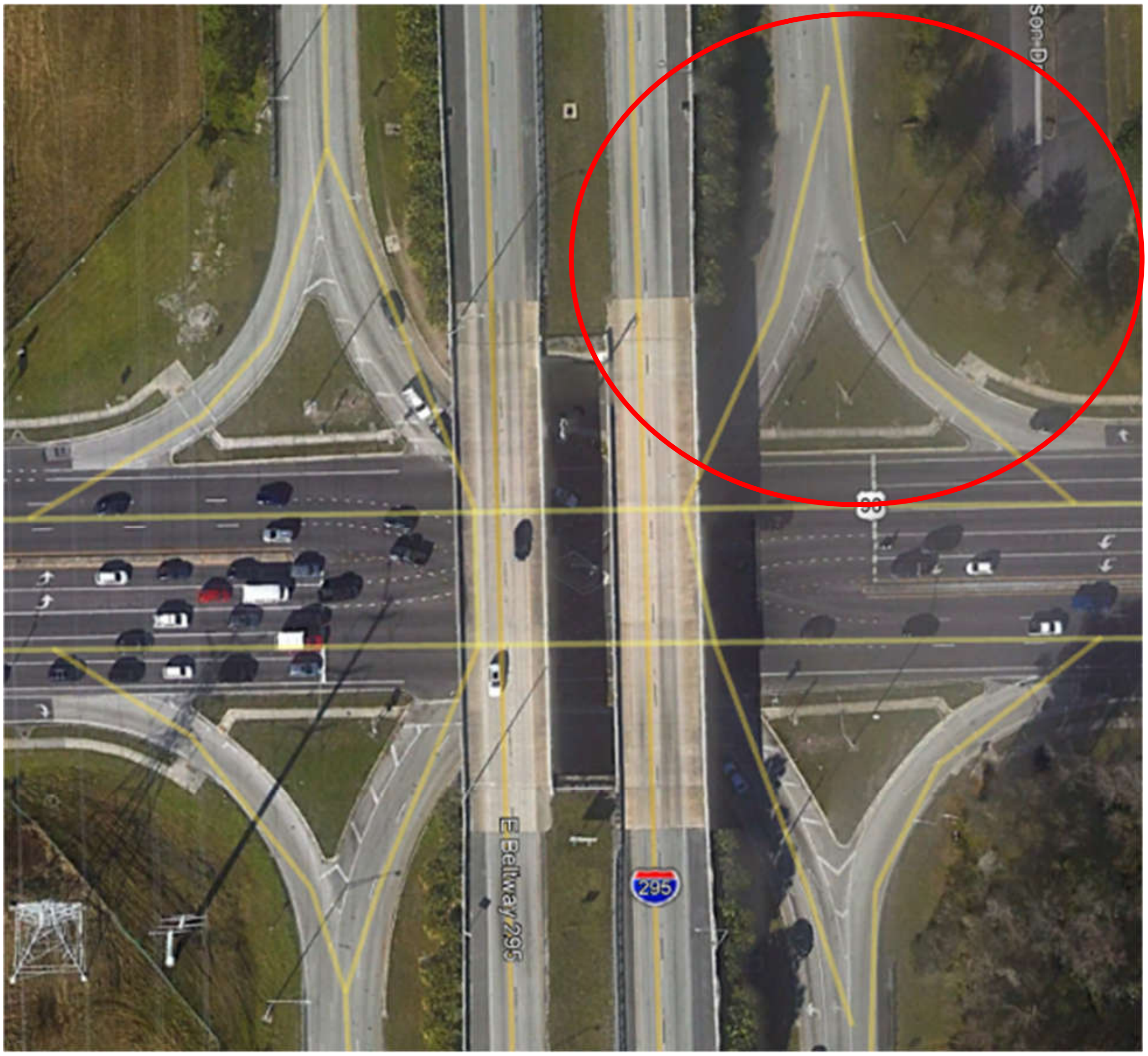


Figure 5-11. Left-turning and right-turning on-ramp segments for SPUI.
Source: Google Earth

Only the portion of on-ramp downstream of the merge point between the left-turn and right-turn segments is shared queue storage. This is analogous to the independent queue storage issue for the on-ramp lanes for loop-ramp designs, but of course on a much smaller scale. Additionally, the further downstream this merge point is from the arterial roadway, the more likely it is that a queue backup will reach this point and possibly complicate the merging process for the right-turn vehicles (since this movement is typically the yielding movement). If the right turn movement can be tied in 'tighter' to the left turn lane(s) and some length of lane extended past the merge point, this should reduce the merging friction and provide more storage for left turn vehicles as well.

General Interchange Configuration Recommendations

Of the interchange forms examined in this project, the DDI provides the best compromise between right-of-way footprint and ability to accommodate a wide variety of traffic characteristics with comparable or better performance measure results than the other forms. If extended arterial progression, particularly two-way, is a major issue, then the SPUI may be a preferred alternative. The ParClo design is also a reasonable alternative for ramp metering implementations, as long as the loop ramp ties into the direct ramp before merging with the freeway mainline (to create additional queue storage). Because of the independent lanes for left- and right-turning traffic, the performance of the ParClo varies more than the DDI with varying traffic conditions. Adding ramp metering to a FullClo without C-D design is not recommended—due to more limited queue storage and reduced acceleration distance for the loop ramps. At a minimum, construction of a C-D lane is recommended. However, strong consideration should be given to converting the FullClo to a ParClo design (converting the exit loop ramps to direct ramps). Although not considered explicitly in the experimental design for this project, incorporating bypass lanes, such as for high-occupancy vehicles, is generally simpler with the non-loop ramp designs. With or without ramp metering, loop ramps are generally more unfriendly to commercial trucks.

Queue Storage Assessment

For a given interchange configuration, one of the most critical issues is to avoid or minimize on-ramp queue spillback onto the adjoining arterial. The percentage of time the ramp meter spends operating in advance queue override mode is a key indicator of how likely such a situation is to occur. To assist with this assessment, a macroscopic queuing analysis tool was developed. This tool is open source and is available at https://github.com/swash17/RampMeterQueueing. This tool considers various traffic, control, and on-ramp roadway characteristics for a given ramp roadway-arterial intersection. It will provide estimates of queue length at every time step and the percentage of time that the advance queue override was activated. While a macroscopic queuing analysis cannot provide the same level of detail of on-ramp operations as a microscopic simulation tool, it can serve reasonably well as a ‘first-cut’ planning/preliminary engineering assessment. The use of this tool can help identify input conditions that may lead to unacceptable operational conditions. An overview of the methodology employed in the macroscopic queuing analysis tool, an example input file, and instructions on how to use the tool are in the GitHub repository.

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APPENDIX A: ROUNDTABLE MEETING SUMMARY

Meeting Date:
14th August 2018

Meeting Location:
FDOT D6, Room # 6204-8 (Procurement Room)

Time Convened:
10:00 AM – 2:15 PM (with 1-hour lunch break)

Attendees:
Scott S. Washburn, University of Florida
Shirin Noei, University of Florida
Jorge Barrios, Senior Engineer, Kittelson & Associates Inc.
Donald Avery, Senior Manager, AECOM/FDOT D6
Rodney Carrero-Vila, FMS/AMS Specialist, FDOT D6
Rossi Gaudio, ITS Engineer, Eland Engineering
Jose Grullon, Traffic Analyst, AECOM/FDOT D6
Alana Majdalawi, Environmental Specialist, FDOT D4
David Needham, 95 Express Program Manager, AECOM/FDOT D6
Tejas Sale, Express Lanes Program Manager, AECOM/FDOT D6
Jillian Scholler, Traffic Engineer III, AECOM/FDOT D4
Daniel Smith, ITS Operations Manager, FDOT D4

Coordination Contact:
Gisselle Vega, Office Manager at FDOT D6

The second task of this project was to convene a roundtable discussion meeting with FDOT D4 and D6 personnel who have been involved in the planning, design, and/or operation of ramp metering systems. The intent of this meeting was to help develop a clear understanding of the agency goals and priorities, lessons learned, and areas where better guidance was needed. The outcomes from this meeting will help us more specifically determine the direction of the remaining tasks in this project.

FDOT D4 is expected to deploy 35 to 110 ramp meters along I-95 Broward County and Palm Beach County in 2019. The ramp meters are going to be fully automated, activated per time of day, operating under local traffic-responsive mode and a fuzzy-logic algorithm. FDOT D6 has deployed 22 ramp meters along I-95 Miami-Dade County, including 10 ramp meters on NB (in operation since 2009) and 12 ramp meters on SB (in operation since 2010), and is expected to deploy 19 ramp meters along SR-826 (from SR-836 to NW 154th St.) in 2019. The ramp meters on NB (from NW 62nd St. to NW 167th St.) are activated during the PM peak, and the ramp meters on SB (from NE 183rd St. to NE 203rd St.) are activated during the AM peak. The metered interchanges along I-95 Miami-Dade County (mostly tight-diamond interchanges) operate under coordinated traffic-responsive mode and fuzzy-logic algorithm.

The meeting started with a presentation on:

- project background,
- project schedule,
- project objectives,
- ramp-metering overview,
- ramp metering in Florida,
- states with specific guidance,
- ramp-metering design standards,
- summary of research papers/projects, and
- sampling of common types of interchange configurations with ramp metering (tight diamond, compressed diamond, single point urban interchange (SPUI), partial cloverleaf, cloverleaf, and diverging diamond interchange (DDI)).

The meeting continued with group discussion and some specific questions on ramp-metering design/operational challenges, best/undesirable practices for accommodating ramp metering at interchanges, and more. The questions and responses are as follows. It should be noted that the following text is a summary, not a transcript. Additionally, some reorganization of the responses was done.

Do you have any recommendations for improving current I-95 configurations?

- Have more detection (i.e., intermediate detectors) on the on-ramps.
- Set limits on queue storage.

What challenges do you currently face with ramp-metering operations as it relates to interchange design?

- Consideration of ramp metering only done after interchange designed.
- Ramp-metering warrant studies showed that almost half of the on-ramps in D4 did not meet the ramp-metering warrants on acceleration distance (i.e., the acceleration distance should be longer than the safe merging distance) and ramp storage (the ramp storage should be greater than the queue length), but they decided to implement ramp metering on all the ramps anyway. The reason for that is because the required minimum acceleration distance and the required storage distance are dynamic variables based on the real-time traffic conditions (i.e., the minimum required acceleration distance is based on the freeway-mainline prevailing speed, and the required storage distance is based on the peak-hour ramp demand). Ramp metering is warranted when the freeway mainline prevailing speed drops below a pre-defined threshold, or when the ramp peak-hour demand exceeds a pre-defined threshold.
- Desirable to have automated control over system on/off and operation. Also, a need for a dynamic zone-based metering system that can group the ramps based on the recurring and non-recurring congestion.
- If two ramp signal heads are installed for one lane (on either side of lane), drivers will likely still form two queues. D6 is using overhead gantry for two-lane on-ramps.
- Could not find accurate information about acceleration distance for fully-loaded tractor trailer.

How much of an issue is on-ramp queue spillback? Are you implementing any specific mitigation measures (e.g., signal timing, geometric modifications)?

- Human observer in TMC looking for queue spillbacks and can manually implement max metering rate.

Are you currently doing anything specific about coordinating arterial signals with ramp metering?

- Some discussion with Broward county for cooperation with upstream signal timing to restrict flow arriving at on-ramp.
- Considering no free-right turns onto on-ramp.

If designing a new interchange, with ramp metering, and without geographical/right-of-way constraints, what considerations would guide your design?

- SPUI: potential issue with left-turn drivers seeing back of queue for on-ramp, and potentially getting stuck in intersection (can this be addressed with flashing beacon, or holding left-turn signal, what kind of driver frustration will this cause?).
- Currently, there is no formal agreement in place between FDOT and counties to limit the queue backup from the on-ramps.
- Diamond interchanges are still good overall.
- C/D roads are good, moving friction off the mainline.

How well do you think ramp metering would work at a DDI?

- Probably a little better than standard diamond because of storage of left turns in outer lanes.

What do you want to see come out of this project? /How can the results of the project help you in your job? /What is needed in guidance?

- Need comprehensive ramp meter warrants.
- D4 and D6 should also consider ramp meters as part of managed lanes network.
- Guidance on release rate of meters.
- Acceleration distance (absolutely should not be violated), queue storage (maybe could compromise), and sight distance (for on-ramps).
- More explicit consideration of queue jumping for slip-ramp configurations (will become a problem for Palmetto project). If not designed properly, drivers try to queue jump and irritate those waiting properly in queue.
- For driver expectation, if one ramp is metered, others should also be metered.
- When/where is upstream warning sign/flasher (meter on) warranted? Also warning for queue backup to upstream end of ramp, so upstream drivers (e.g., left-turners at diamond interchange) do not get stuck blocking intersection because they cannot enter ramp.
- Need to review assumption of design speed for acceleration distance from ramp meter.
- Need to revisit truck acceleration capabilities.
- Would like to see bypass lanes for trucks and buses (Palmetto/25th), especially for upgrade on-ramps.
- Consider a sign that says: “trucks do not stop for ramp meter”, and maybe shut off ramp meter, if truck in queue is detected.
- When acceleration distance is insufficient for mainline free-flow speed, do not turn on meter, unless freeway speed drops.
- Add material to PPM to help designers with accommodating ramp metering.
- Include in PPM details about how to incorporate ramp metering for a given design (will a given design work or not work with ramp metering).
- Guidance on loop ramp storage vs. delay (i.e., does potential increased delay outweigh increased storage capability? what is the driver perception?).
- More CCTV coverage specified during design, particularly for loop ramps (want to see queue, ramp signal, and signalized intersection).
- If there is any chance of including ramp metering someday at the interchange, it must be considered from the beginning (e.g., part of PD&E).
- Guidance on how traffic operations with metering could impact surrounding facility system.
- More consideration of operations for a given design.
- A decision flow chart for ramp metering (e.g., are you designing a new ramp? is there any ITS device?).
- If ramp is closed, but needed for emergency response, how to handle emergency vehicles.

Things to avoid

- Inclines, tight turns, poor sight distance (crest curves).
- Inconsistency in geometric design of ramps.

Potential future research ideas

- Other ways to use ramp metering (e.g., wrong-way driving, full freeway closures).
- More research on driver perceptions of interchange design.
- Alternative intersection designs that could potentially be used for interchange design (e.g., jug handle).

Other discussions

- Palmetto Expressway/NW 67th Ave. is a sample metered SPUI (in the design stage).
- Dolphin Expressway/NW 27th Ave. is one of the two operating DDIs in Florida.
- Coppins Rd./Sample Rd. (North Broward) and Palmetto Expressway/US 27th (North Okeechobee) are sample metered loop interchanges (both in the design stage) with two adjacent ramp meters (one on diagonal on-ramp and one on loop on-ramp).
- Not sure if freeway-to-freeway metering is being considered in Florida.

APPENDIX B: SIMULATION TOOL OVERVIEW

This project relied heavily on the use of traffic simulation. Moreover, since field data from interchange sites were not included in this project, which would assist with simulation model calibration/validation, it was necessary that a reliable simulation tool be used for this project. SwashSim (https://swashsim.miraheze.org/wiki/Main_Page) is the chosen simulation tool. More background on this tool and its development follows.

Dr. Washburn has 30 years of traffic simulation experience. One of his first microscopic traffic modeling projects was to implement the Seattle-area ramp metering algorithm into the FORTRAN code of INTRAS—the predecessor to FRESIM (which was later merged with NETSIM to form CORSIM) that ran in a mainframe environment. While Dr. Washburn has worked with a variety of simulation tools over the years, his primary expertise was with CORSIM. This included not just application to research and course projects, but also leading the development/implementation of new features in CORSIM, such as two-lane highway and toll plaza modeling, and performance measure calculations such as percent-time-spent-following, follower density, and acceleration noise.

Due to the outdated software architecture of CORSIM and the limitations this imposed for CORSIM to keep pace with the traffic modeling demands of the future, Dr. Washburn began, circa 2011, development on what was essentially the next generation CORSIM; that is, SwashSim. SwashSim initially "inherited" all of the vehicle movement models from CORSIM, as well as many other algorithms related to the simulation logic. However, none of the existing CORSIM code was utilized, as SwashSim was built from scratch, with a modern, state-of-the-art software architecture and object-oriented programming language (C# / .NET Framework). This new architecture supports a high level of fidelity with respect to temporal and spatial modeling resolution and the ability to incorporate any number of advanced modeling concepts. A sampling of the modeling features included in SwashSim are:

- A 0.1-second simulation time step (CORSIM uses 1.0 s),
- Explicit trajectory generation a vehicle's entire path through the network (CORSIM relied on the animation component to "fill in" some aspects of a vehicle path, such as movement through an intersection area,
- *Car-following model*: Modified Pitt (an extension of the Pitt model used in CORSIM[4]);
- *Lane-changing model* (mandatory, discretionary): largely based on CORSIM models[5];
- *Gap acceptance model*: based on the models used in CORSIM but correlated to driver type[6];
- *Two-lane highway passing behavior*: based on the models implemented in CORSIM[7];
- *Vehicle dynamics*: utilizes powertrain characteristics of vehicles (engine, transmission) and resistance forces (aerodynamic, rolling, grade) to determine maximum acceleration capability[8];

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[4] Details on these models can be found at https://swashsim.miraheze.org/wiki/Car_Following.
[5] Details on these models can be found at https://swashsim.miraheze.org/wiki/Lane_Changing.
[6] Details on these models can be found at https://swashsim.miraheze.org/wiki/Gap_Acceptance
[7] Details on these models can be found at https://swashsim.miraheze.org/wiki/Two-Lane_Highway_Passing.
[8] Details on these models can be found at https://swashsim.miraheze.org/wiki/Maximum_Acceleration.

- *Vehicle characteristics*: includes 14 different vehicles by default (such as a low performance Honda Accord, a higher-performance Chevy Impala, a Ford 150 pickup truck, a Chevy Blazer SUV, and a tractor + semi-trailer combination)[9], with the ability to include more;
- *Driver characteristics*: contains 10 driver types by default (type 1-the most conservative driver, type 10-the most aggressive driver); treating drivers as distinct objects (i.e., driver characteristics are not embedded with vehicle characteristics), consequently enabling the user to separately customize vehicle and driver characteristics[10];
- *Signal control modes*: pre-timed and actuated signal control. The signal controller architecture replicates the standard NEMA ring and barrier architecture[11];
- *Ramp metering*: able to run three dynamic ramp-metering algorithms: ALINEA, Fuzzy logic, and demand/capacity[12], in addition to pre-timed control; and (text was missing i.e. there was no text after and)
- *Route assignment method*: randomly assigning a turning movement for the next link based on user-specified turning percentages for the link. Using an origin-destination (O-D) demand matrix with user equilibrium traffic assignment is under development[13].

More information on these concepts and other aspects of SwashSim can be found in https://swashsim.miraheze.org/wiki/Main_Page.

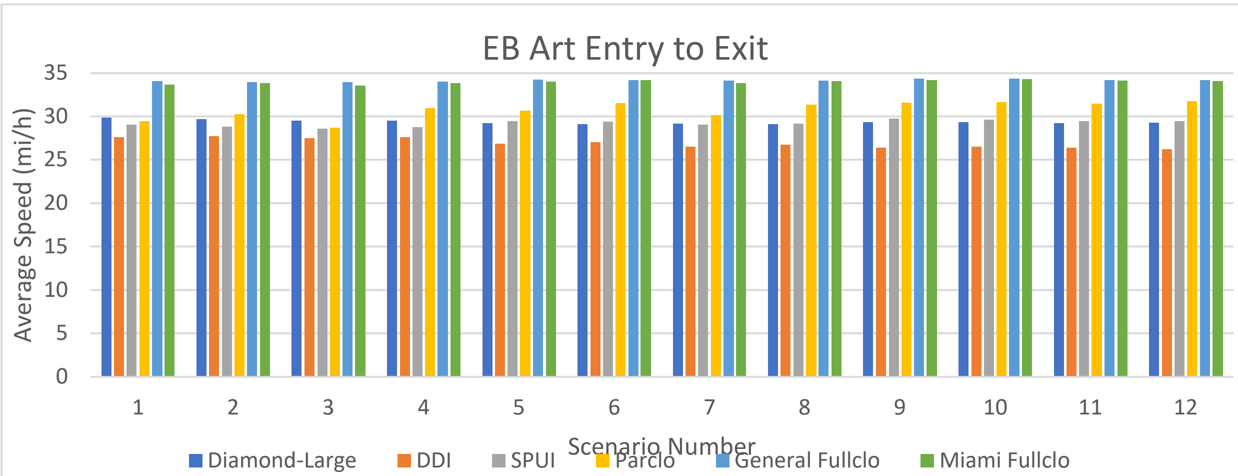
Dr. Washburn has used SwashSim in several research projects and many of his classes, including Introduction to Transportation Engineering, Traffic Engineering, Freeway Operations and Simulation, and Advanced Traffic Simulation.

[9] Full list of vehicles can be found at https://swashsim.miraheze.org/wiki/Vehicles.
[10] Driver details can be found at https://swashsim.miraheze.org/wiki/Drivers.
[11] Details on signal control can be found at https://swashsim.miraheze.org/wiki/Traffic_Signal_Operations.
[12] Details on these algorithms can be found at https://swashsim.miraheze.org/wiki/Ramp_Metering.
[13] Details on these models can be found at https://swashsim.miraheze.org/wiki/Route_Assignment.

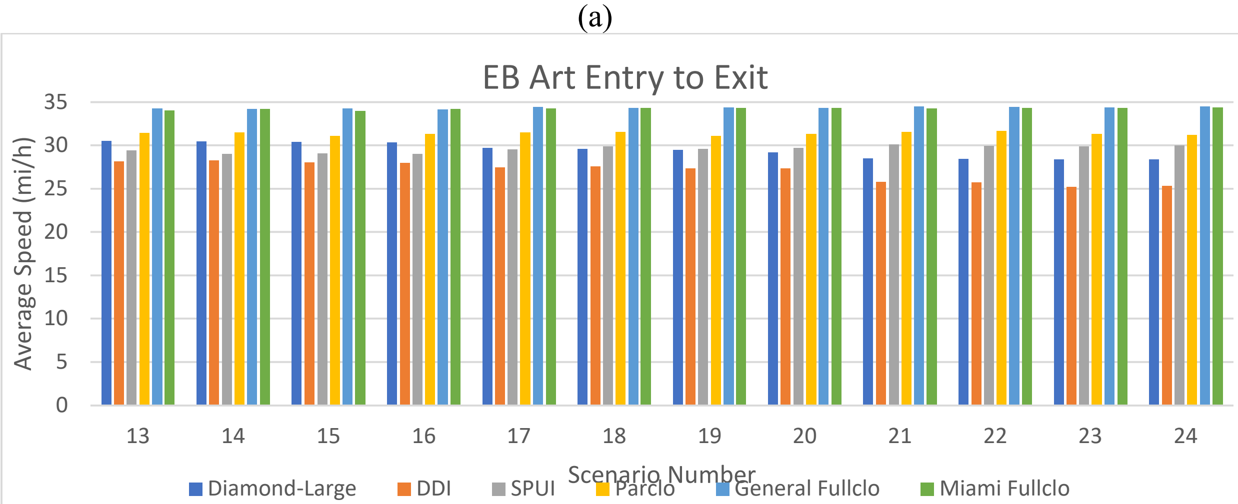
APPENDIX C: ADDITIONAL RESULTS OUTPUT DATA

Average Travel Speed

Figure C-1 through Figure C-6 present the average travel speed for a given travel path through the interchange, across all interchanges, by scenario. Scenarios 1-12 are the high-demand scenarios and scenarios 13-24 are the medium-demand scenarios.

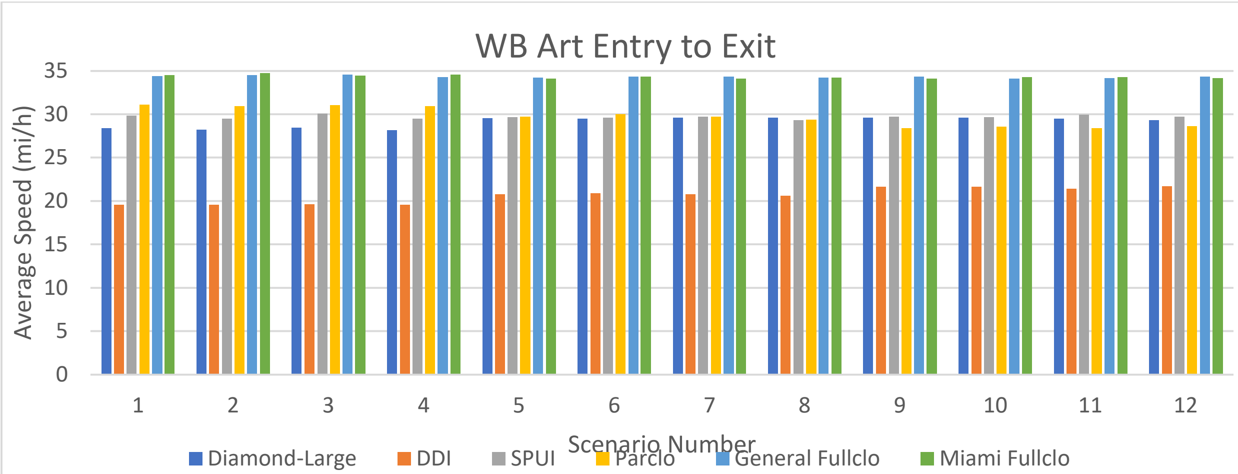


(a)

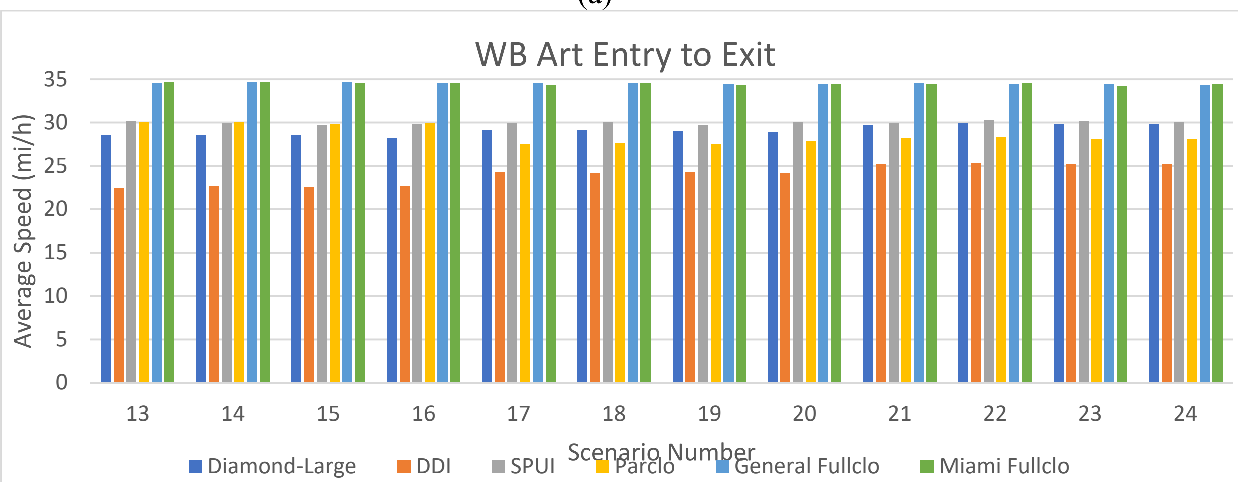


(b)

Figure C-1. Average travel speed for path EB entry to exit: (a) high- (b) medium-demand scenarios

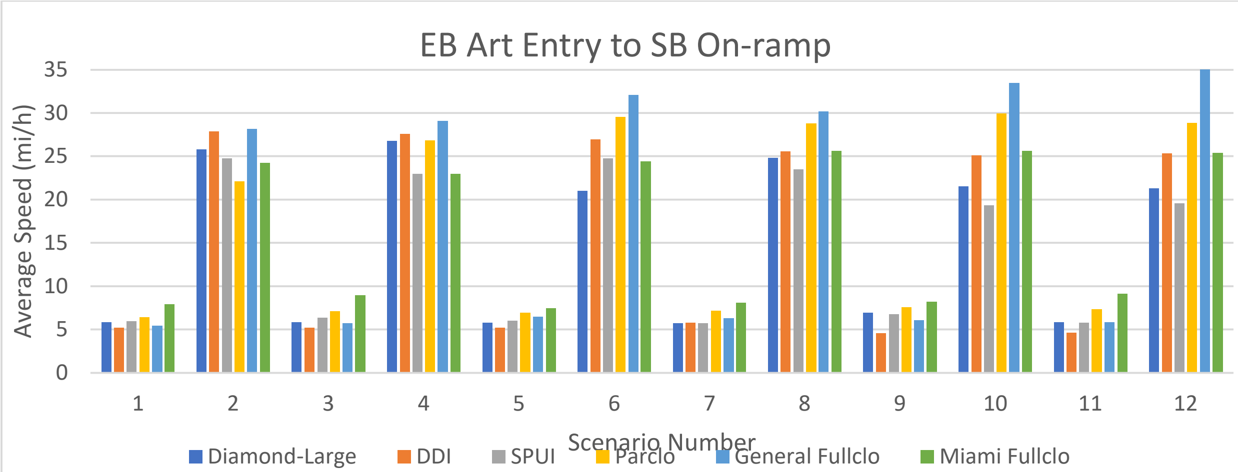


(a)

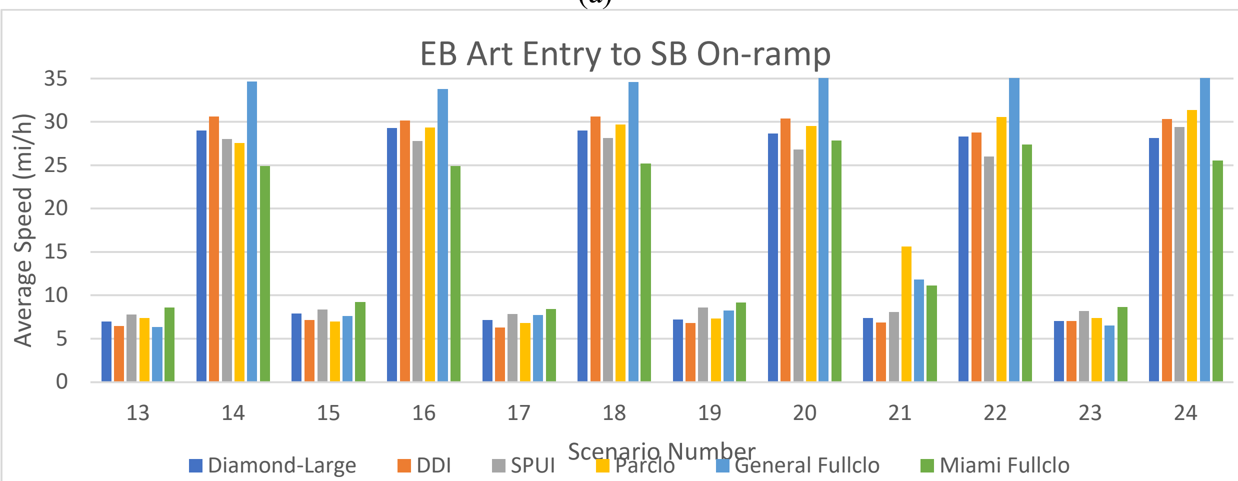


(b)

Figure C-2. Average travel speed for path WB entry to exit: (a) high- (b) medium-demand scenarios

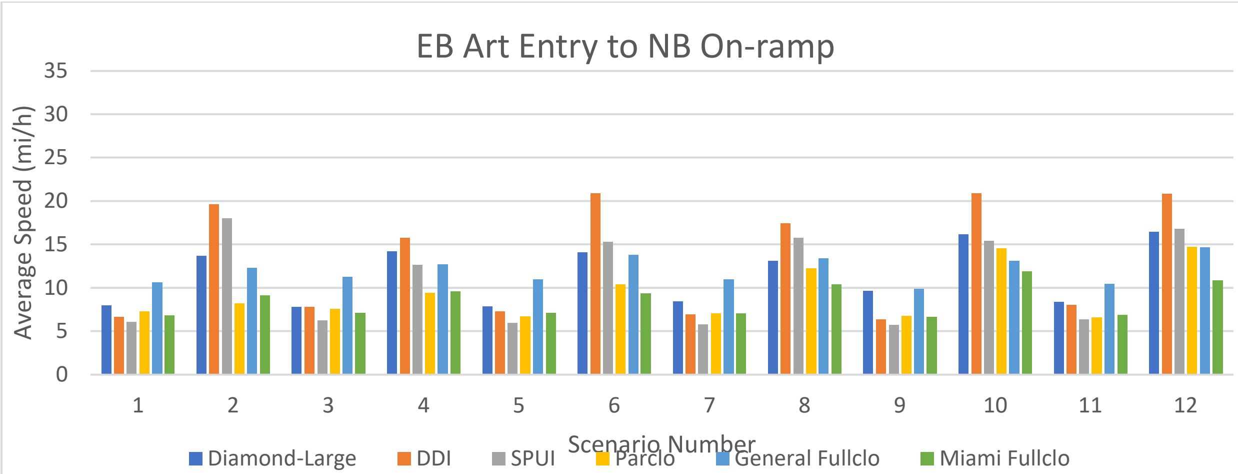


(a)

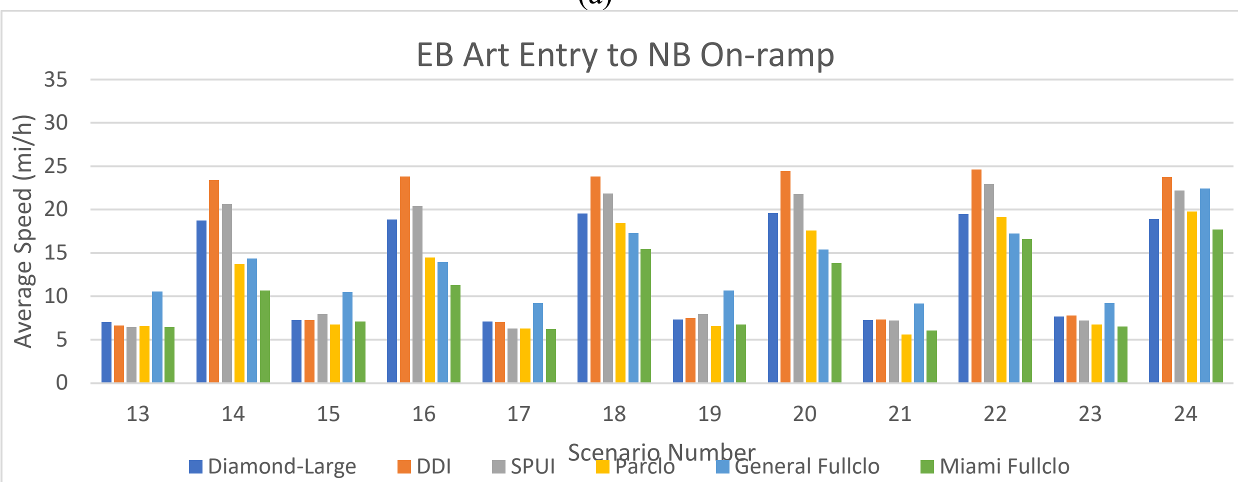


(b)

Figure C-3. Average travel speed for path EB entry to SB on-ramp: (a) high- (b) medium-demand scenarios

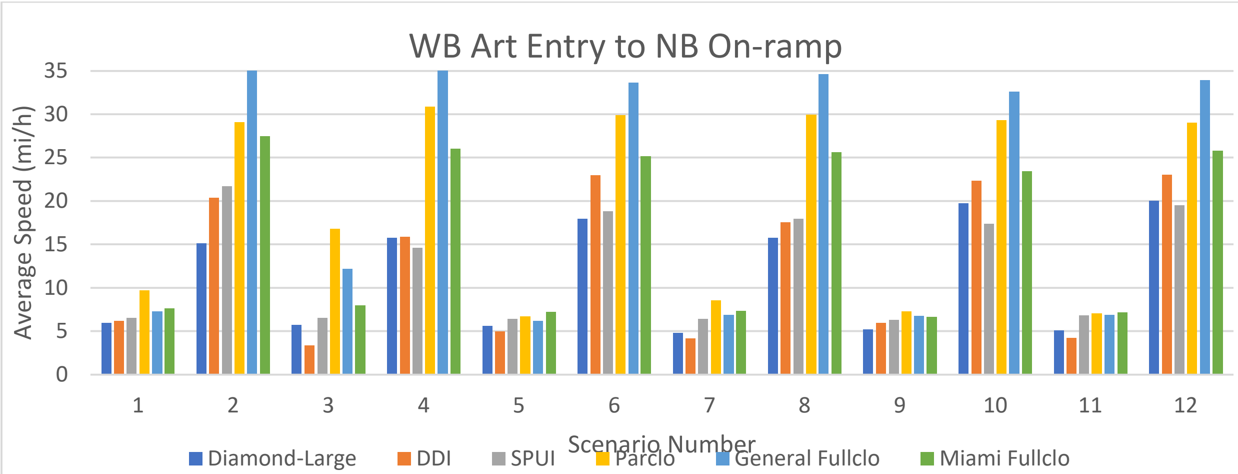


(a)

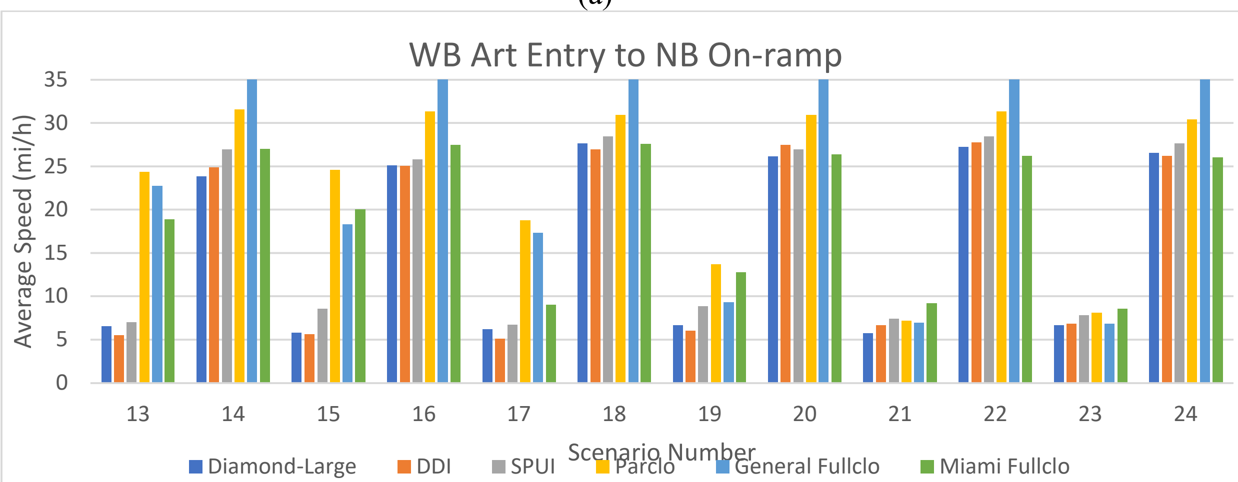


(b)

Figure C-4. Average travel speed for path EB entry to NB on-ramp: (a) high- (b) medium-demand scenarios

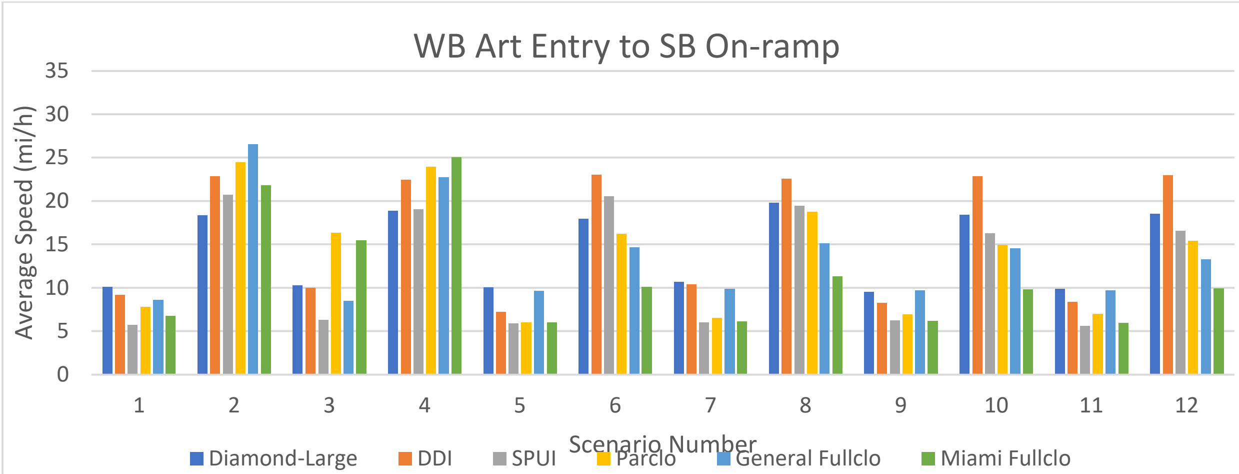


(a)

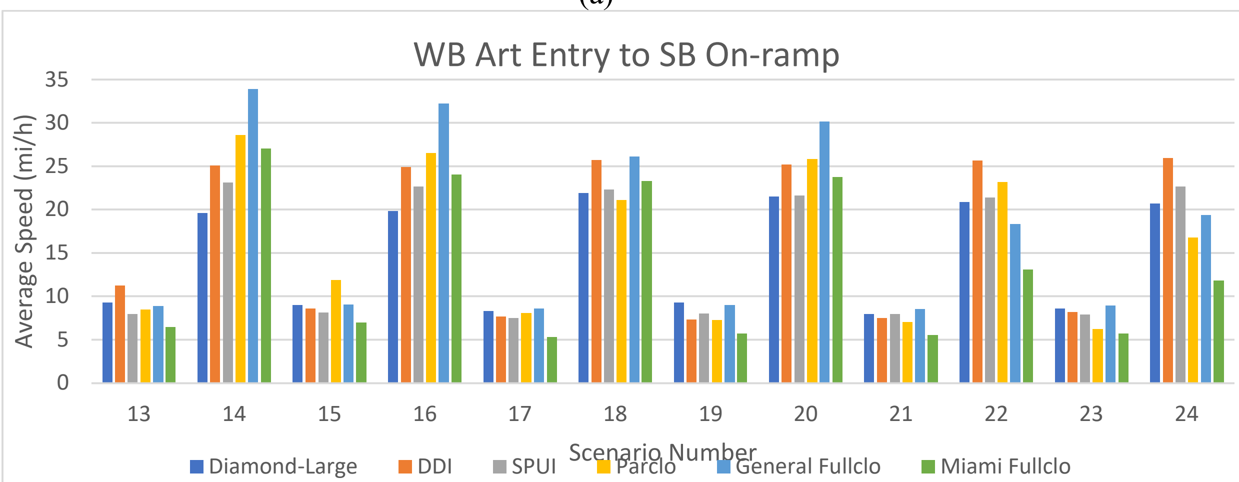


(b)

Figure C-5. Average travel speed for path WB entry to NB on-ramp: (a) high- (b) medium-demand scenarios



(a)

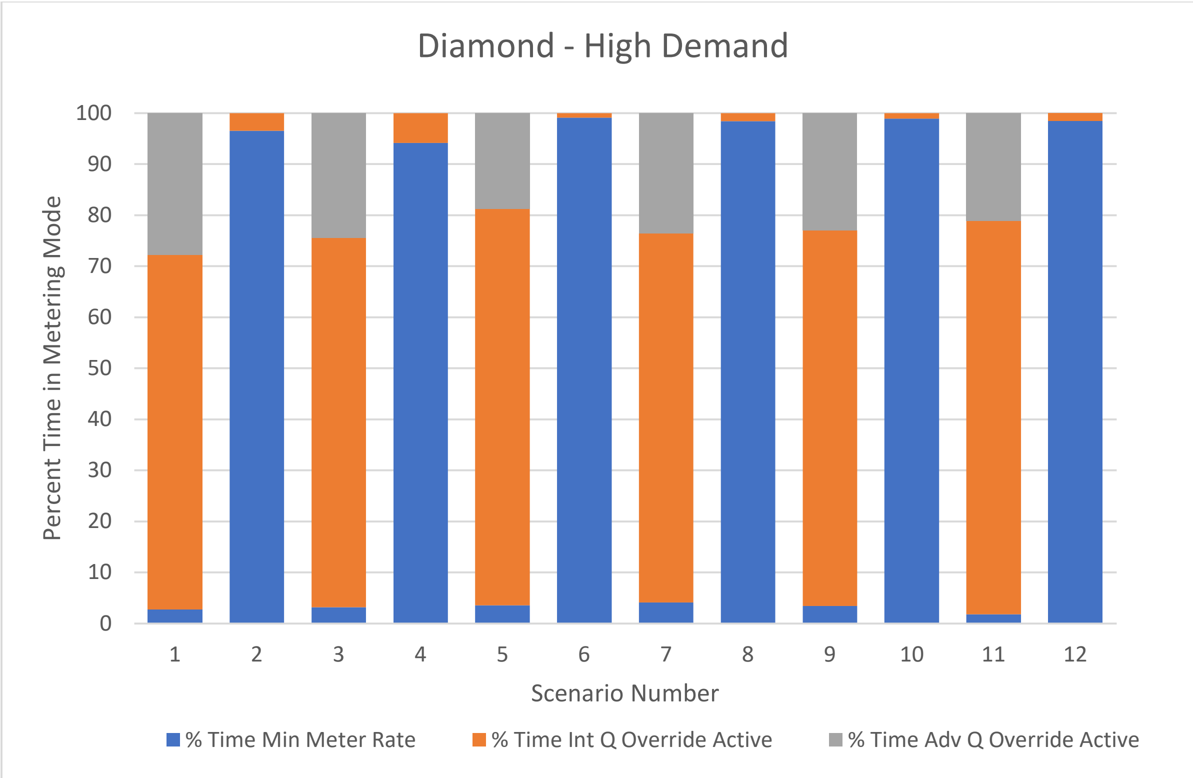


(b)

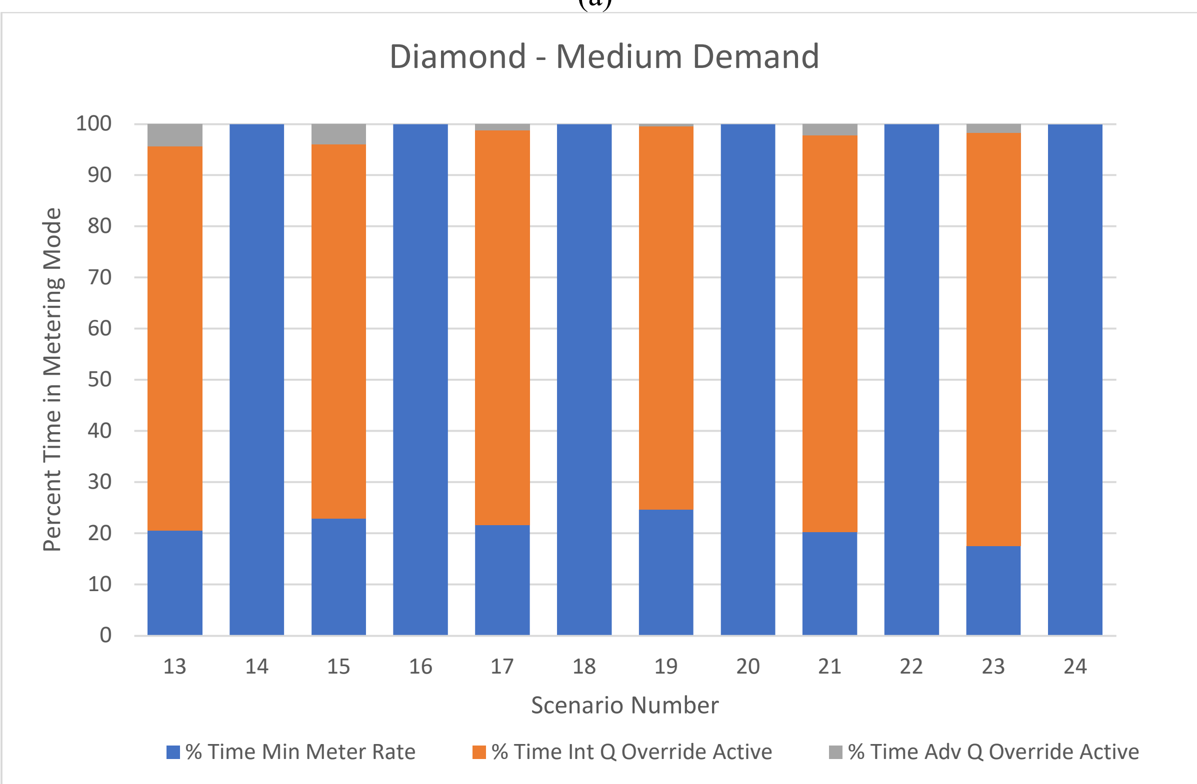
Figure C-6. Average travel speed for path WB entry to SB on-ramp: (a) high- (b) medium-demand scenarios

% Time in Metering Mode

The results presented in Figure C-7 through Figure C-12 are grouped by the demand level. Scenarios 1-12 are the high-demand scenarios and scenarios 13-24 are the medium-demand scenarios. These results correspond to an aggregation of both on-ramp metering controllers.



(a)



(b)

Figure C-7. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for Diamond: (a) high- (b) medium-demand scenarios

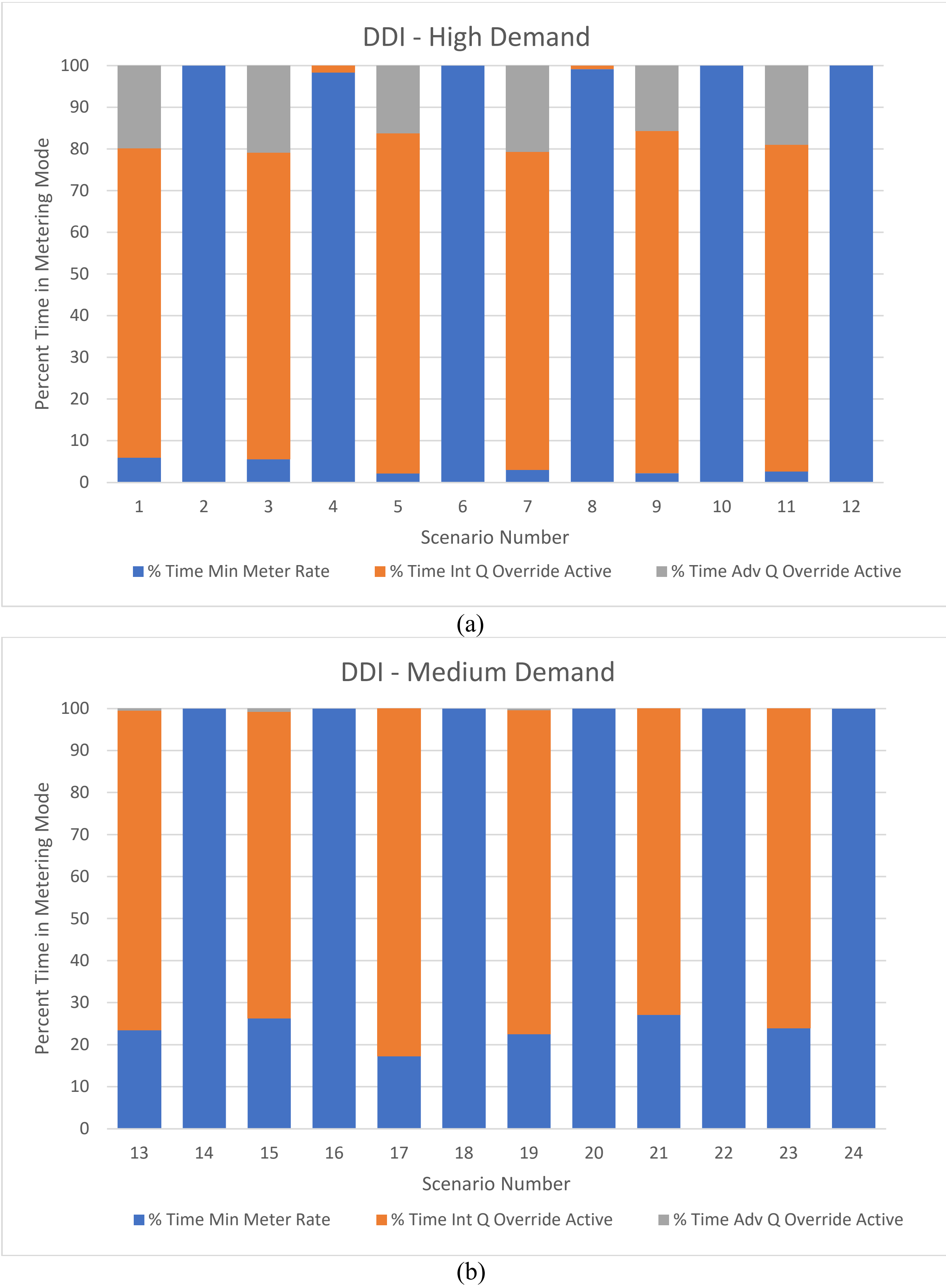
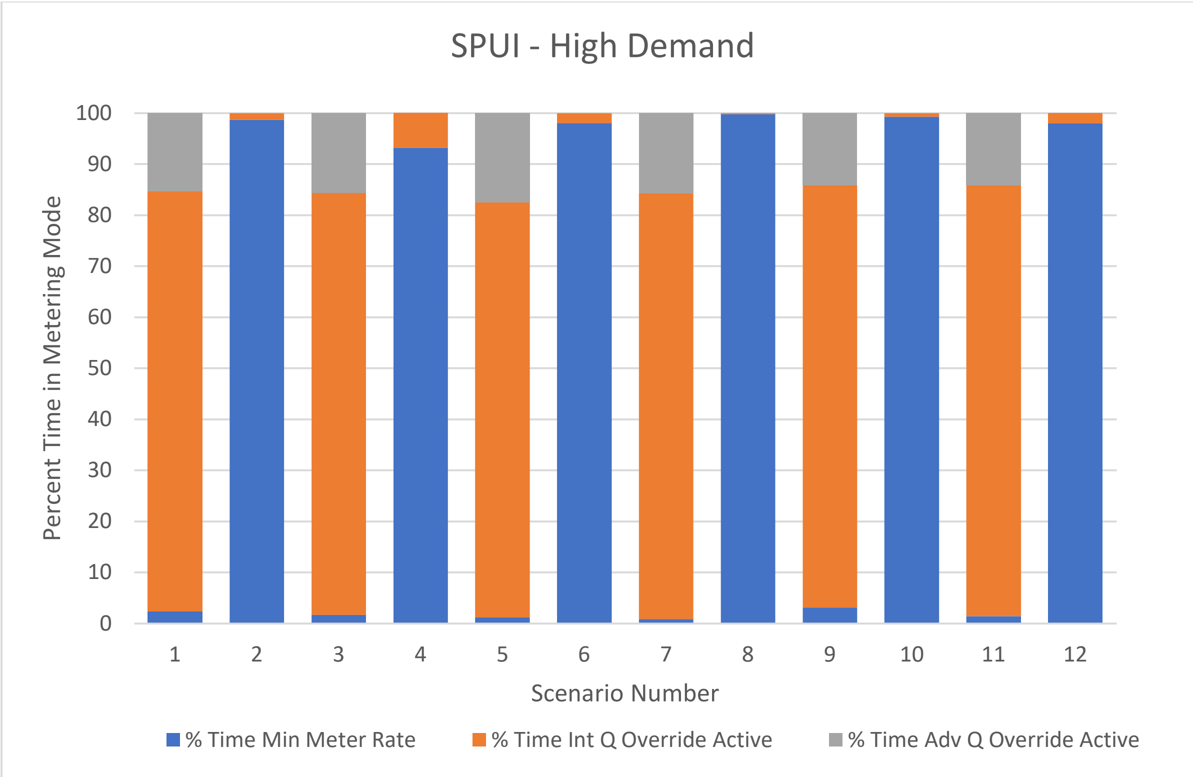
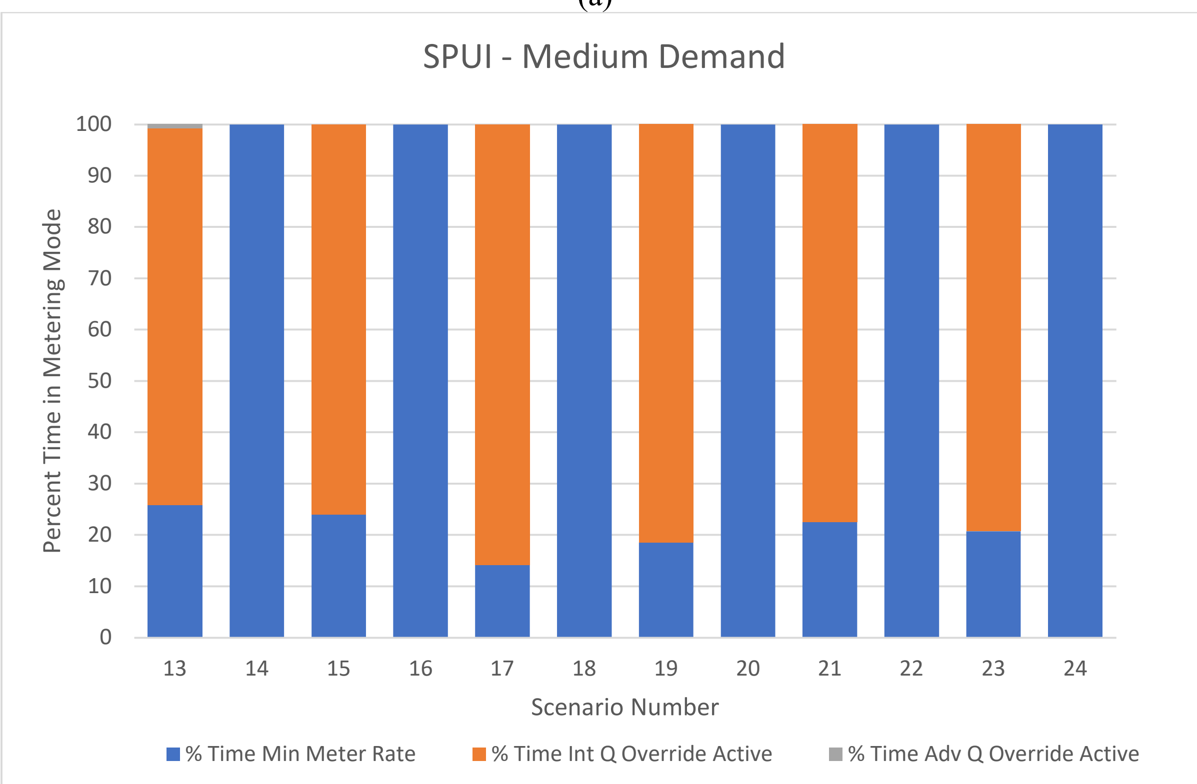


Figure C-8. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for DDI: (a) high- (b) medium-demand scenarios

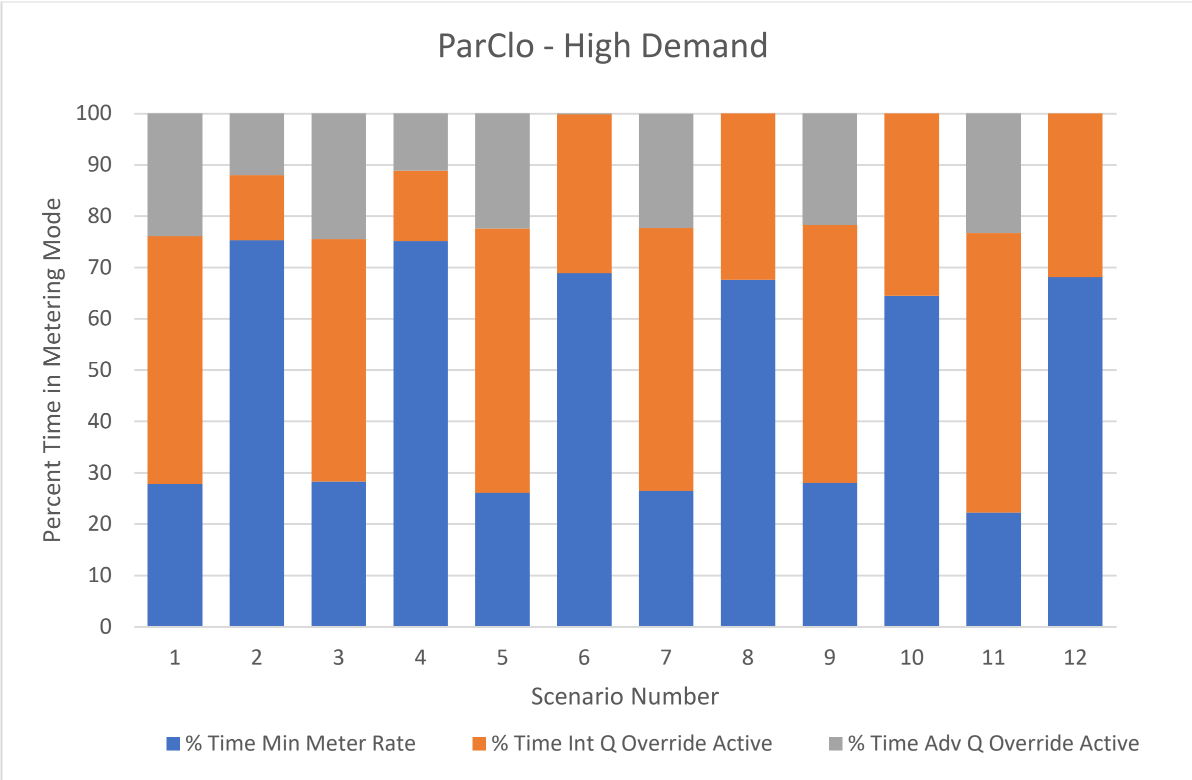


(a)

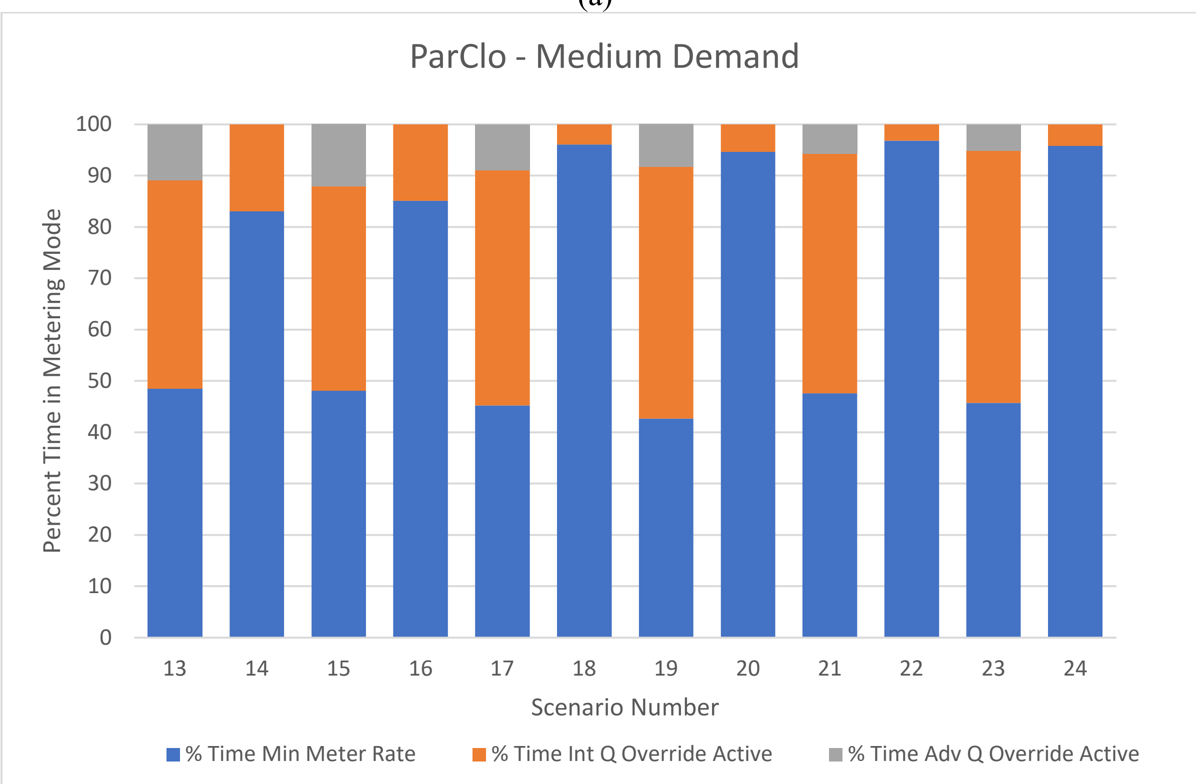


(b)

Figure C-9. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for SPUI: (a) high- (b) medium-demand scenarios

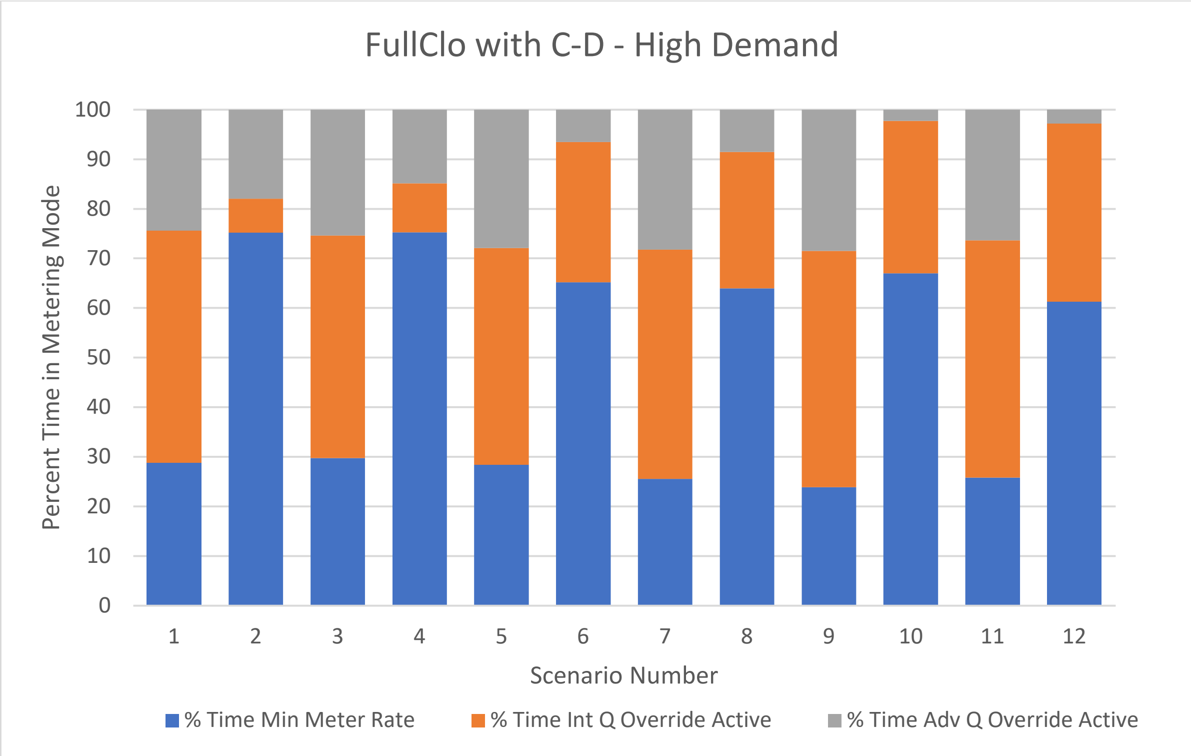


(a)

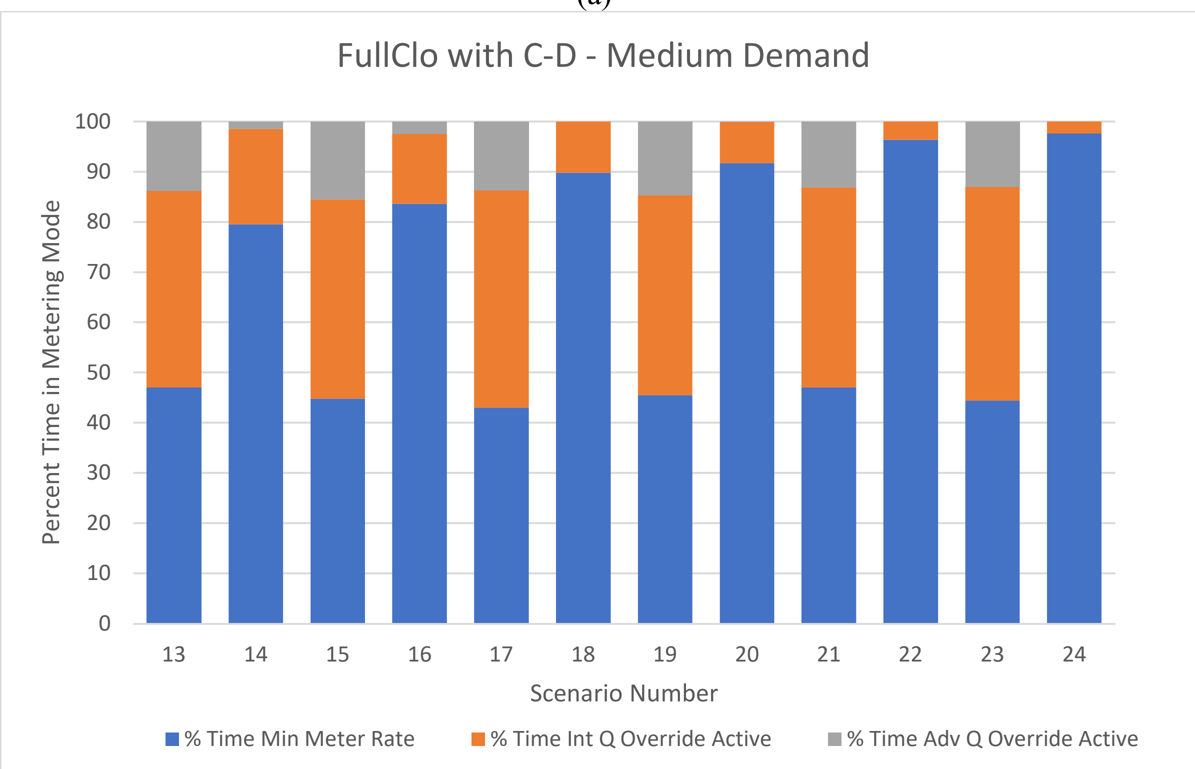


(b)

Figure C-10. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for ParClo: (a) high- (b) medium-demand scenarios

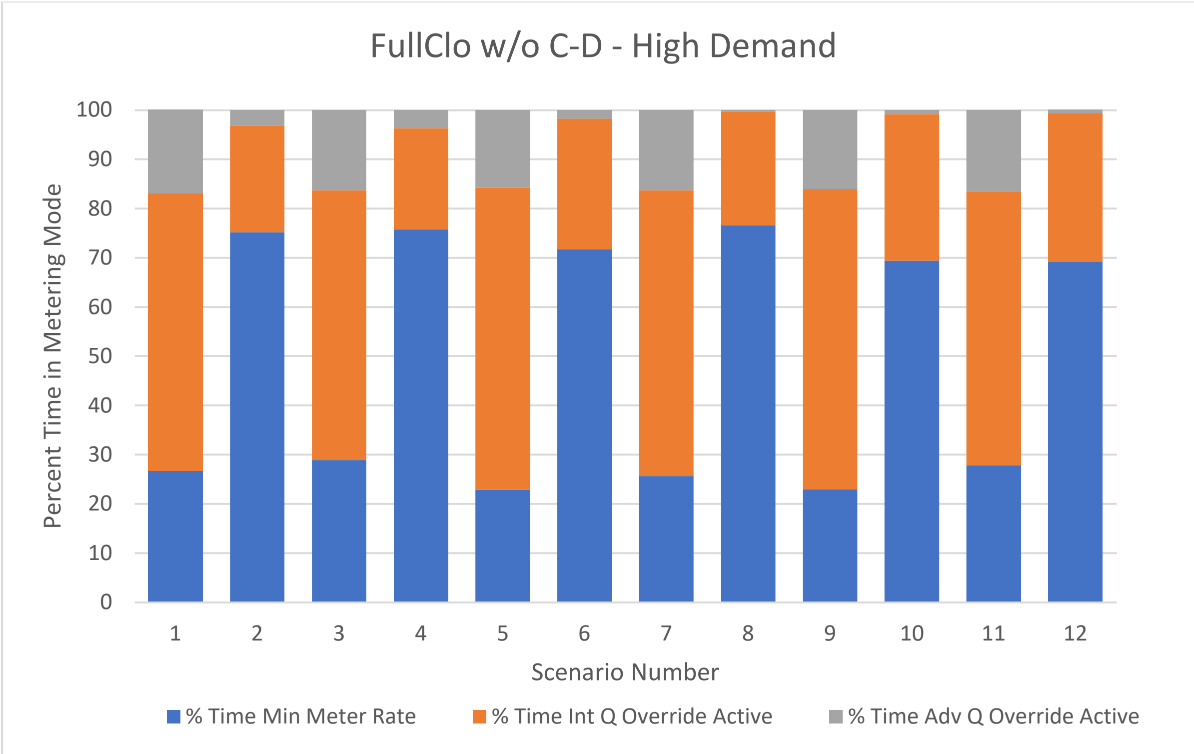


(a)

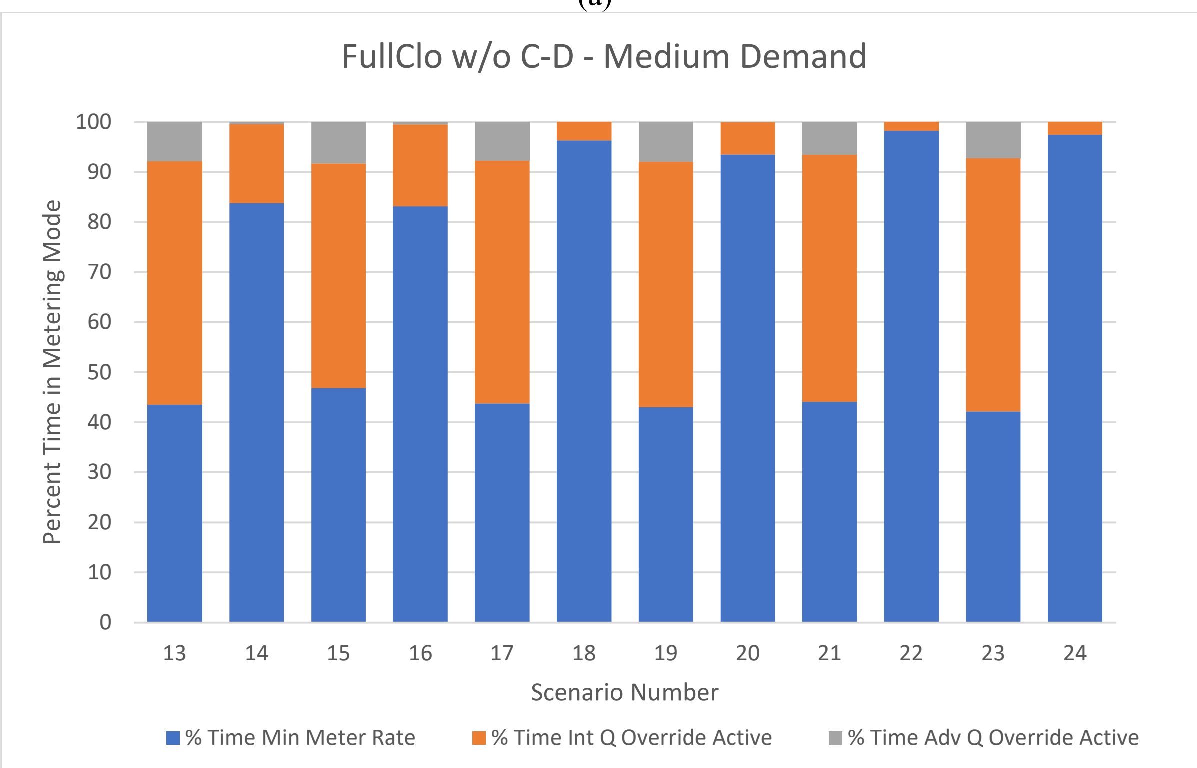


(b)

Figure C-11. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for FullClo with C-D: (a) high- (b) medium-demand scenarios



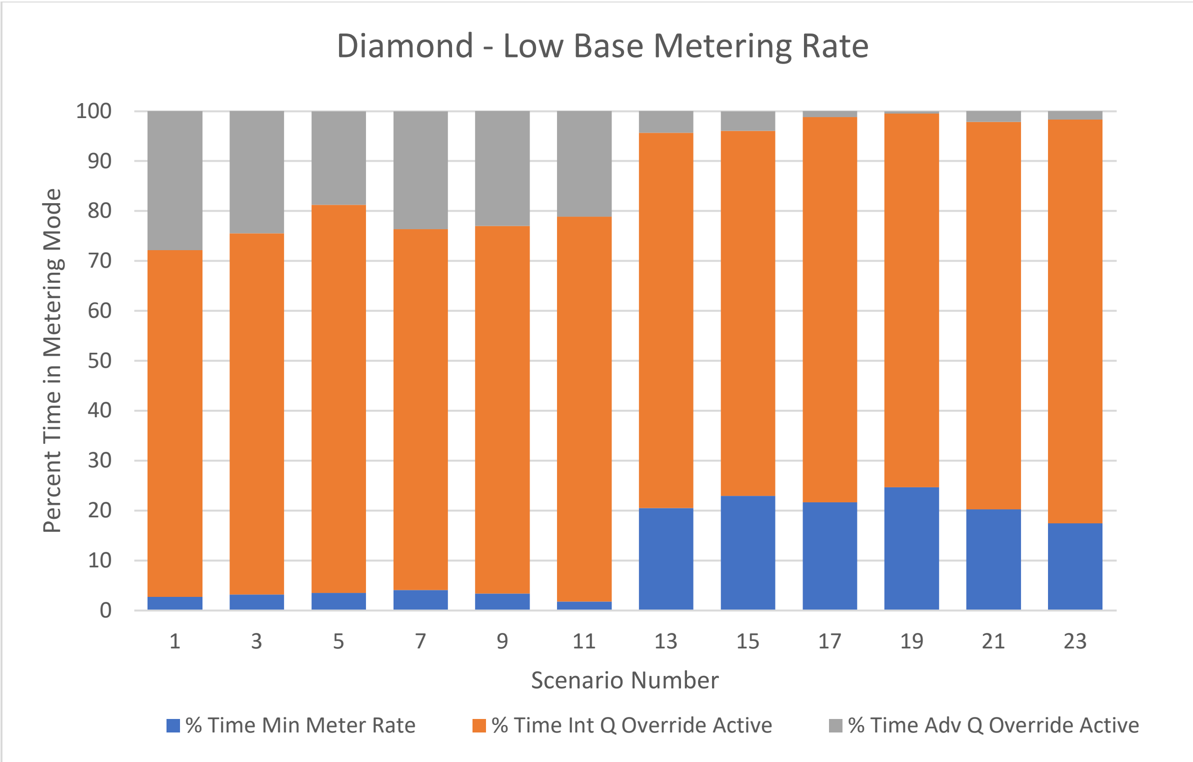
(a)



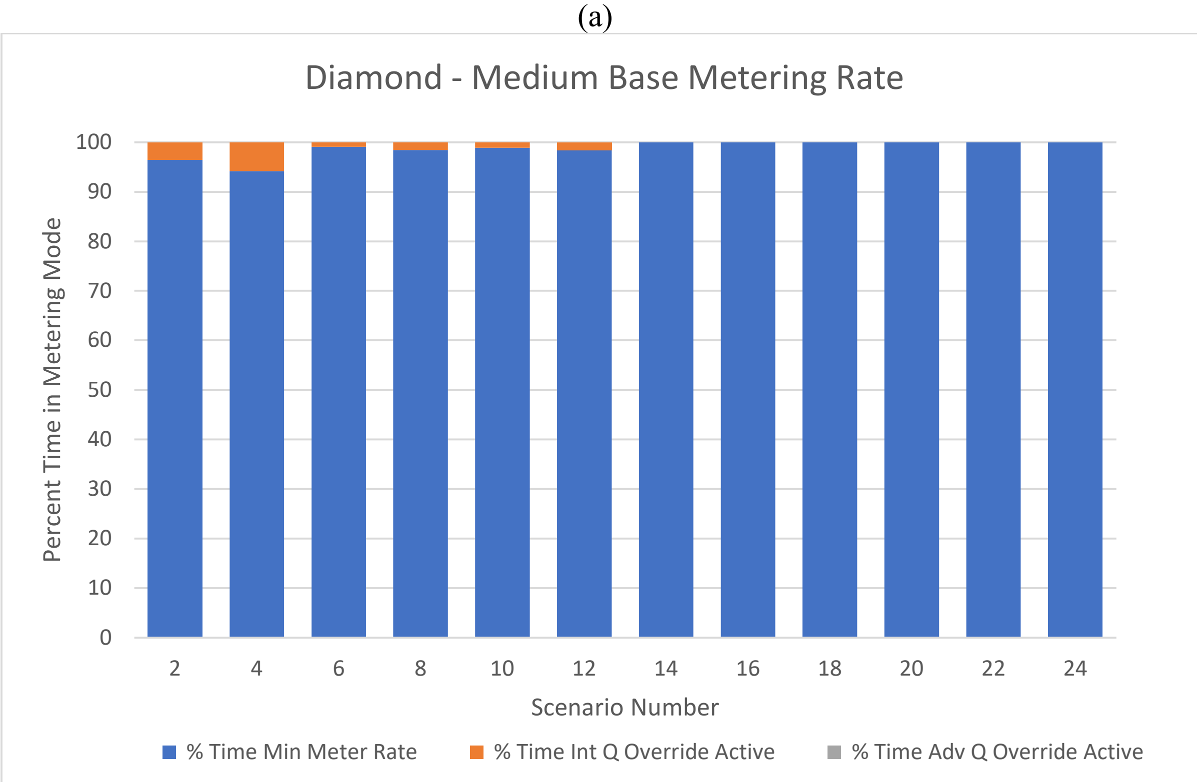
(b)

Figure C-12. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for FullClo w/o C-D: (a) high- (b) medium-demand scenarios

The results presented in Figure C-13 through Figure C-18 are grouped by the base metering rate level. The odd-numbered scenarios use the low base metering rate (240 veh/h). The even-numbered scenarios use the medium base metering rate (550 veh/h).

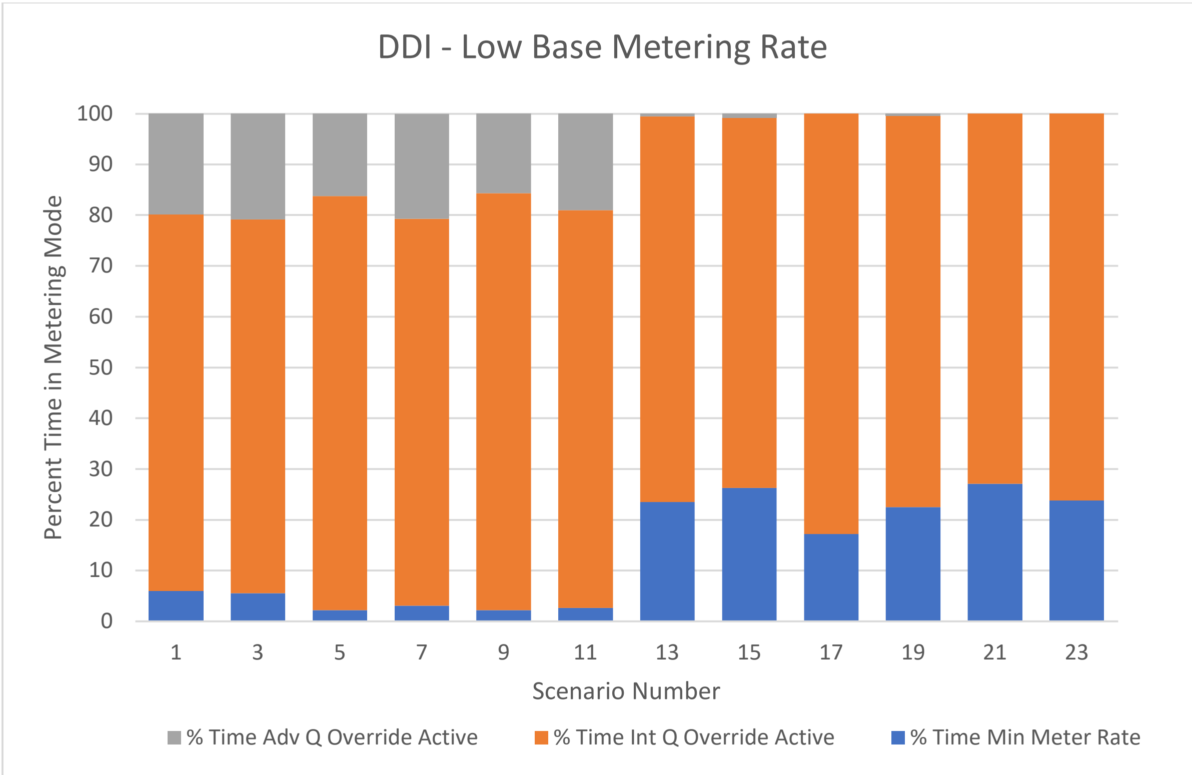


(a)

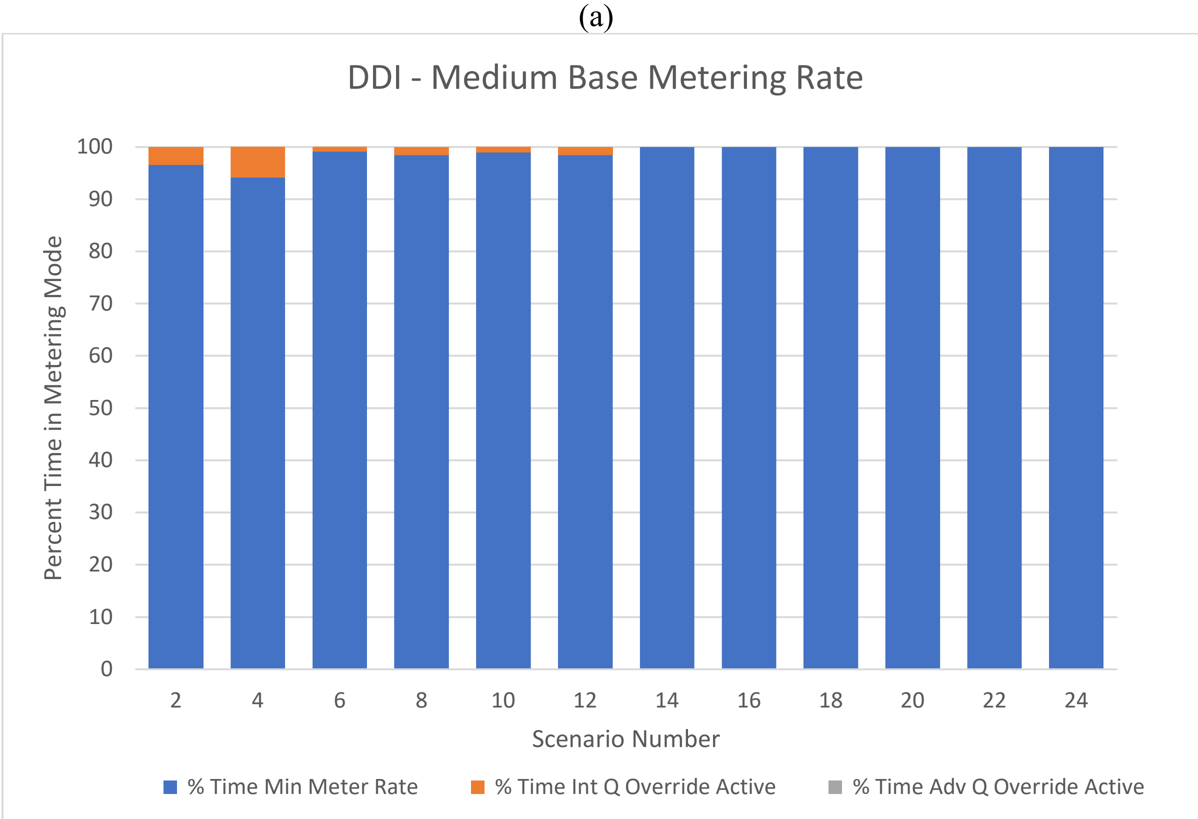


(b)

Figure C-13. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for Diamond: (a) low (b) medium base metering rate scenarios

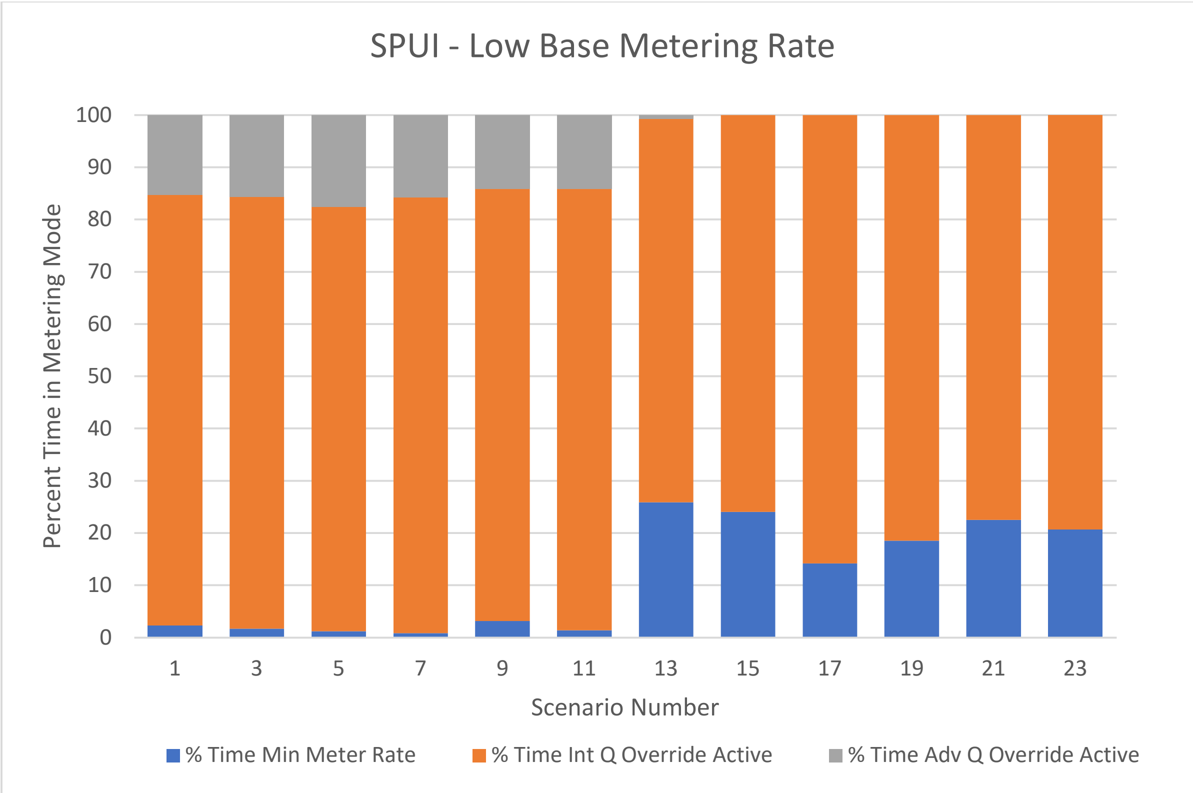


(a)

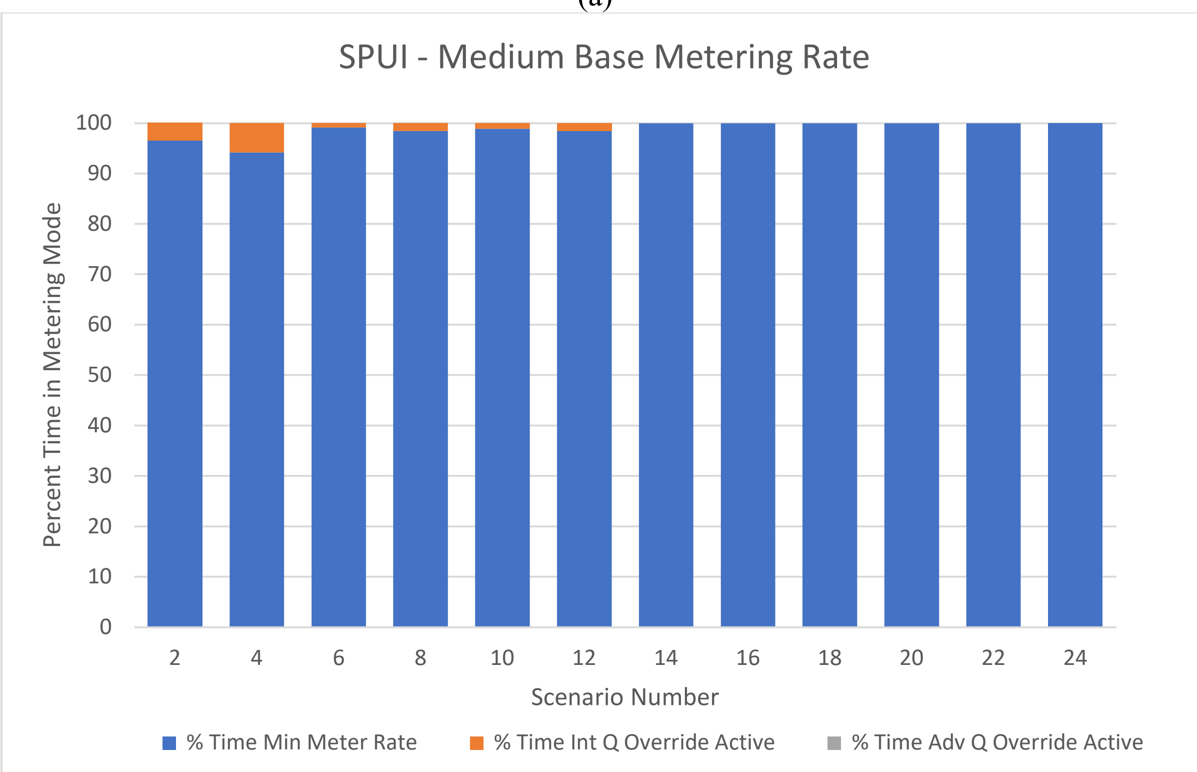


(b)

Figure C-14. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for DDI: (a) low (b) medium base metering rate scenarios

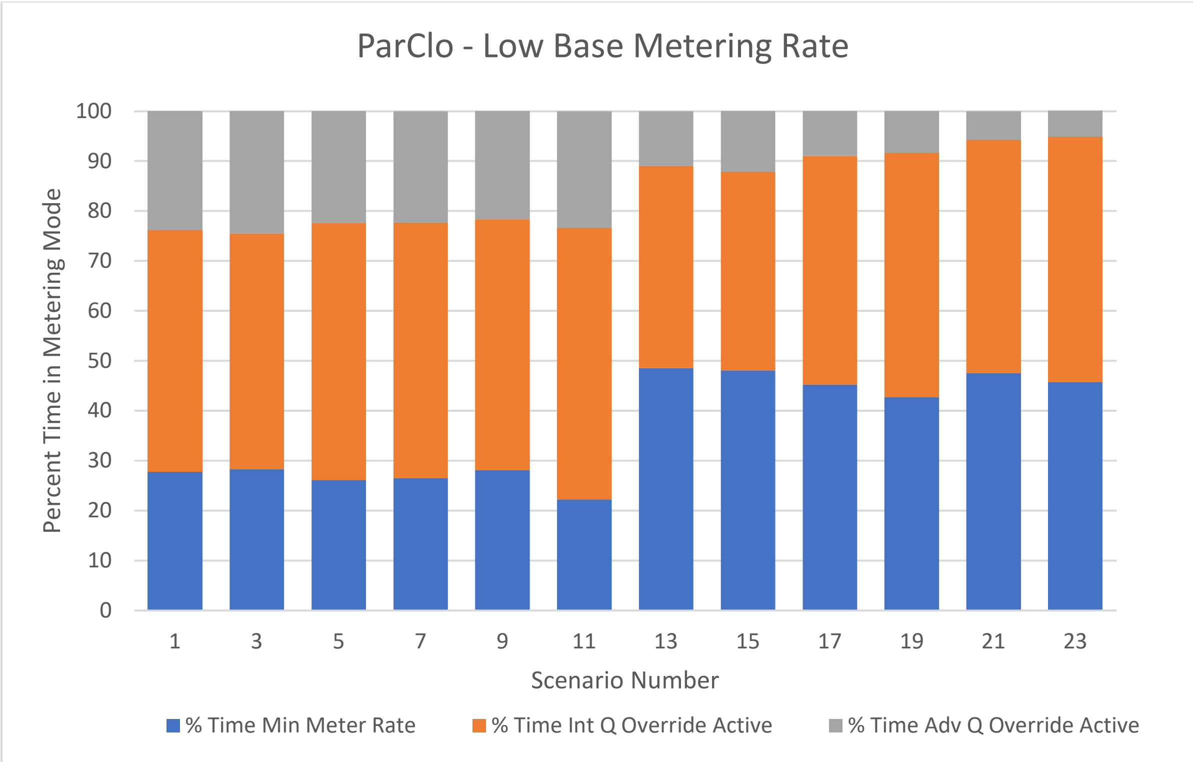


(a)

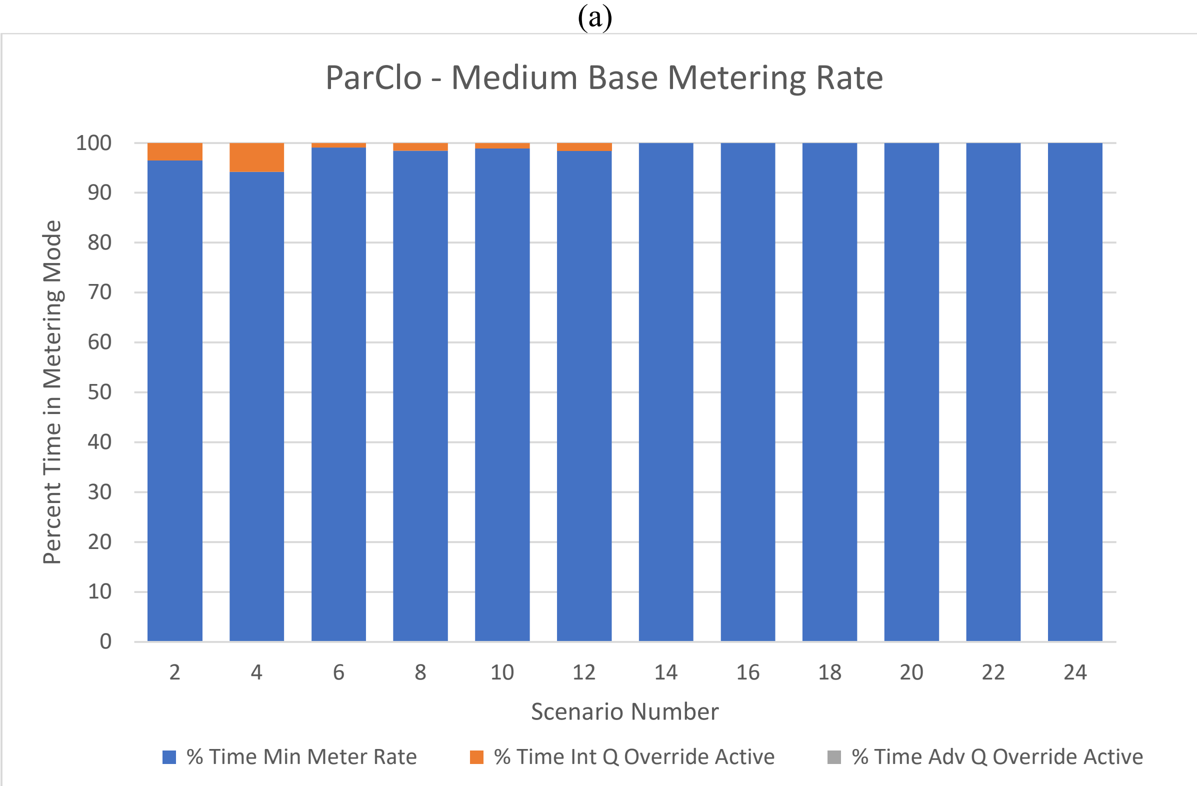


(b)

Figure C-15. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for SPUI: (a) low (b) medium base metering rate scenarios

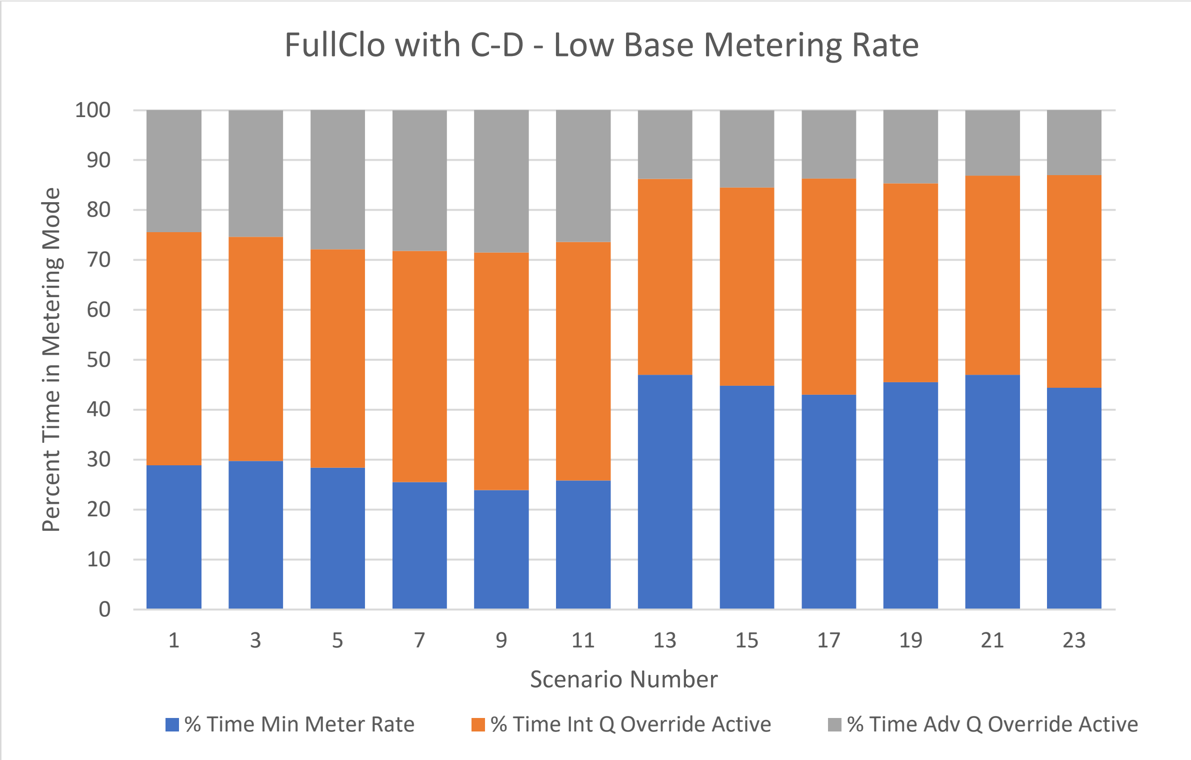


(a)

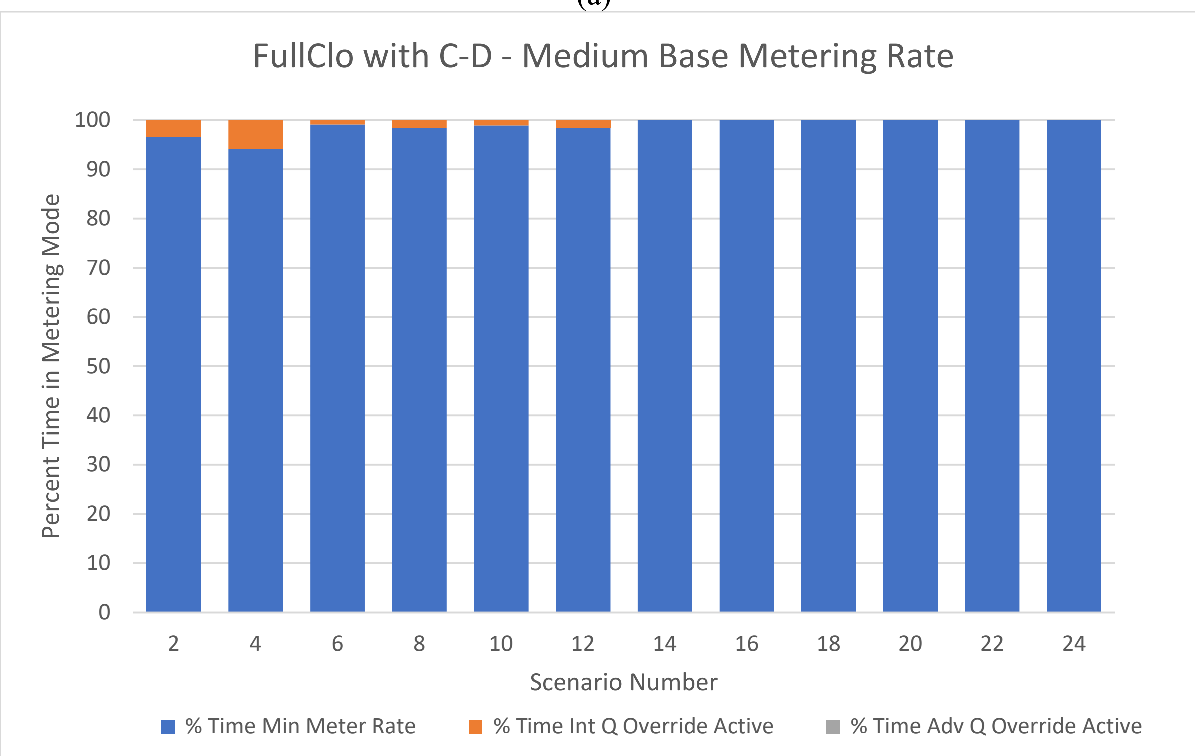


(b)

Figure C-16. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for ParClo: (a) low (b) medium base metering rate scenarios

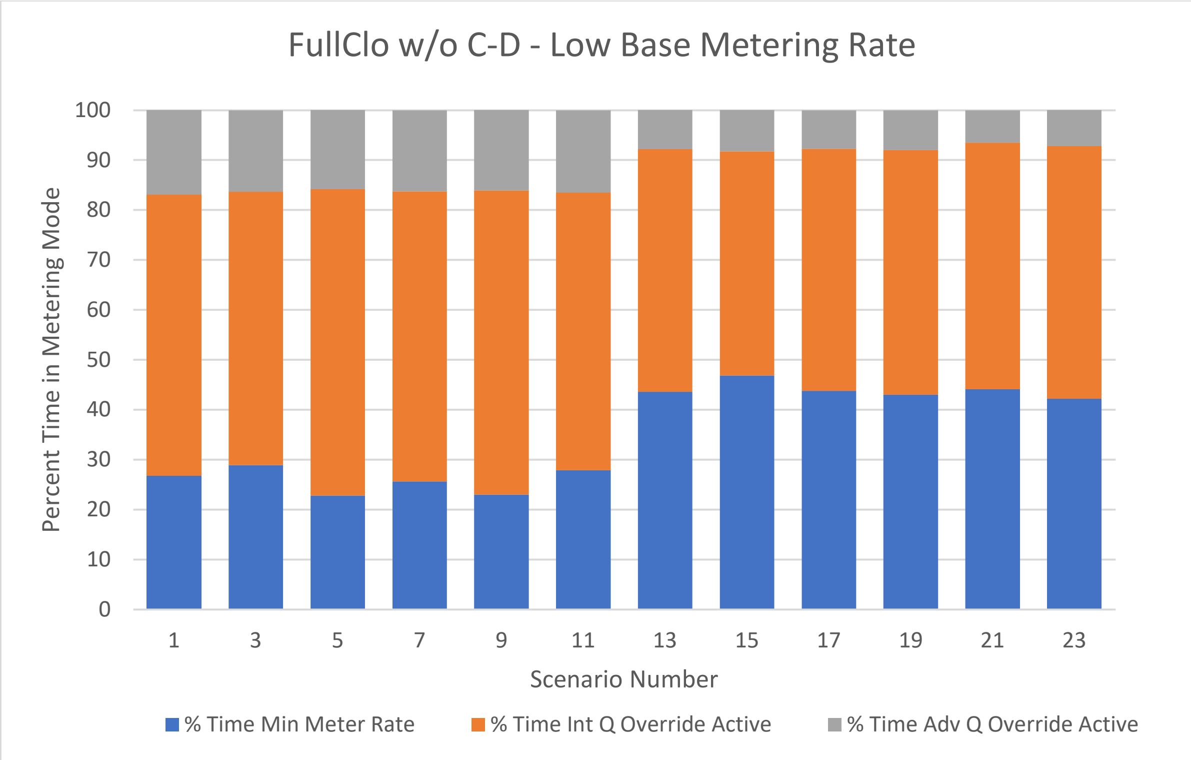


(a)

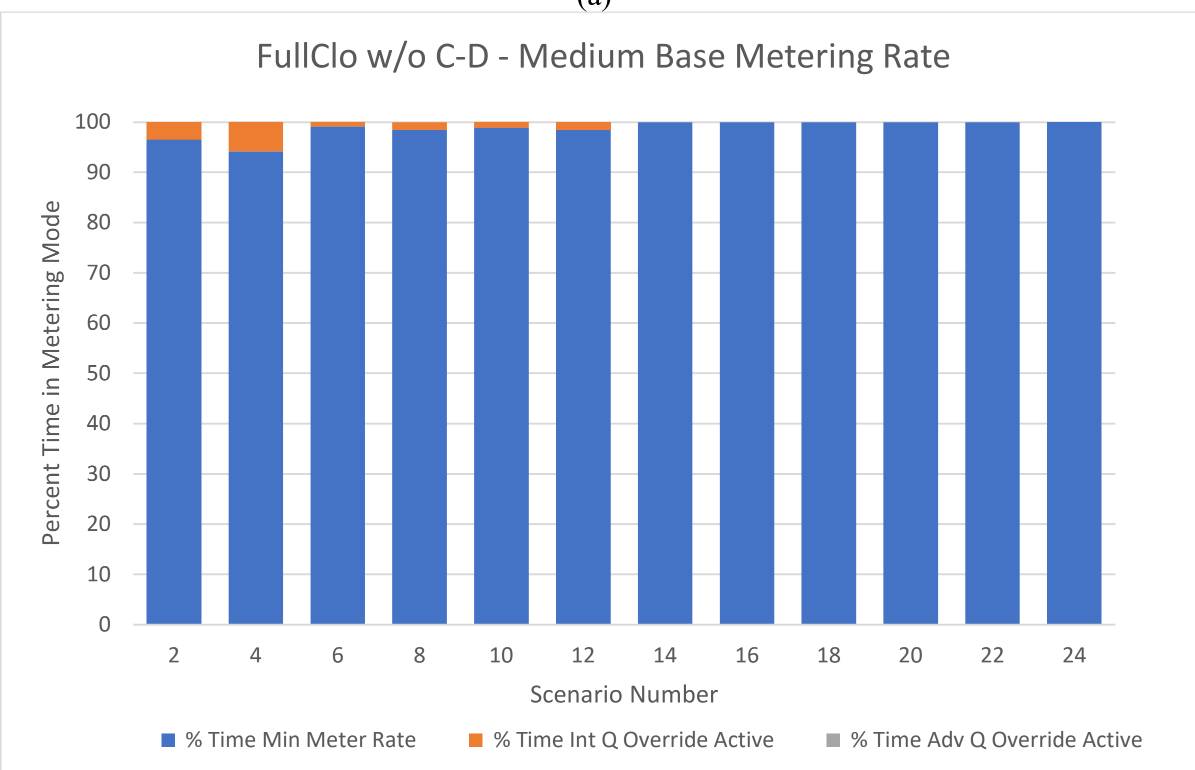


(b)

Figure C-17. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for FullClo with C-D: (a) low (b) medium base metering rate scenarios



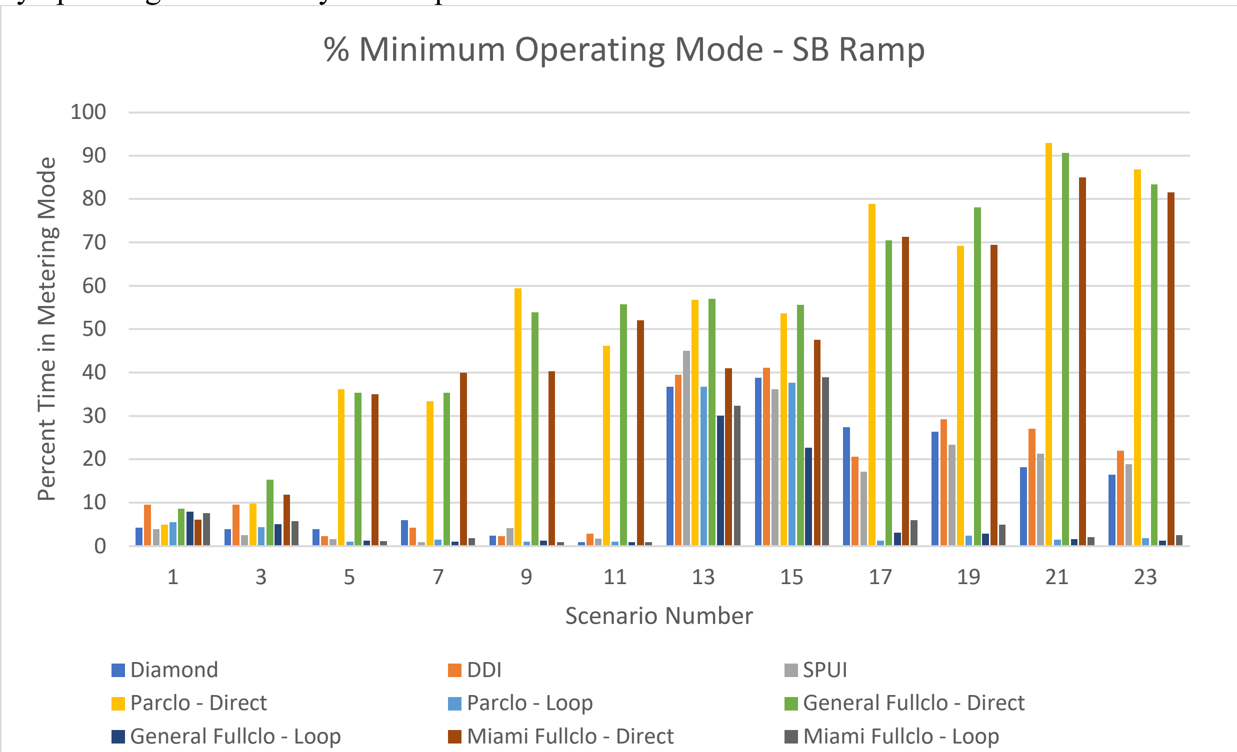
(a)



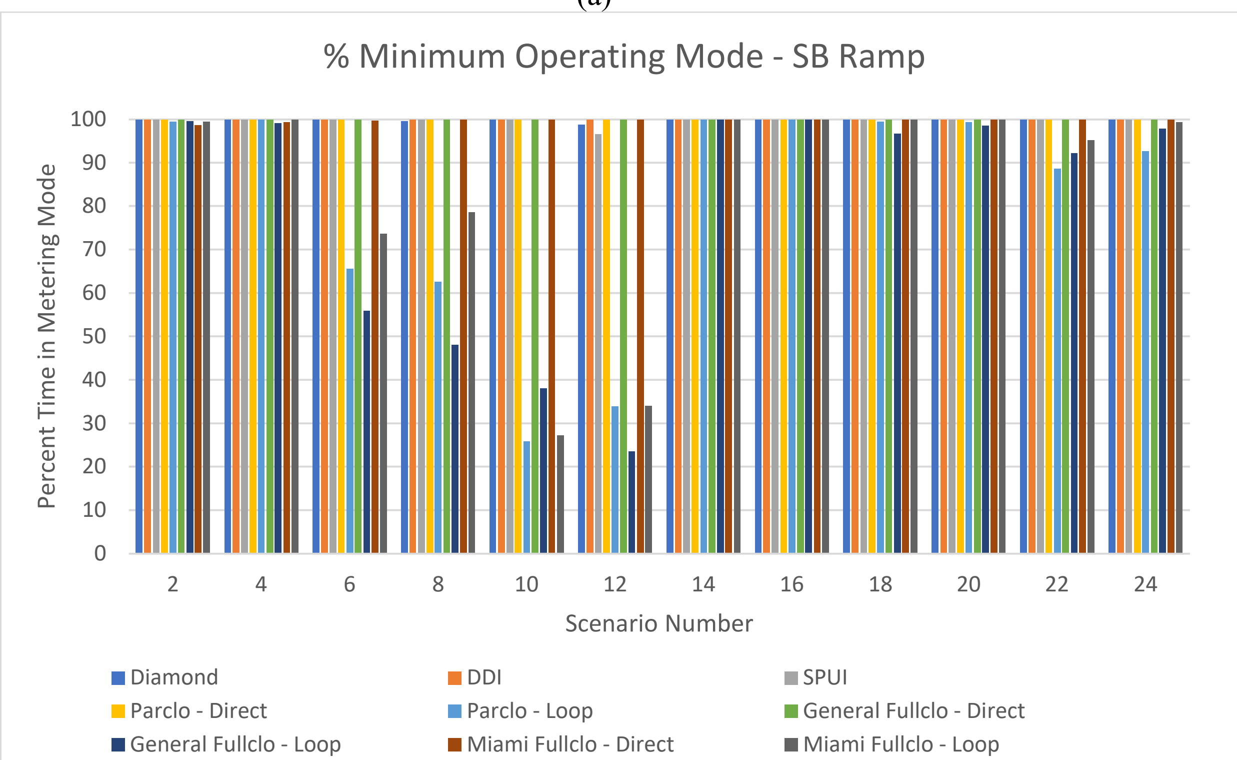
(b)

Figure C-18. Percent time spent in base, intermediate queue override, and advance queue override metering modes, for FullClo w/o C-D: (a) low (b) medium base metering rate scenarios

The results presented in Figure C-19 through Figure C-24 show the results for all interchanges by operating mode and by on-ramp.

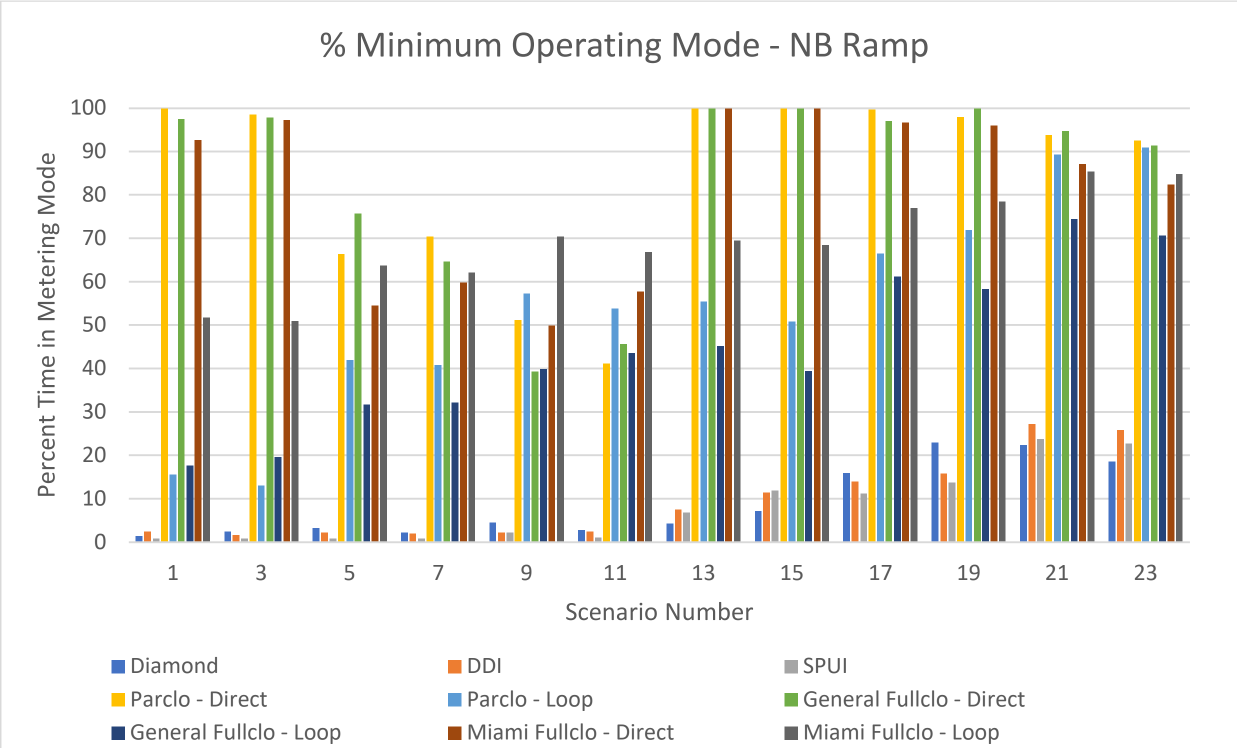


(a)

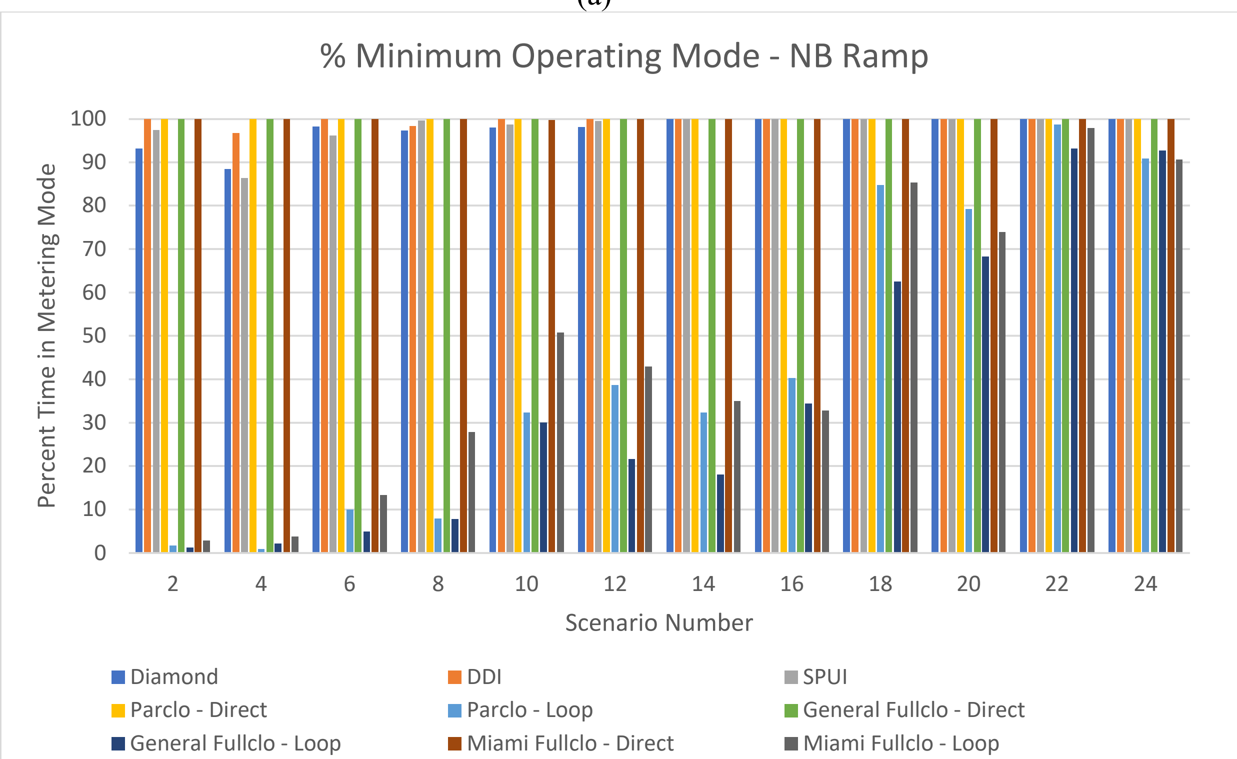


(b)

Figure C-19. Percent time spent in base metering mode for SB on-ramp: (a) low (b) medium base metering rate scenarios

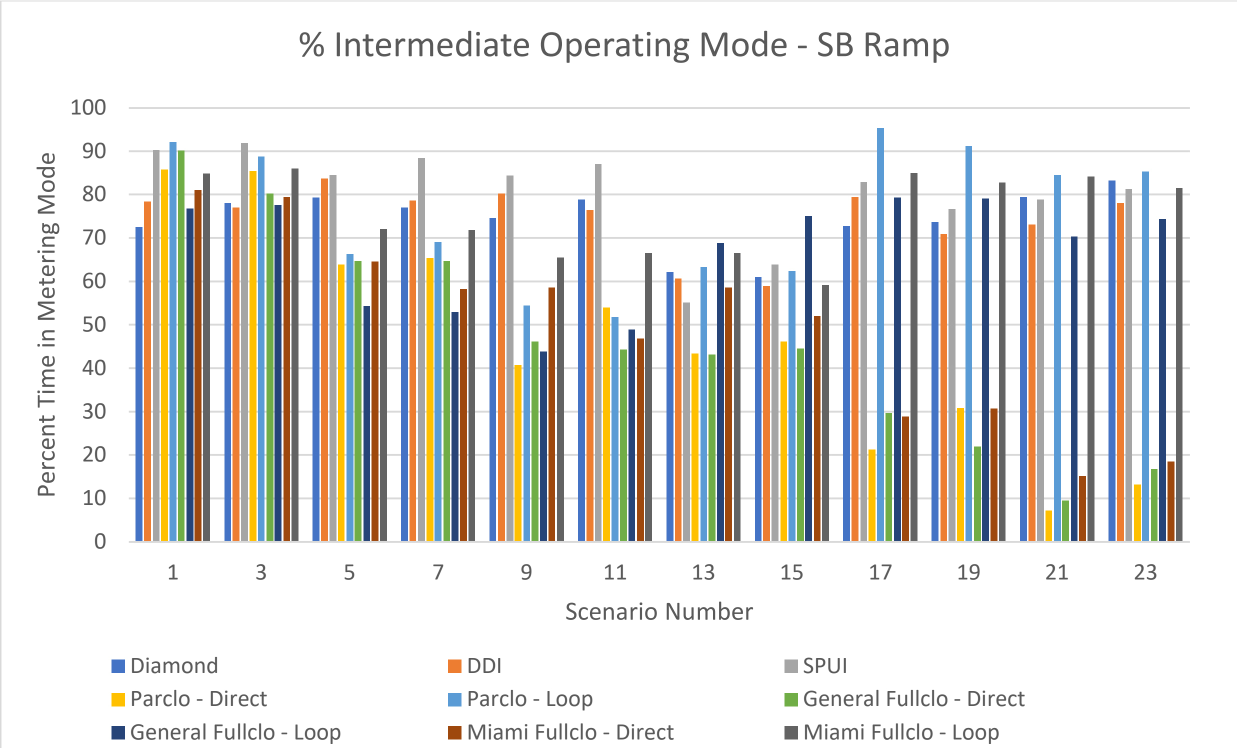


(a)

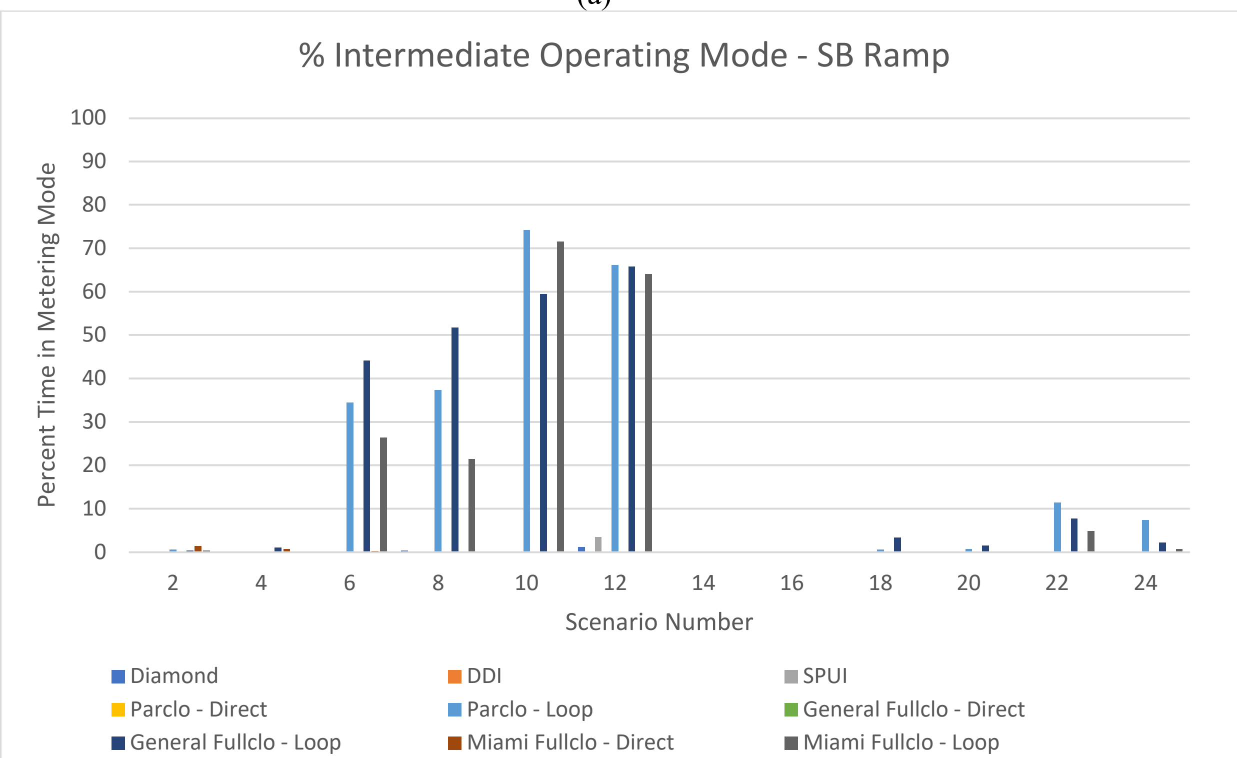


(b)

Figure C-20. Percent time spent in base metering mode for NB on-ramp: (a) low (b) medium base metering rate scenarios

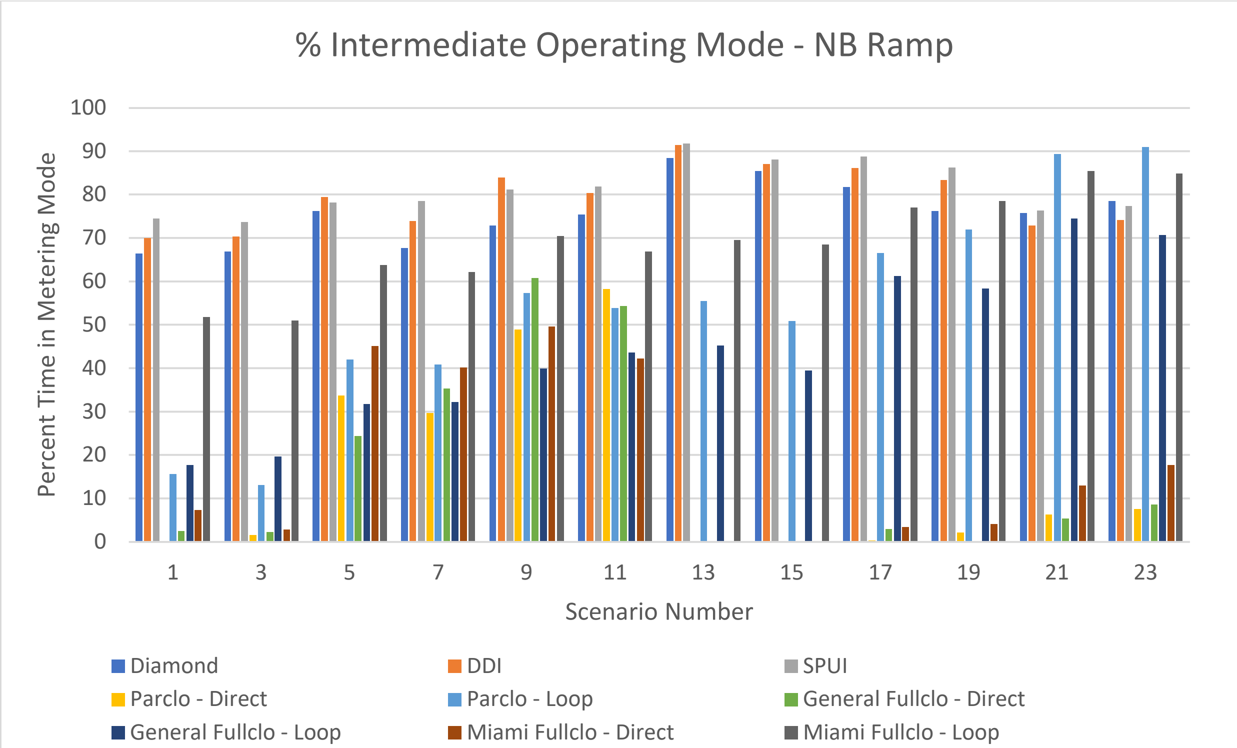


(a)

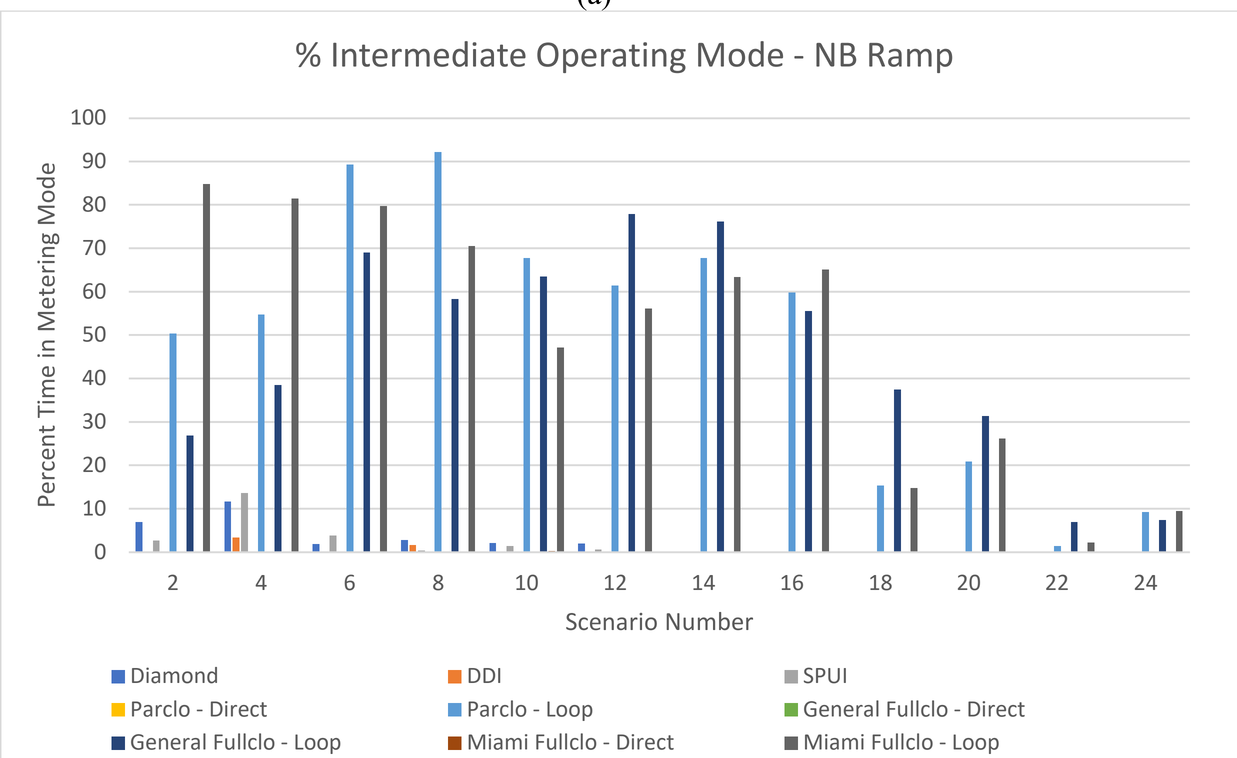


(b)

Figure C-21. Percent time spent in intermediate queue override mode for SB on-ramp: (a) low (b) medium base metering rate scenarios

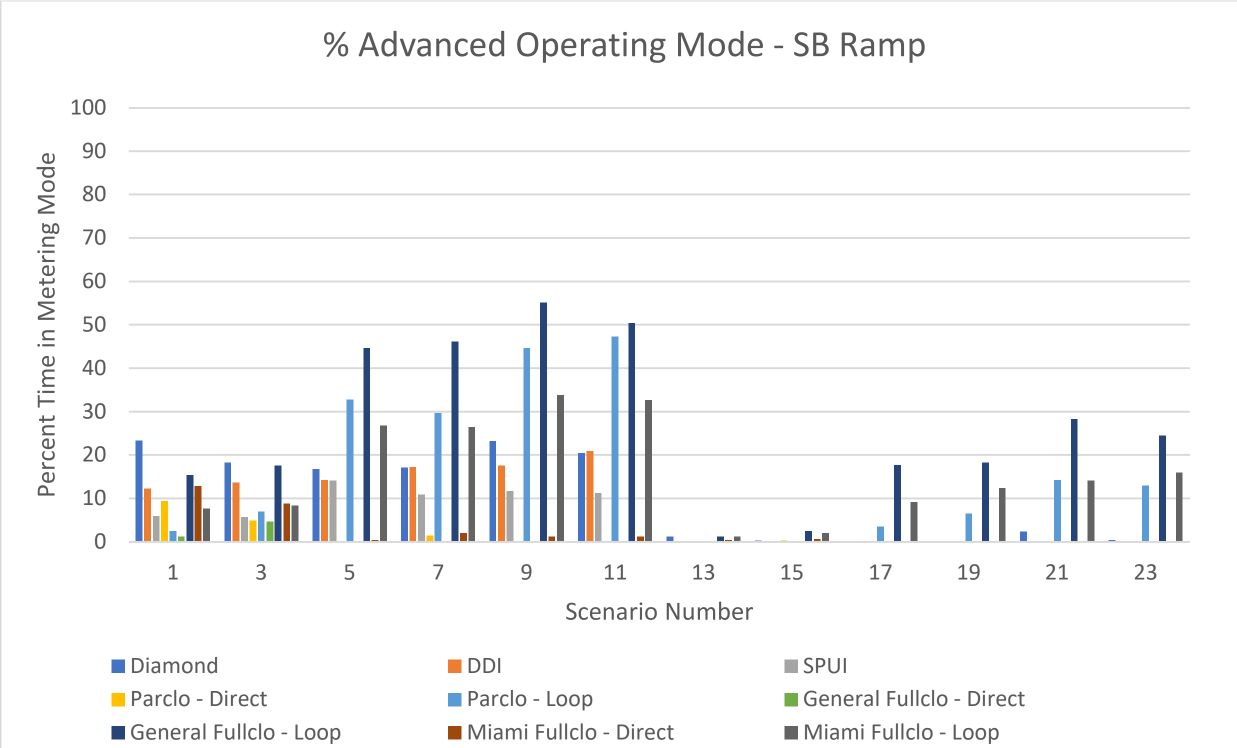


(a)

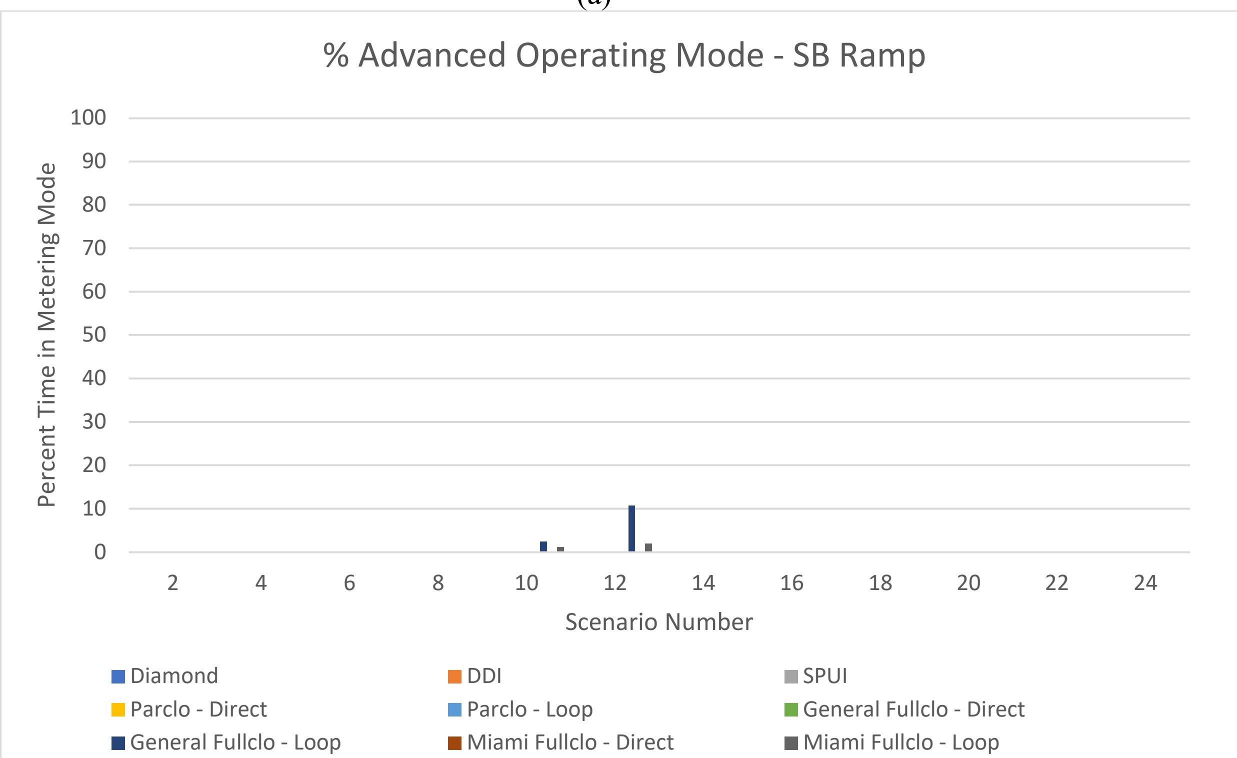


(b)

Figure C-22. Percent time spent in intermediate queue override mode for NB on-ramp: (a) low (b) medium base metering rate scenarios

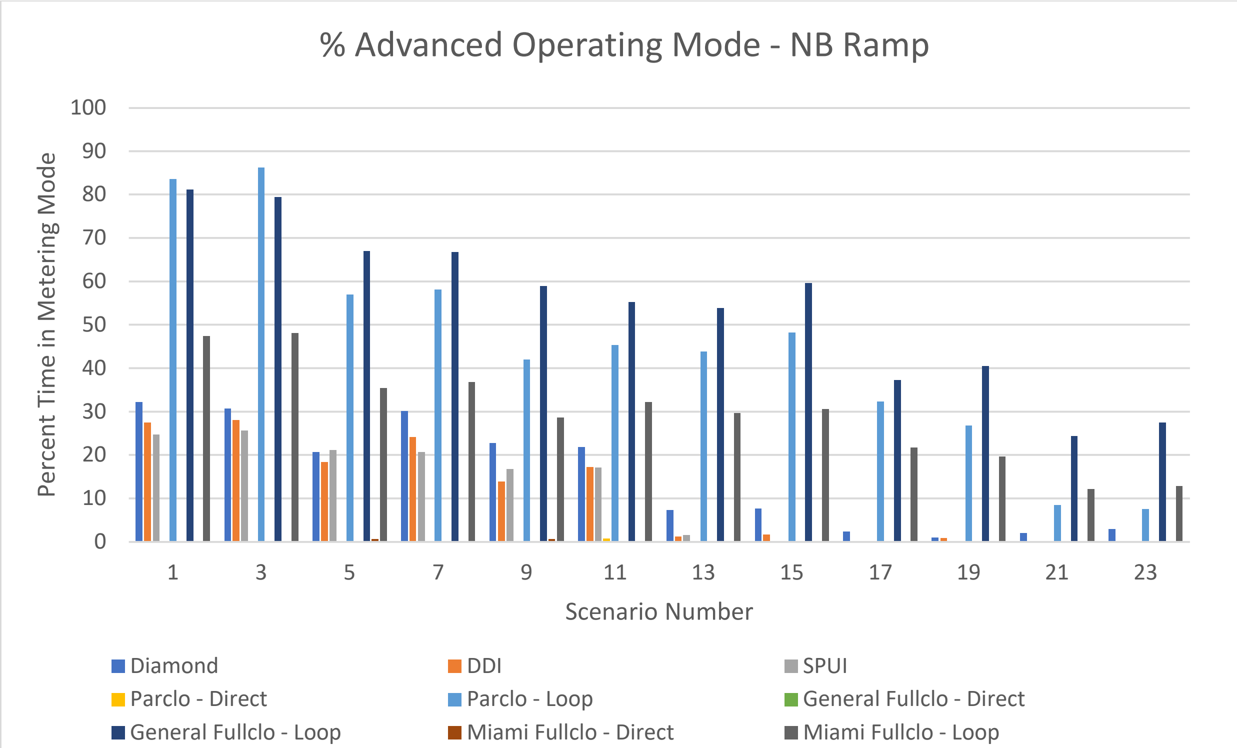


(a)

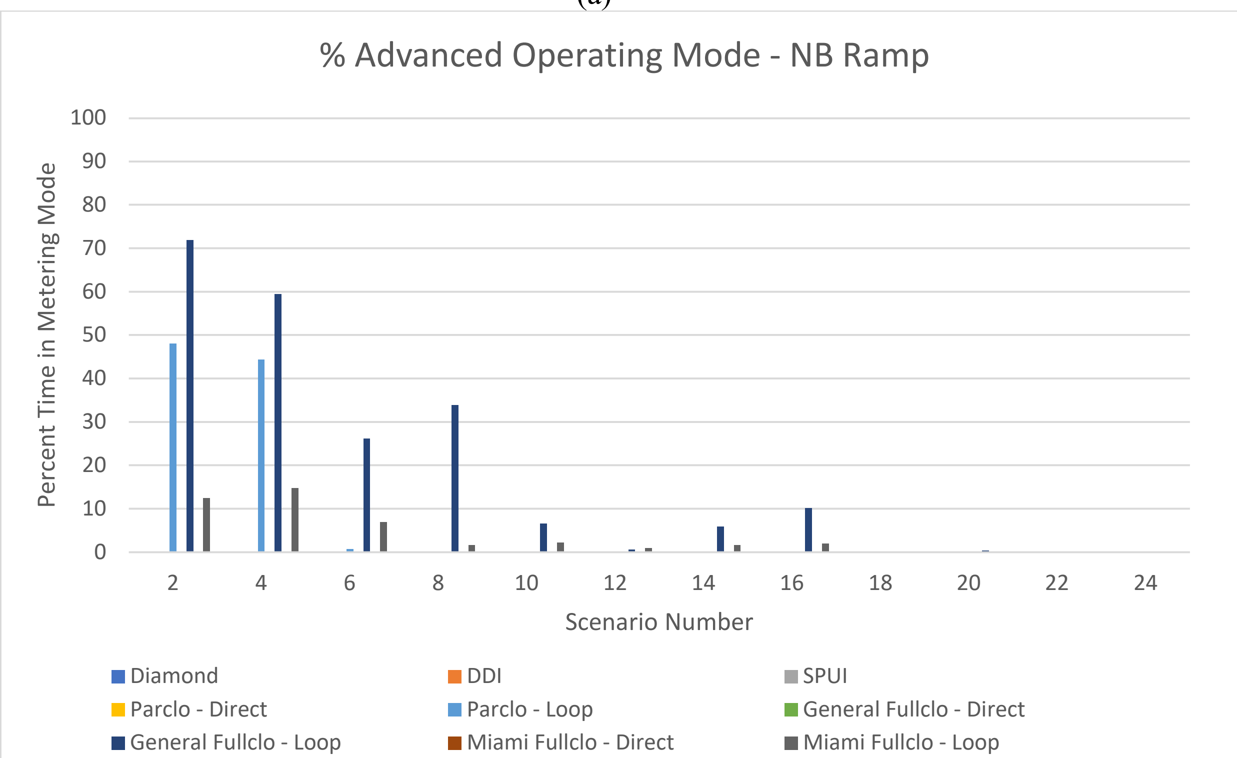


(b)

Figure C-23. Percent time spent in advance queue override mode for SB on-ramp: (a) low (b) medium base metering rate scenarios



(a)



(b)

Figure C-24. Percent time spent in advance queue override mode for NB on-ramp: (a) low (b) medium base metering rate scenarios

Tables C-1 through C-4 present the average metering rates and average metering delays.

Table C-1. Average metering rate (veh/h), Diamond, DDI, SPUI, ParClo interchanges

	Diamond		DDI		SPUI		ParClo					
Scenario	**NB**	**SB**	**NB**	**SB**	**NB**	**SB**	**NB Direct**	**NB Loop**	**NB Total**	**SB Direct**	**SB Loop**	**SB Total**
1	579	471	519	457	553	457	120	415	535	231	219	451
2	571	490	556	481	573	483	124	417	540	230	247	477
3	568	460	534	485	555	459	122	423	545	220	228	448
4	589	484	538	495	595	467	129	413	543	232	251	483
5	520	494	509	478	539	496	155	352	508	183	293	476
6	549	504	523	503	563	511	160	367	527	189	315	504
7	552	495	508	471	540	487	151	354	504	189	286	475
8	546	512	512	497	551	511	160	368	528	202	316	518
9	521	528	495	500	515	483	168	316	484	161	321	482
10	549	523	528	516	549	523	177	344	521	173	348	521
11	524	520	496	497	519	491	180	325	505	173	329	502
12	538	534	508	526	538	555	184	338	522	188	345	532
13	458	361	408	356	432	355	100	318	418	166	182	348
14	452	360	433	385	480	386	102	343	445	179	181	360
15	452	360	414	371	417	370	95	328	423	168	182	350
16	461	385	443	365	453	394	94	340	433	179	194	373
17	413	384	398	377	420	405	120	292	412	144	223	367
18	422	394	416	405	439	406	127	293	420	155	241	396
19	396	384	413	393	415	396	123	278	400	154	229	382
20	421	406	419	400	434	415	126	299	425	147	252	399
21	399	410	386	398	393	395	128	234	362	129	247	377
22	408	406	410	409	440	415	133	270	402	136	282	418
23	414	407	379	385	400	403	130	233	363	137	246	382
24	419	414	399	420	425	420	132	285	418	131	276	407

Table C-2. Average metering rate (veh/h), FullClo with C-D, FullClo w/o C-D interchanges

	FullClo with C-D						FullClo w/o C-D					
Scenario	**NB Direct**	**NB Loop**	**NB Total**	**SB Direct**	**SB Loop**	**SB Total**	**NB Direct**	**NB Loop**	**NB Total**	**SB Direct**	**SB Loop**	**SB Total**
1	123	409	532	212	245	456	128	326	454	240	226	466
2	119	433	551	230	260	489	120	381	501	225	250	475
3	124	403	527	213	254	468	121	328	450	224	230	453
4	125	419	544	227	263	490	119	382	501	228	248	476
5	147	371	518	182	321	504	166	300	465	185	279	464
6	151	388	540	194	324	518	166	367	534	197	305	502
7	158	372	530	183	325	508	160	301	461	185	274	459
8	164	394	557	201	331	532	160	350	510	200	302	501
9	179	352	531	166	348	513	170	283	453	183	293	476
10	168	350	518	184	343	527	178	328	506	181	352	533
11	173	343	516	164	336	500	163	289	452	171	292	464
12	188	353	540	180	362	542	180	336	516	173	347	520
13	99	338	437	165	193	357	101	285	387	179	187	366
14	101	360	461	173	198	371	100	344	444	176	191	366
15	96	352	449	165	202	367	101	285	386	174	185	359
16	96	350	446	179	202	381	100	346	446	189	202	391
17	123	298	421	151	257	408	117	263	380	149	233	382
18	118	315	433	149	262	412	128	292	420	158	238	396
19	118	307	425	143	258	400	122	260	382	151	240	391
20	125	308	433	151	262	414	133	305	437	160	244	404
21	129	269	398	128	283	411	134	241	375	135	245	380
22	134	275	408	134	288	422	146	274	419	137	265	401
23	132	277	408	137	272	408	139	243	382	138	252	390
24	136	281	416	139	279	418	140	276	416	141	269	410

Table C-3. Average metering delay (s/veh), Diamond, DDI, SPUI, ParClo interchanges

	Diamond		DDI		SPUI		ParClo					
Scenario	**NB**	**SB**	**NB**	**SB**	**NB**	**SB**	**NB Direct**	**NB Loop**	**NB Total**	**SB Direct**	**SB Loop**	**SB Total**
1	489	458	686	532	475	449	207	461	668	398	530	927
2	131	47	97	35	120	41	17	403	420	38	57	95
3	438	427	804	493	455	448	209	433	641	351	536	887
4	138	46	134	33	130	39	18	376	394	40	64	104
5	465	426	699	594	466	487	354	509	863	376	600	976
6	104	56	64	42	106	56	20	278	299	26	156	182
7	474	453	711	569	452	455	318	489	807	353	558	911
8	93	61	100	43	77	54	20	254	274	27	160	187
9	485	518	678	636	476	477	370	576	946	365	547	912
10	93	56	63	63	72	62	23	190	213	22	205	227
11	488	456	690	626	454	451	356	541	897	359	525	885
12	78	69	53	46	65	86	24	195	219	26	187	213
13	415	341	565	392	389	340	61	555	616	356	375	732
14	48	27	31	22	48	32	17	194	211	24	27	51
15	471	302	574	380	380	315	57	531	589	351	338	688
16	48	29	33	22	46	32	15	171	186	22	29	51
17	433	319	502	440	377	350	212	588	799	340	611	951
18	38	30	28	22	39	34	17	131	148	22	52	73
19	357	298	474	448	340	344	201	575	776	342	550	891
20	38	32	26	22	36	34	18	117	135	19	63	81
21	389	351	430	449	354	361	264	646	910	270	608	878
22	34	33	25	23	36	36	18	74	92	18	99	117
23	357	334	405	408	337	354	282	592	874	322	594	916
24	33	33	27	23	34	34	18	97	115	18	89	107

Table C-4. Average metering delay (s/veh), FullClo with C-D, FullClo w/o C-D interchanges

	FullClo with C-D						FullClo w/o C-D					
Scenario	NB Direct	NB Loop	NB Total	SB Direct	SB Loop	SB Total	NB Direct	NB Loop	NB Total	SB Direct	SB Loop	SB Total
1	444	369	812	250	278	528	205	342	547	172	236	409
2	29	58	87	17	244	261	30	50	81	12	194	207
3	393	357	751	248	270	518	203	324	527	116	225	342
4	34	64	98	16	230	246	28	50	78	13	181	194
5	381	322	704	417	293	710	200	289	489	244	266	510
6	21	168	190	18	227	245	25	152	177	17	191	208
7	369	309	678	388	280	668	185	280	465	232	246	479
8	26	167	193	19	214	233	22	140	162	17	174	192
9	396	309	705	385	298	684	196	277	474	242	276	518
10	19	184	203	19	192	211	21	184	205	20	175	195
11	352	305	658	375	289	664	186	257	443	227	253	480
12	19	193	212	26	187	213	19	169	187	18	162	180
13	371	371	742	53	310	363	206	356	563	69	278	347
14	19	27	46	15	202	216	17	26	43	10	180	190
15	369	366	735	72	288	360	194	334	527	63	260	323
16	18	28	46	15	184	198	18	30	48	11	171	182
17	386	373	759	227	336	562	185	333	518	118	301	419
18	18	73	91	16	158	174	16	48	65	12	120	132
19	337	352	690	143	317	460	183	314	496	143	289	432
20	16	56	71	16	133	150	17	41	58	14	144	158
21	285	350	635	306	355	661	159	323	482	190	327	517
22	17	115	131	17	91	107	13	71	84	14	85	99
23	282	348	629	289	340	629	153	302	455	197	303	499
24	15	93	108	17	99	116	14	75	89	14	86	100

APPENDIX D: SUPPLEMENTAL INFORMATION FOR THREE-LANE RAMP METERING

Chapter 2 (Literature Review) contained some information about multilane metering, primarily two lanes. This section provides some supplemental information regarding metering for three on-ramp lanes.

Geometric design and operation of multilane metered on-ramps is not discussed comprehensively in any of the existing ramp-metering design manuals. California, Nevada, Oregon, and Wisconsin are the only states that have warrants for 3-lane ramp metering. Additionally:

- California has provided typical layouts for 3-lane metered diagonal on-ramp, 3-lane metered loop on-ramp, and 3-lane metered connector as well as typical signing and pavement marking for 3-lane metered loop on-ramp including the regulatory signs used for simultaneous and staggered release operations of multilane metered on-ramps.
- Utah has provided typical detection for 3-lane metered on-ramp.
- Wisconsin has provided the regulatory and optional signs for median-separated and non-median separated 3-lane metered on-ramps in addition to their locations with respect to the stop bar.
- Wang and Dang (2012) proposed locations of overhead- and side-mounted signal heads for a 3-lane metered loop on-ramp.

Table D-1 provides location and coordinates of sample three-lane metered on-ramps in California.

Table D-1. Sample three-lane metered on-ramps in California

Site #	City	Latitude, Longitude
1	Sherman Oaks	34°08'47.6"N, 118°28'15.6"W
2	San Diego	32°47'27.03"N, 117° 6'44.97"W
3	Sunnyvale	37°20'2.38"N, 122° 3'23.78"W
4	Santa Rosa	38°27'40.86"N, 122°43'33.32"W
5	Hercules	38°00'46.6"N, 122°16'18.9"W

Following are aerial and street-view figures for each site.

Site 1

Figure D-1. Three-lane metering, site 1: aerial view
Source: Google Earth

Figure D-2. Three-lane metering, site 1: street view
Source: Google Earth

Site 2

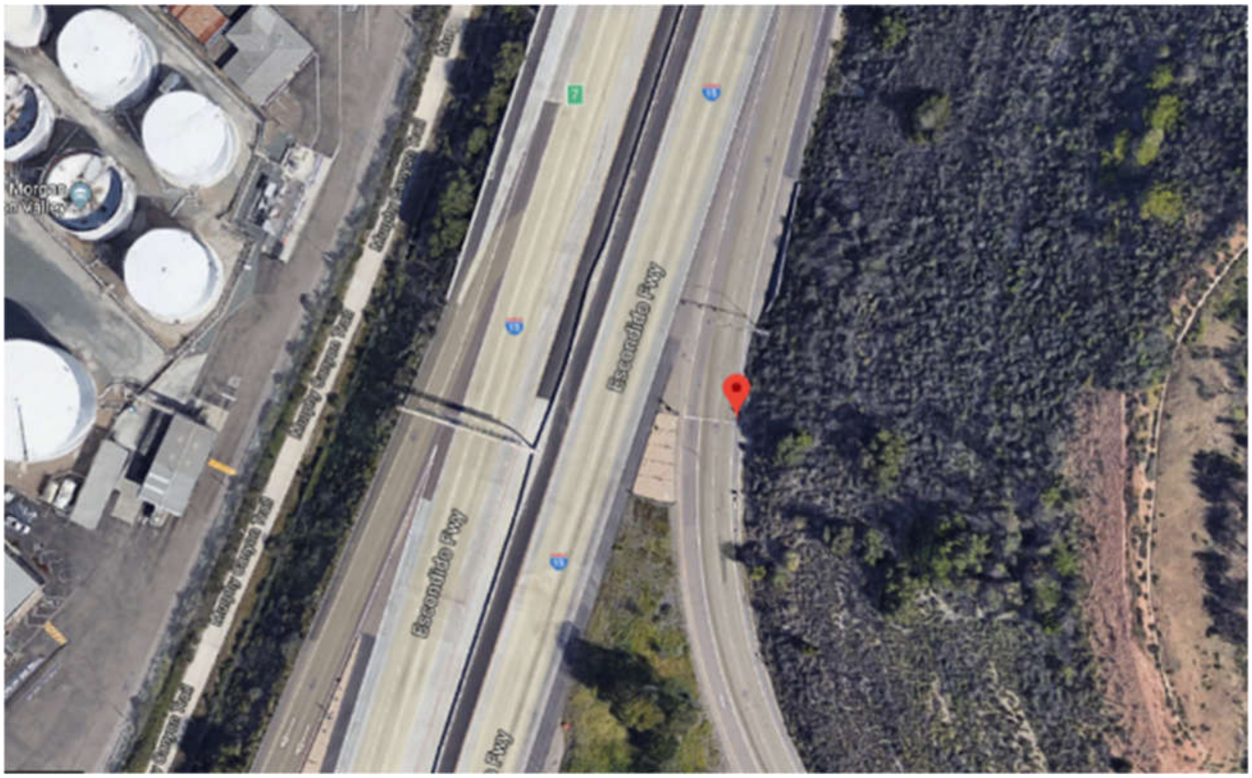


Figure D-3. Three-lane metering, site 2: aerial view
Source: Google Earth

Figure D-4. Three-lane metering, site 2: street view
Source: Google Earth

Site 3

Figure D-5. Three-lane metering, site 3: aerial view
Source: Google Earth

Figure D-6. Three-lane metering, site 3: street view
Source: Google Earth

Site 4

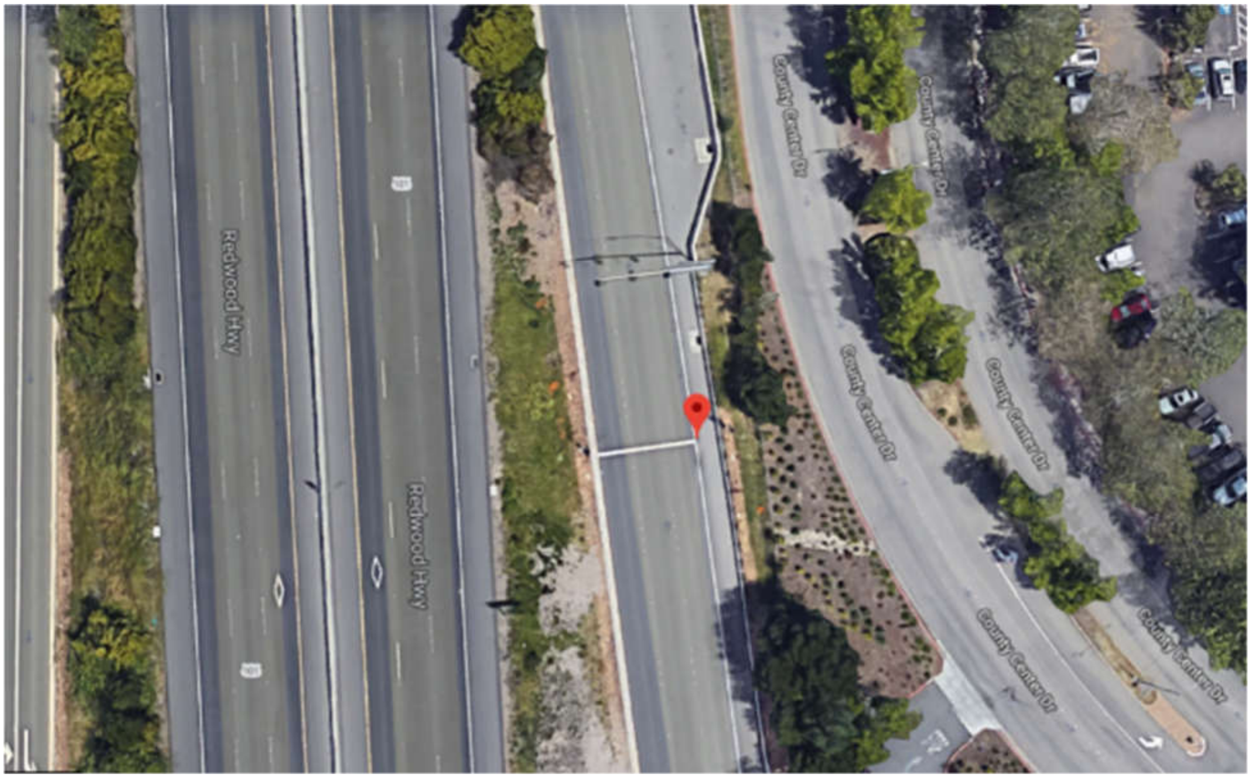


Figure D-7. Three-lane metering, site 4: aerial view
Source: Google Earth

Figure D-8. Three-lane metering, site 4: street view
Source: Google Earth

Site 5

Figure D-9. Three-lane metering, site 5: aerial view
Source: Google Earth

Figure D-10. Three-lane metering, site 5: street view
Source: Google Earth

Ramp-Metering Design Guides

California

Caltrans warrants installation of 3-lane ramp meters (2 SOV+1 HOV) at locations where the peak-hour volume exceeds 1800 veh/h and the percentage of HOVs on the ramp exceeds 9% of the ramp peak-hour volume.

There is no difference in calculating metering rates for GP lanes and HOV lanes in staggered operation of multi-lane meters. This makes coordinating staggered metering on all lanes possible. Staggered operation of multi-lane metered ramps in California is based on NTCIP 1207.

- Mutex (mutual exclusive): Only one lane in a dependency group shall indicate green at a time;
- Fixed offset (0-25.5 s in 0.1 s increment): there is an offset between green indications for two lanes in a dependency group;
- Fractional offset: the allocated green time for a dependency group is split equally between the lanes in that group.

There are no 4-lane metered on-ramps which are controlled by a single controller. However, there is a location in Fresno where four lanes are controlled in two dependency lane groups.

Figure D-11 through Figure D-16 are a sample of plan sheets from the Caltrans Ramp Metering Design Manual (2016) for configuring on-ramps for multilane metering.

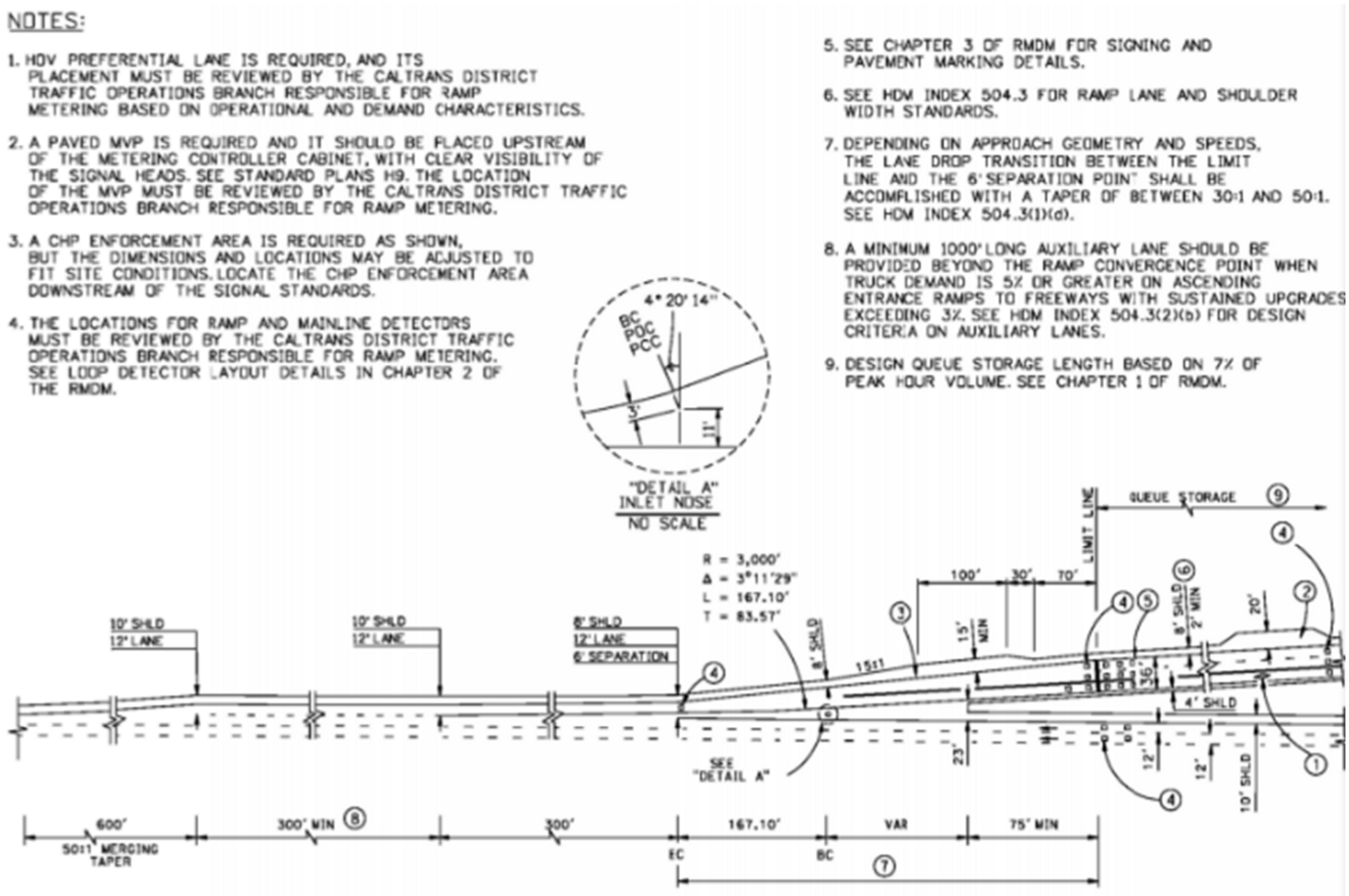


Figure D-11. Typical 3-lane metered diagonal on-ramp (2 GP lanes+1 HOV lane)
Source: Figure 1-3 of the Caltrans Ramp Metering Design Manual, 2016

NOTES:

1. HOV PREFERENTIAL LANE IS REQUIRED, AND ITS PLACEMENT MUST BE REVIEWED BY THE CALTRANS DISTRICT TRAFFIC OPERATIONS BRANCH RESPONSIBLE FOR RAMP METERING BASED ON OPERATIONAL AND DEMAND CHARACTERISTICS.

2. A PAVED MVP IS REQUIRED AND IT SHOULD BE PLACED UPSTREAM OF THE METERING CONTROLLER CABINET, WITH CLEAR VISIBILITY OF THE SIGNAL HEADS. SEE STANDARD PLANS H9. THE LOCATION OF THE MVP MUST BE REVIEWED BY THE CALTRANS DISTRICT TRAFFIC OPERATIONS BRANCH RESPONSIBLE FOR RAMP METERING.

3. A CHP ENFORCEMENT AREA IS REQUIRED AS SHOWN, BUT THE DIMENSIONS AND LOCATIONS MAY BE ADJUSTED TO FIT SITE CONDITIONS. LOCATE THE CHP ENFORCEMENT AREA DOWNSTREAM OF THE SIGNAL STANDARDS.

4. THE LOCATIONS FOR RAMP AND MAINLINE DETECTORS MUST BE REVIEWED BY THE CALTRANS DISTRICT TRAFFIC OPERATIONS BRANCH RESPONSIBLE FOR RAMP METERING. SEE LOOP DETECTOR LAYOUT DETAILED IN CHAPTER 2 OF THE RMDM.

5. SEE CHAPTER 3 OF RMDM FOR SIGNING AND PAVEMENT MARKING DETAILS.

6. SEE HDM INDEX 504.3 FOR RAMP LANE AND SHOULDER WIDTH STANDARDS.

7. DEPENDING ON APPROACH GEOMETRY AND SPEEDS, THE LANE DROP TRANSITION BETWEEN THE LIMIT LINE AND THE 6' SEPARATION POINT SHALL BE ACCOMPLISHED WITH A TAPER OF BETWEEN 30:1 AND 50:1. SEE HDM INDEX 504.3(1)(d).

8. A MINIMUM 1000' LONG AUXILIARY LANE SHOULD BE PROVIDED BEYOND THE RAMP CONVERGENCE POINT WHEN TRUCK DEMAND IS 5% OR GREATER ON ASCENDING ENTRANCE RAMPS TO FREEWAYS WITH SUSTAINED UPGRADES EXCEEDING 3%. SEE HDM INDEX 504.3(2)(b) FOR DESIGN CRITERIA ON AUXILIARY LANES.

9. DESIGN QUEUE STORAGE LENGTH BASED ON 7% OF PEAK HOUR VOLUME. SEE CHAPTER 1 OF RMDM.

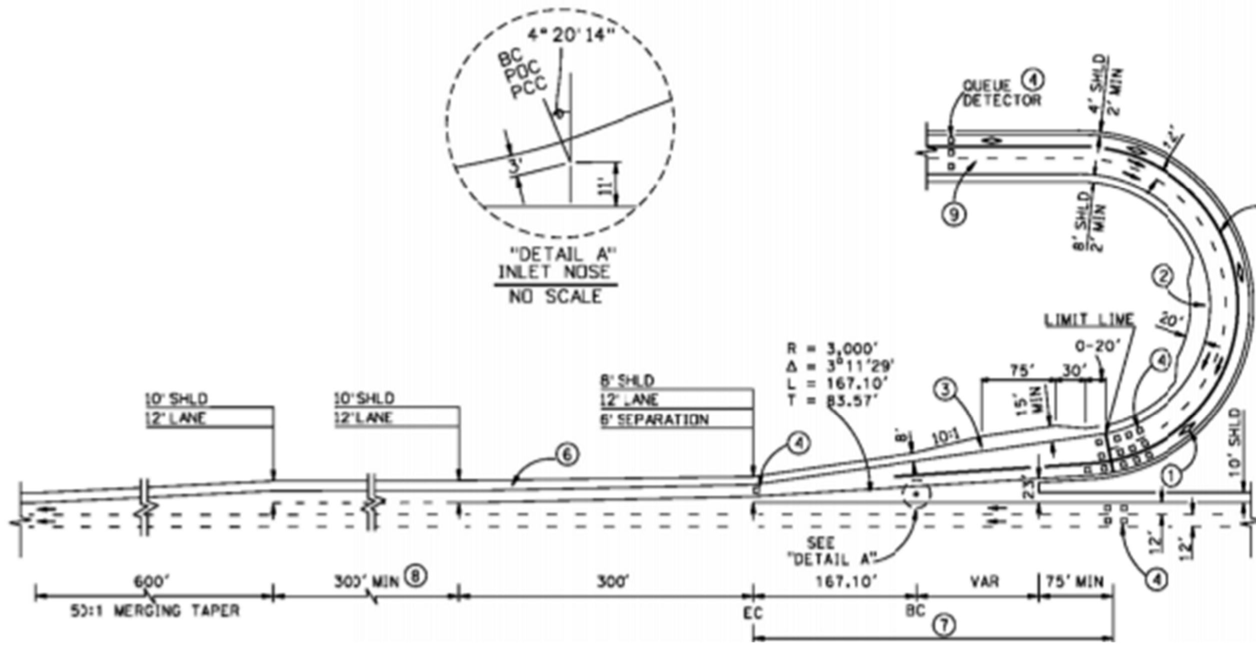


Figure D-12. Typical 3-lane metered loop on-ramp (2 GP lanes+1 HOV lane)
Source: Figure 1-4 of the Caltrans Ramp Metering Design Manual, 2016

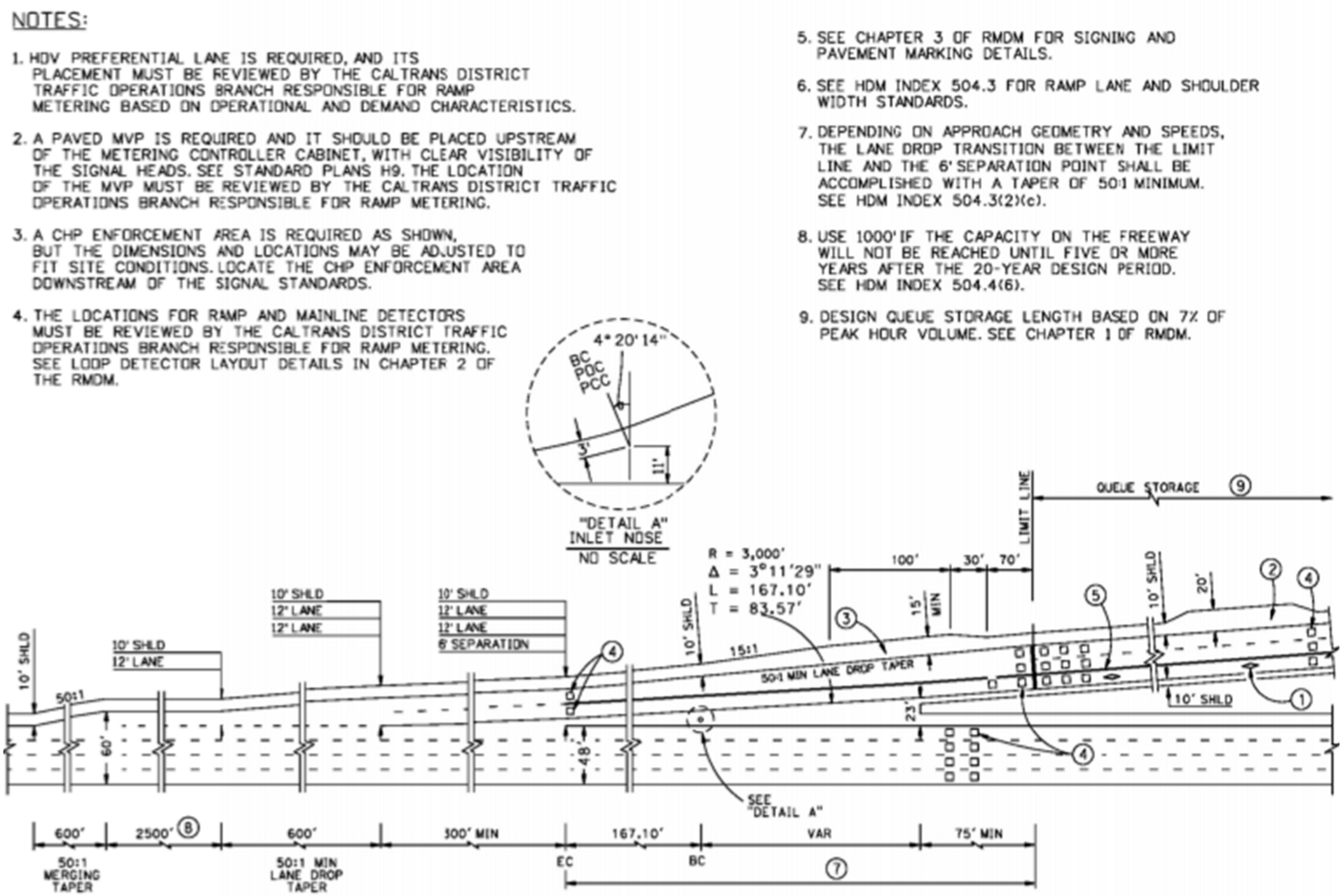


Figure D-13. Typical 3-lane metered connector (2 GP lanes+1 HOV lane)
Source: Figure 1-5 of the Caltrans Ramp Metering Design Manual, 2016

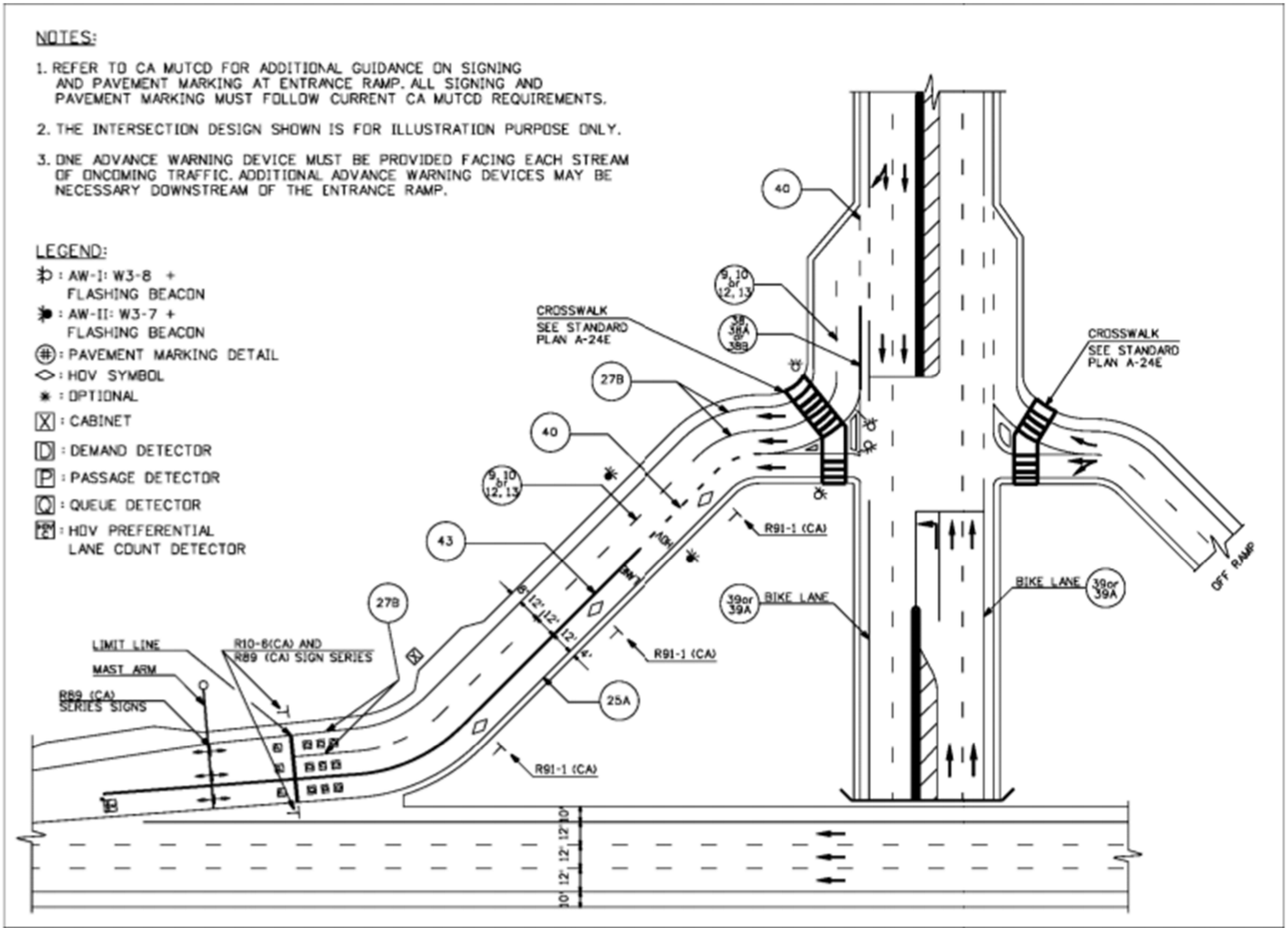


Figure D-14. Typical signing/pavement marking for 3-lane metered on-ramp (2 GP lanes+1 HOV lane)
Source: Figure 3-3 of the Caltrans Ramp Metering Design Manual, 2016

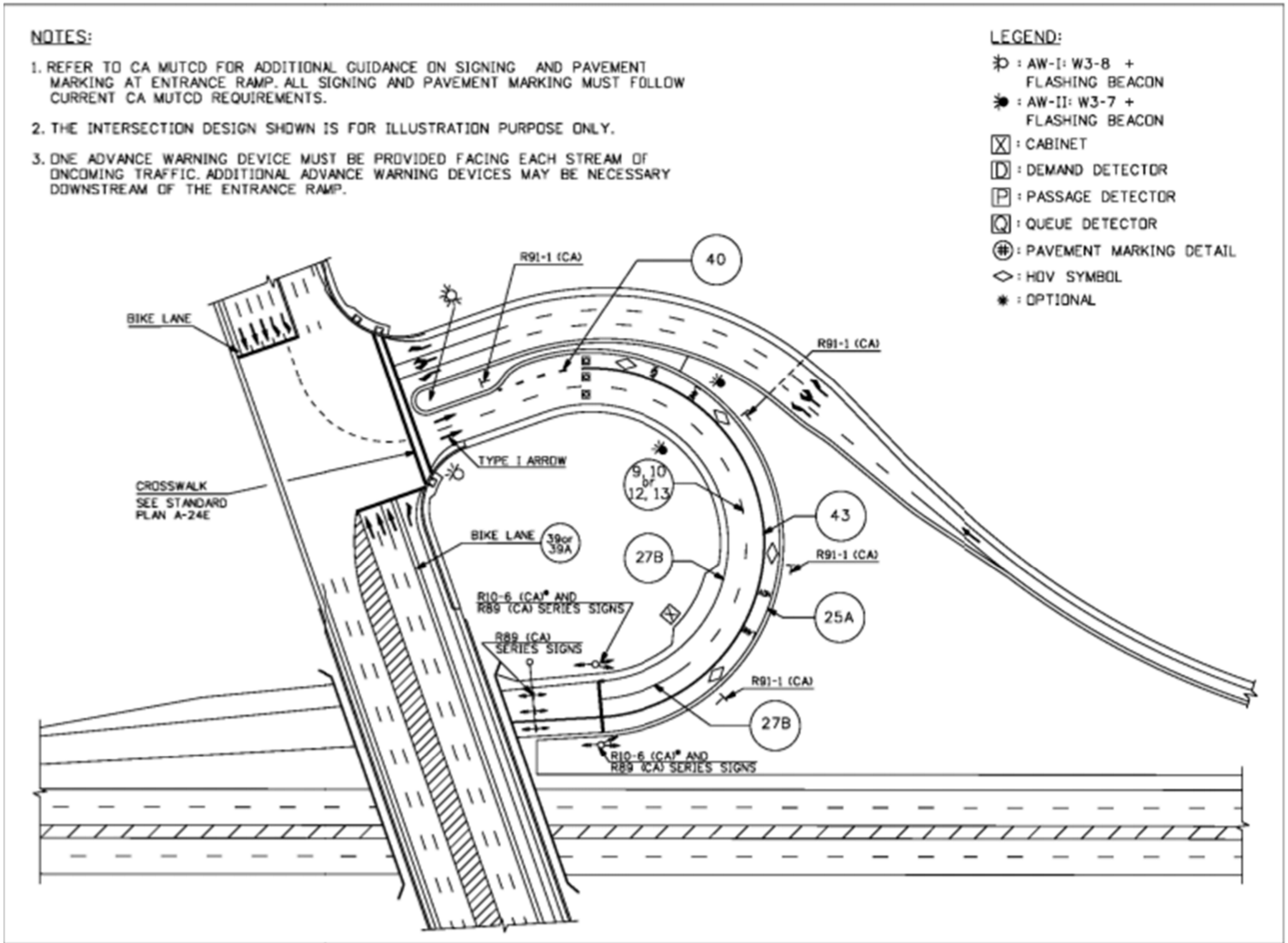


Figure D-15. Typical signing/pavement marking for 3-lane metered loop on-ramp (2 GP lanes+1 HOV lane)
Source: Figure 3-4 of the Caltrans Ramp Metering Design Manual, 2016

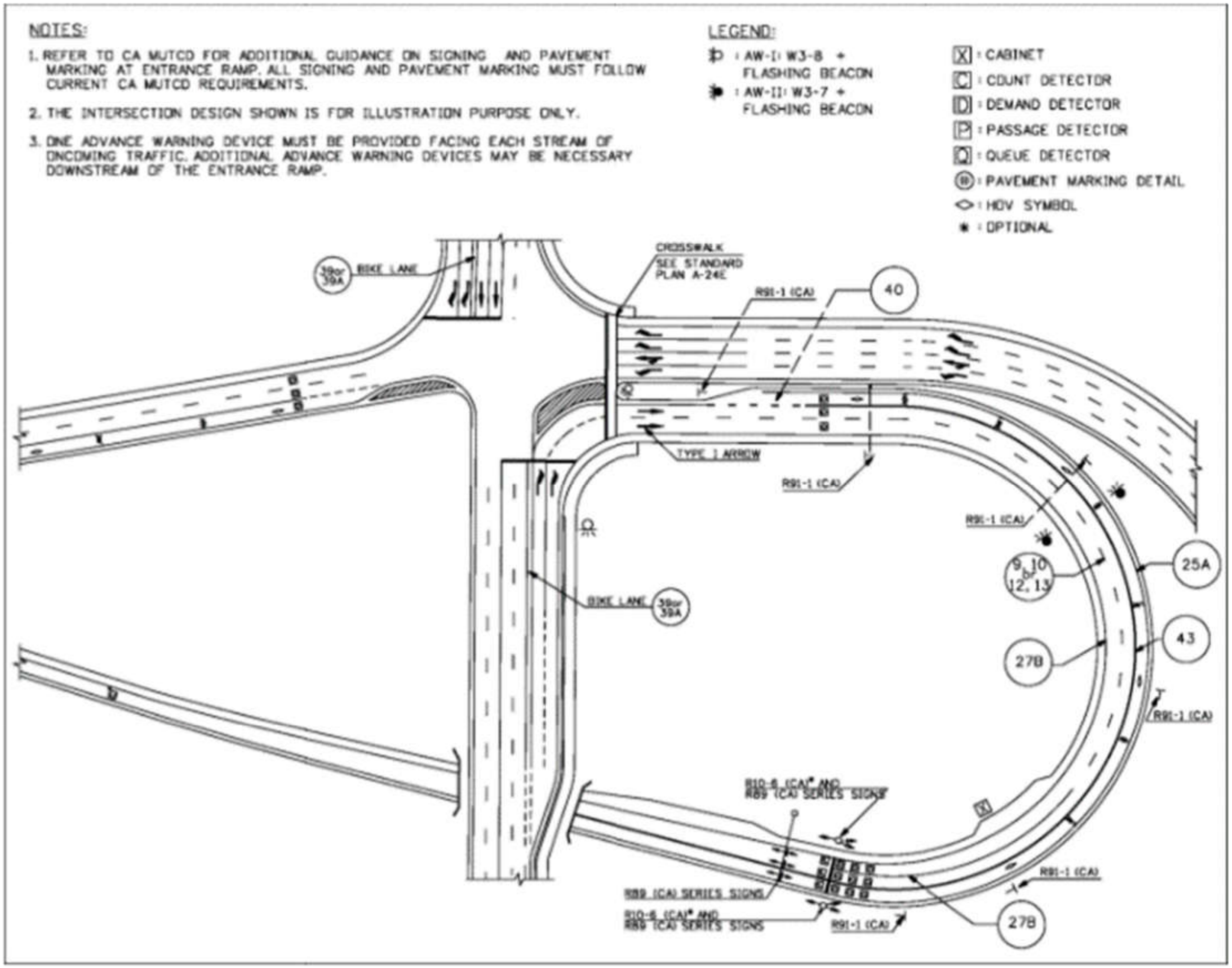


Figure D-16. Typical signing/pavement marking for 3-lane metered loop on-ramp (2 GP lanes+1 HOV lane)
Source: Figure 3-5 of the Caltrans Ramp Metering Design Manual, 2016

Caltrans uses an R89-1 (CA) sign for simultaneous release operations and an R89-2 (CA) sign for staggered release operations of multilane metered on-ramps (Figure D-17).

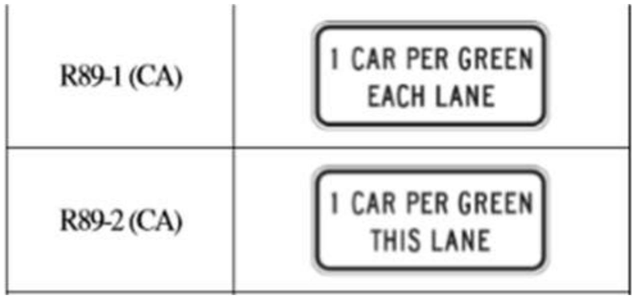


Figure D-17. California regulatory signs
Source: Table 3-1 of Caltrans Ramp Metering Design Manual

Nevada

NDOT warrants installation of 3-lane ramp meters at locations where the peak-hour ramp demand volume is 1500-1600 veh/h. The ramp-metering strategy for 3-lane metered on-ramps is one-vehicle per green with cycle length of 6-6.5 sec.

Utah

UDOT warrants installation of 3-lane ramp metering (3 SOV) at locations where the peak-hour demand volume is 1350-1720 veh/h, or 3 SOV+1 HOV if the percentage of HOVs on the ramp also exceeds 10% of the ramp peak-hour volume.

Wisconsin

WisDOT warrants installation of 3-lane ramp meters (2 SOV+1 HOV) at locations where the peak-hour design-year volume exceeds 720 veh/h and percentage of HOVs on the ramp exceeds 9% of the ramp peak-hour volume. Three-lane metered loop ramps are not recommended. All lanes of multilane metered ramps shall taper into one, with a minimum 30:1 taper ratio, before merging into the freeway.

For 3-lane metered on-ramps (median separated), a R10-6 (L or R) sign is fastened to a side-mounted signal. For 3-lane metered on-ramps (non-median separated), a R10-6 (L or R) sign is placed on a mast arm signal used for two lanes. R10-10 (L, C, or R) (MOD) signs are used for better operation of multilane meters. W4-2 (L or R) signs are placed 75 ft-100 ft downstream of the stop bar of multilane meters depending on the location of signs, signals, and beginning of taper.

Figure D-18 and Figure D-19 are samples of plan sheets from the WisDOT ITS Design Manual (2000) for configuring on-ramps for multilane metering.

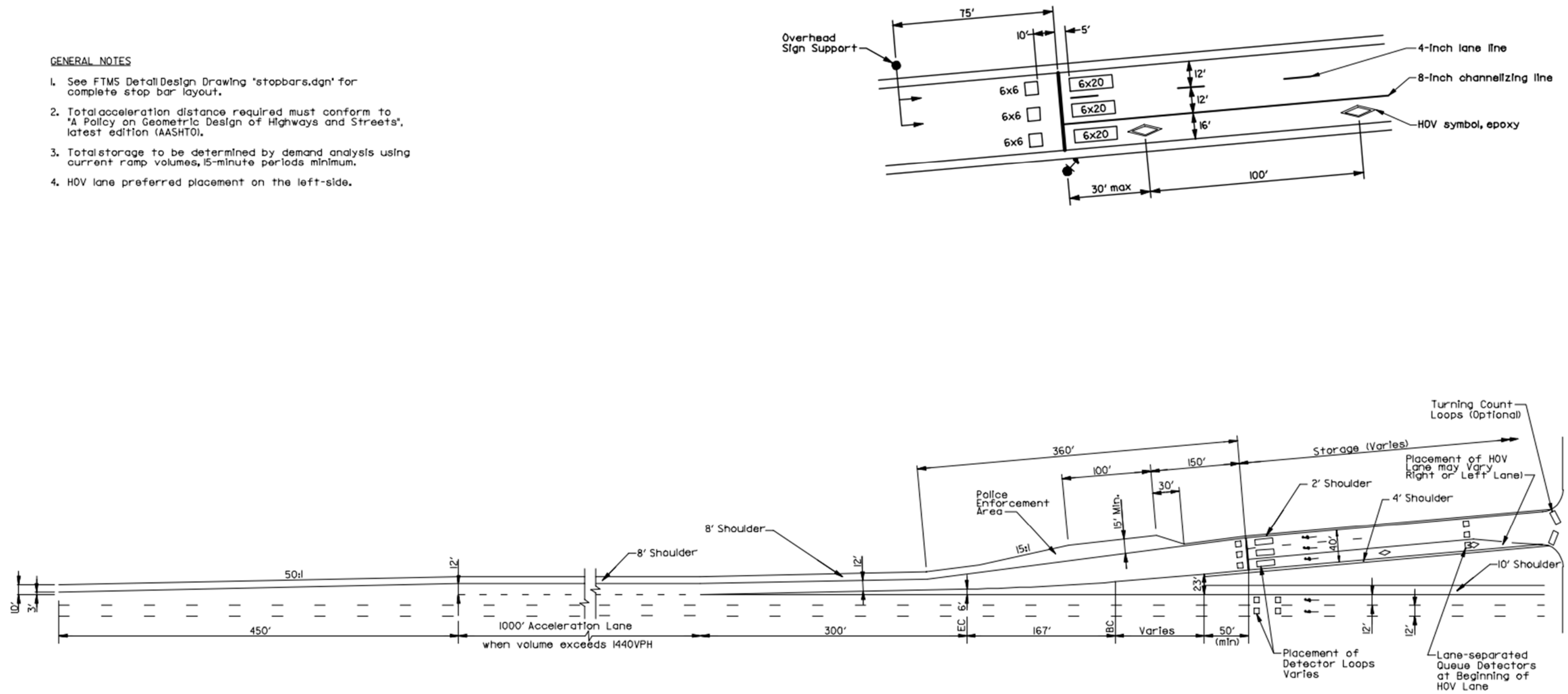


Figure D-18. Typical 3-lane metered slip on-ramp (non-separated HOV)
Source: Figure 3-7 of the WisDOT ITS Design Manual, 2000

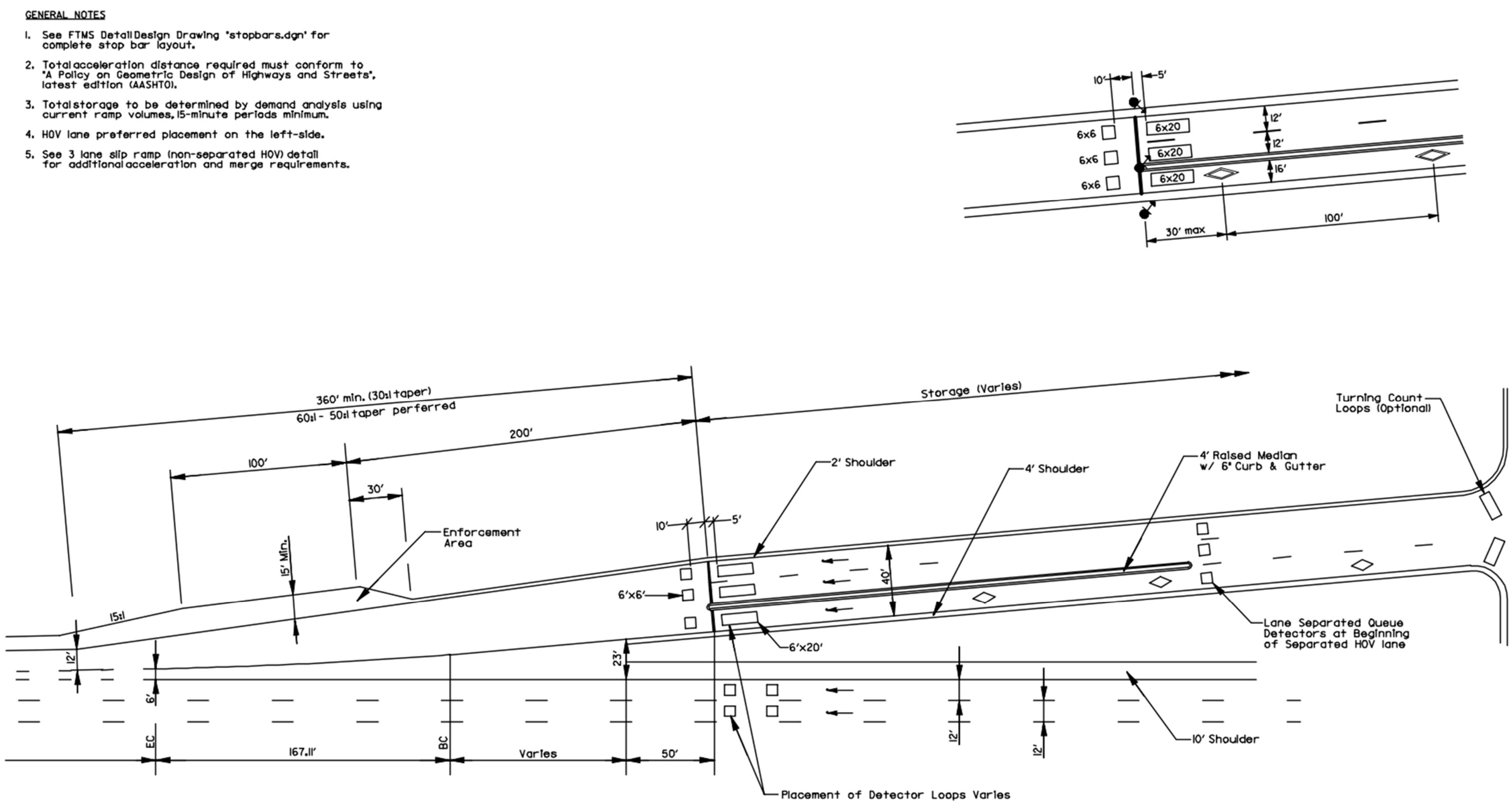


Figure D-19. Typical 3-lane metered slip on-ramp (separated HOV)
Source: Figure 3-8 of the WisDOT ITS Design Manual, 2000

Research Articles

According to a study on the state-of-the-art ramp-metering practices in California, Georgia, Minnesota, Nevada, Texas, Utah, Washington, and Wisconsin (Wang and Dang, 2012), signals are generally installed overhead on mast arms for 3-lane on-ramps. Side-mounted signals may be installed in addition to overhead-mounted meters for better visibility. For 3-lane metered on-ramps with staggered operation, one signal face shall be mounted over each separately-controlled lane (MUTCD requirement) (Figure D-20 and Figure D-21). In California, overhead-mounted signals are placed at least 70 ft downstream of the ramp-meter stop bar to be visible for both approaching and stopped vehicles.

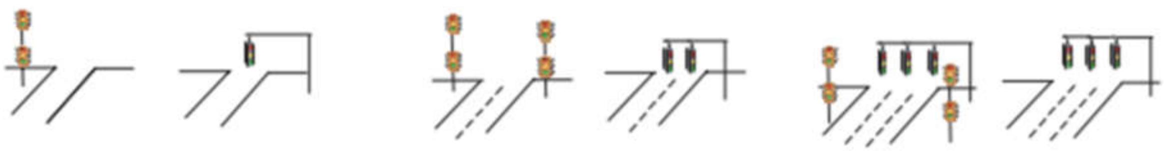
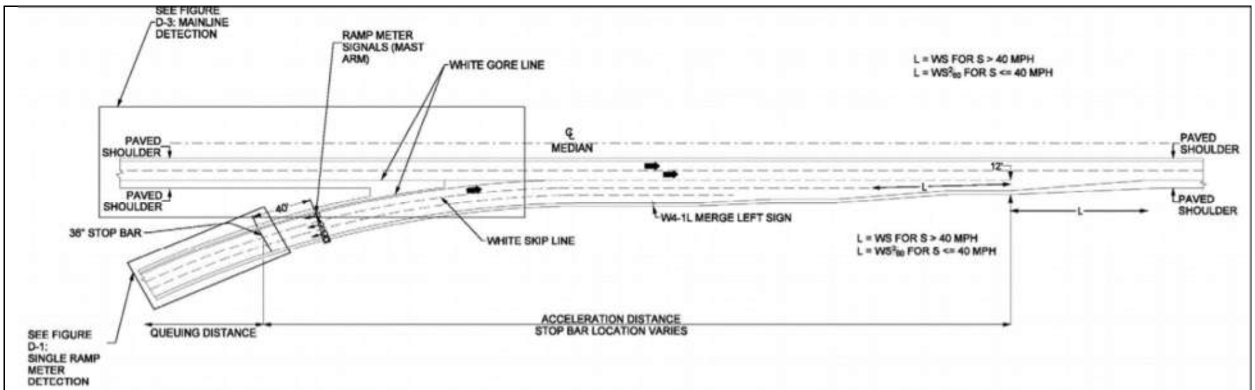


Figure D-20. Signal placement for 1-, 2-, and 3-lane metered on-ramps in California
Source: Wang and Dang, 2012



Notes:

1. When truck volumes exceed 5% on 3% or greater ascending grades, provide 500’ of auxiliary lane between the ramp meter stop bar and point where the ramp and mainline edges of pavement are 10’ apart.
2. See AASHTO Exhibit 3-1 for acceleration distances.
3. Signal heads shall be mast arm mounted as site conditions dictate for three or more lanes.
4. Install W4-1L merge left sign, in accordance with Table 2C-54 or the MUTCD.
5. See Figure C-3 for signalization and equipment.
6. See Figure D-1 for ramp meter detection.
7. See Figure D-3 for mainline detection.
8. See Figure E-1 and E-3 for ramp meter signing and pavement markings.

Figure D-21. Signal placement for 3-lane metered on-ramp
Source: Ramp Metering Feasibility Study for Cabarrus, Gaston, Iredell and Mecklenburg Counties, 2017

Figure D-22 shows locations of overhead- and side-mounted signal heads for a typical loop on-ramp (measurements are specified in Table D-2 for a 3-lane metered loop on-ramp) that satisfy the required stopping sight distance.

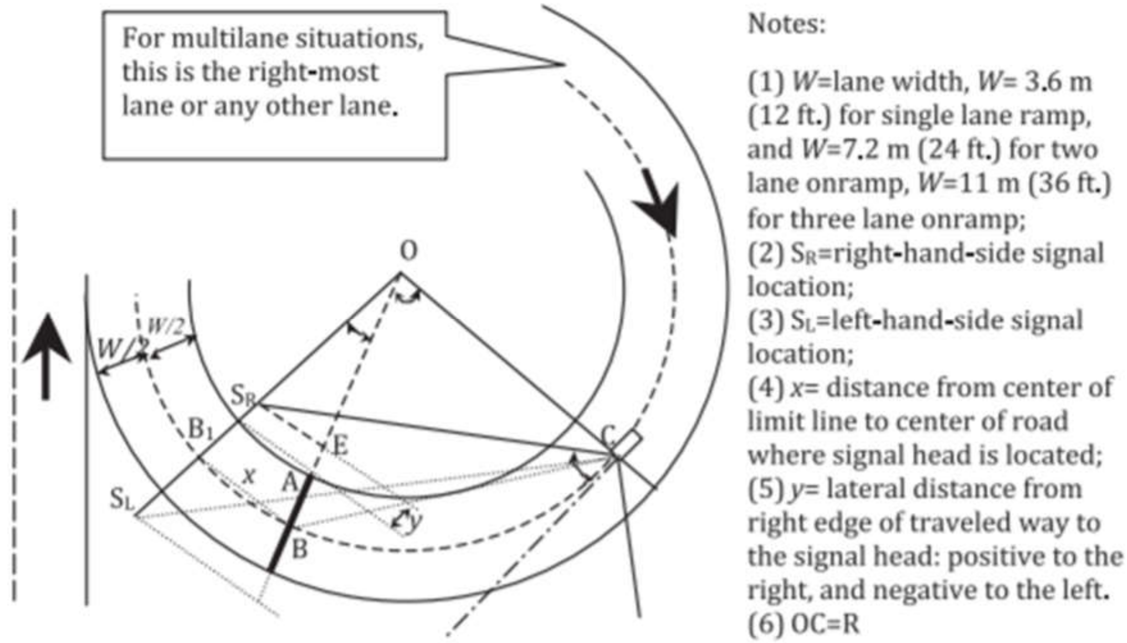


Figure D-22. Locations of overhead- and side-mounted signal heads
Source: Wang and Dang, 2012

Since overhead-mounted signals need a larger radius to satisfy the required stopping sight distance at loop on-ramps (Table D-2), addition of side-mounted signals, particularly left-hand-side mounted signals, may be helpful.

Table D-2. Comparison of overhead- and side-mounted signal heads for 3-lane loop on-ramps

Signal location	**x (m)**	**y (m)**	**R (m)**
Overhead-mounted	21.34	9.14*	123.4
Right-hand-side mounted (inner curve)	0.61	2.44**	187.5
Left-hand-side mounted (outer curve)	0.61	12.19*	15.2

* Lateral distance from left edge of traveled way to the signal head.
** Lateral distance from right edge of traveled way to the signal head.
Source: Wang and Dang, 2012