Traffic Bottlenecks: Identification and Solutions

PUBLICATION NO. FHWA-HRT-16-064

NOVEMBER 2016

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

FOREWORD

The far-reaching detrimental impacts on mobility, the environment, and the economy caused by recurring congestion are well known. This report adds to the vast amount of research on congestion mitigation by developing a new approach for ranking traffic bottlenecks, introducing a new playbook of 70 bottleneck mitigation strategies, conducting a benefit-cost analysis of 5 low-cost bottleneck mitigation strategies, and introducing 3 new bottleneck mitigation strategies. This report describes a data-driven congestion and bottleneck identification software tool and details low-cost operations-focused solutions, including dynamic lane use, contraflow or reversible lane use, hard shoulder lane use, lane width reduction, and modest extension of auxiliary lanes. This report will be of interest to practitioners involved in the transportation operations discipline.

Brian Cronin
Director, Office of Operations
Research and Development

TECHNICAL DOCUMENTATION PAGE

1. Report No. FHWA-HRT-16-064	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Traffic Bottlenecks: Identification and Solutions		5. Report Date November 2016
		6. Performing Organization Code
7. Author(s) David Hale, Ramanujan Jagannathan, Michalis Xyntarakis, Peng Su, Ximiao Jiang, Jiaqi Ma, Jia Hu, and Cory Krause		8. Performing Organization Report No.
9. Performing Organization Name and Address Leidos, Inc. 11251 Roger Bacon Drive Reston, VA 20190		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. DTFH61-12-D-00020
12. Sponsoring Agency Name and Address Office of Operations Research and Development Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296		13. Type of Report and Period Covered Research Report
		14. Sponsoring Agency Code HRDO-20
15. Supplementary Notes The Contracting Officer's Representative was Joe Bared, HRDO-20.		

16. Abstract

The objective of this project was to develop practical methods for prioritizing and mitigating traffic bottlenecks, which are one of the top causes of surface transportation congestion in the United States. Through this project, the following were developed: a new approach for ranking traffic bottlenecks, a new playbook of 70 bottleneck mitigation strategies, a benefit-cost (B/C) analysis of 5 low-cost bottleneck mitigation strategies, and 3 new bottleneck mitigation strategies.

Regarding the new approach for ranking traffic bottlenecks, a data-driven congestion and bottleneck identification software tool was created with numerous performance measures. In parallel, extensive traffic simulations were conducted to assess the operational benefits of underrated strategies as opposed to popular strategies, like ramp metering, which have been extensively researched and implemented in recent decades. Moreover, the project focused on low-cost solutions as opposed to solutions requiring excessive infrastructure investments or advanced vehicle technologies. These solutions involved dynamic lane use, contraflow or reversible lane use, hard shoulder lane use, lane width reduction, and modest extension of auxiliary lanes. Research results demonstrated that these solutions produced favorable B/C ratios with only minor modifications to existing infrastructure. The project further developed preliminary design guidance on signing, signalization, and striping for these strategies, with a follow-on human factors study for two of the strategies.

17. Key Words Research, Traffic bottlenecks, Dynamic lane use, Traffic simulation		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161. http://www.ntis.gov	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 179	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized.

SI* (MODERN METRIC) 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)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AIIR	*Alternative Intersection and Interchange Report*
ARM	annual reliability matrix
ATM	active traffic management
AVD	average vehicle delay
B/C	benefit-cost
BII	Bottleneck Intensity Index
C&S	Colts Neck Road and Sunrise Valley Drive
CBI	Congestion and Bottleneck Identification
CLT	contraflow left turn
CMS	changeable message sign
DDI	diverging diamond interchange
D.I.V.E.	duration, intensity, variability, and extent
DJC	dynamic junction control
DLG	dynamic lane grouping
DLT	displaced left-turn intersection
DLTI	displaced left-turn interchange
DMC	dynamic merge control
DMS	dynamic message sign
DRLT	dynamic reversible left turn
EB	eastbound
FHWA	Federal Highway Administration
FI	fatal and injury
GEH	Geoffrey E. Havers
GPS	Global Positioning System
HCM	*Highway Capacity Manual*
HOT	high-occupancy toll
HOV	high-occupancy vehicle
HSR	hard shoulder running
ITS	intelligent transportation system
LOS	level of service

LT	left turn
MUT	median U-turn intersection
NB	northbound
NHI	National Highway Institute
O&M	operations and maintenance
PCE	passenger car equivalent
PDO	property damage only
PeMS	performance measurement system
RCUT	restricted crossing U-turn intersection
R&S	Reston Parkway and Sunrise Valley Drive
RITIS	Regional Integrated Transportation Information System
RRMSE	relative root mean square error
RT	right turn
SB	southbound
SHRP2	Second Strategic Highway Research Program
STM	spatio-temporal matrix
STOL	Saxton Transportation Operations Laboratory
TCL	clearance time
TFHRC	Turner-Fairbank Highway Research Center
TH	through
TMC	traffic message channel
TTI	Texas A&M Transportation Institute
USDOT	U.S. Department of Transportation
v/c	volume/capacity
VDOT	Virginia Department of Transportation
v/l	volume/lane
VSL	variable speed limit
WB	westbound
WisDOT	Wisconsin Department of Transportation

EXECUTIVE SUMMARY

It is usually accepted that unnecessary traffic delays and vehicle emissions produce adverse impacts on quality of life. The Texas A&M Transportation Institute's (TTI) *TTI's 2012 Urban Mobility Report* further summarizes the ways in which traffic problems are tied to the Nation's economy.(1) Given these impacts on mobility, the environment, and the economy, decades of research have been performed on congestion and bottleneck mitigation.(2–4) However, this research is becoming outdated. New technologies and new ideas are expanding the array of possible solutions. Root causes of congestion and bottlenecks, which have been identified by prior research, have been found to be oversimplified.(5) As a result, the traditional approaches must be updated, and new solutions must be found to solve these bigger problems.

To solve the U.S. traffic congestion problem, mitigation or elimination of bottlenecks is believed to be a top priority. The 2004 report, *Traffic Congestion and Reliability: Linking Solutions to Problems*, indicates that bottlenecks are a major cause of traffic congestion.(5) For example, figure 1 implies that bottlenecks are the single largest source of traffic congestion. Another reason for prioritizing mitigation or elimination of bottlenecks is the exponentially increasing delay experienced by drivers affected by bottlenecks.

Figure 1. Graph. Traffic congestion causes from 2004.(5)

Figure 2 illustrates the simplified version of this well-known relationship, which is documented in the 2010 *Highway Capacity Manual* (HCM).(6) With traffic congestion, there is typically a tipping point at which smooth vehicle flow breaks down, and it transitions into stop-and-go conditions. Subsequently, these stop-and-go conditions often form a bottleneck, severely degrading mobility throughout the affected area. Any successful efforts to prevent bottlenecks from forming, minimize the duration of unavoidable bottlenecks, or minimize the intensity of unavoidable bottlenecks would bring much greater benefits than congestion reductions in areas where bottlenecks do not occur. Indeed, a 5 percent reduction in congestion on the left side of the congestion curve shown in figure 2 would bring only modest reductions in vehicle delay, whereas a 5 percent reduction in congestion on the right side of the congestion curve (i.e., typical bottleneck conditions) would bring much steeper reductions in vehicle delay.

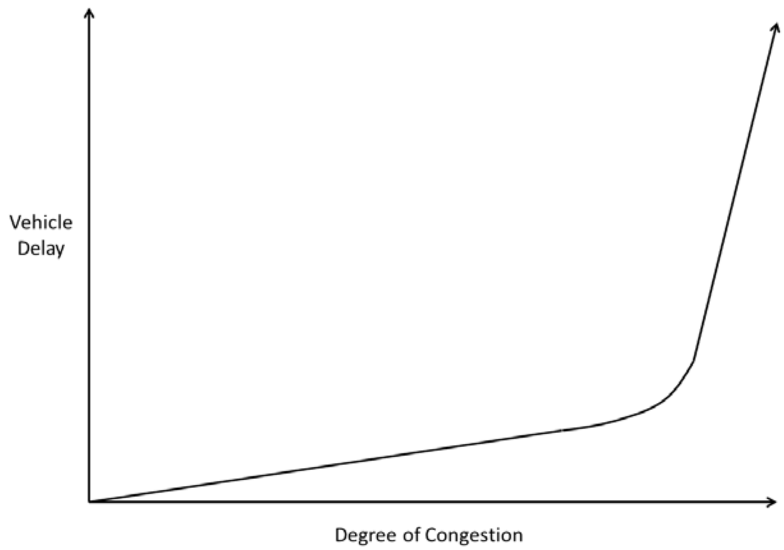


Figure 2. Graph. Vehicle delay versus degree of traffic congestion.

Given that bottleneck mitigation is a top priority, the question becomes how can bottleneck mitigation be achieved or improved. The bottleneck mitigation process can be divided into three subprocesses: identification, diagnosis, and solutions. In order to be mitigated, the bottleneck must first be identified and diagnosed.

Although traffic engineers and typical drivers often have a good sense of when and where bottlenecks tend to occur in their area, significant room for improvement exists in the ability to precisely quantify and prioritize these bottlenecks. This process is known as "bottleneck identification." To justify investments towards improved traffic operations, engineers and policymakers need scientific and accurate methods of bottleneck identification. However, status quo methods are limited and/or outdated. Peak-hour analyses are becoming outdated as a sole source of bottleneck identification because they fail to account for changing conditions throughout the year. There has been a movement toward reliability modeling, which attempts to capture these annual effects. But due to significant input data and calibration requirements, the reliability models suffer from practicality issues. Additionally, there have been recent improvements in data-driven intelligent transportation system (ITS) technologies, which identify bottlenecks in real time. However, there is room for improvement in the robustness of performance measures derived from these technologies. Finally, some engineers have compared and ranked bottlenecks on the basis of experience and judgment. Despite their cost effectiveness, judgment-based qualitative assessments lack credibility unless backed by quantitative results. This report discusses project-specific software development, which produced innovative performance measures for bottleneck identification. This report also includes the insights that can be gained through the use of these performance measures. Figure 3 illustrates the Congestion and Bottleneck Identification (CBI) software tool developed as part of this project. It is hoped that the new performance measures will be adopted by States and/or commercial products for a new level of robustness in bottleneck identification.

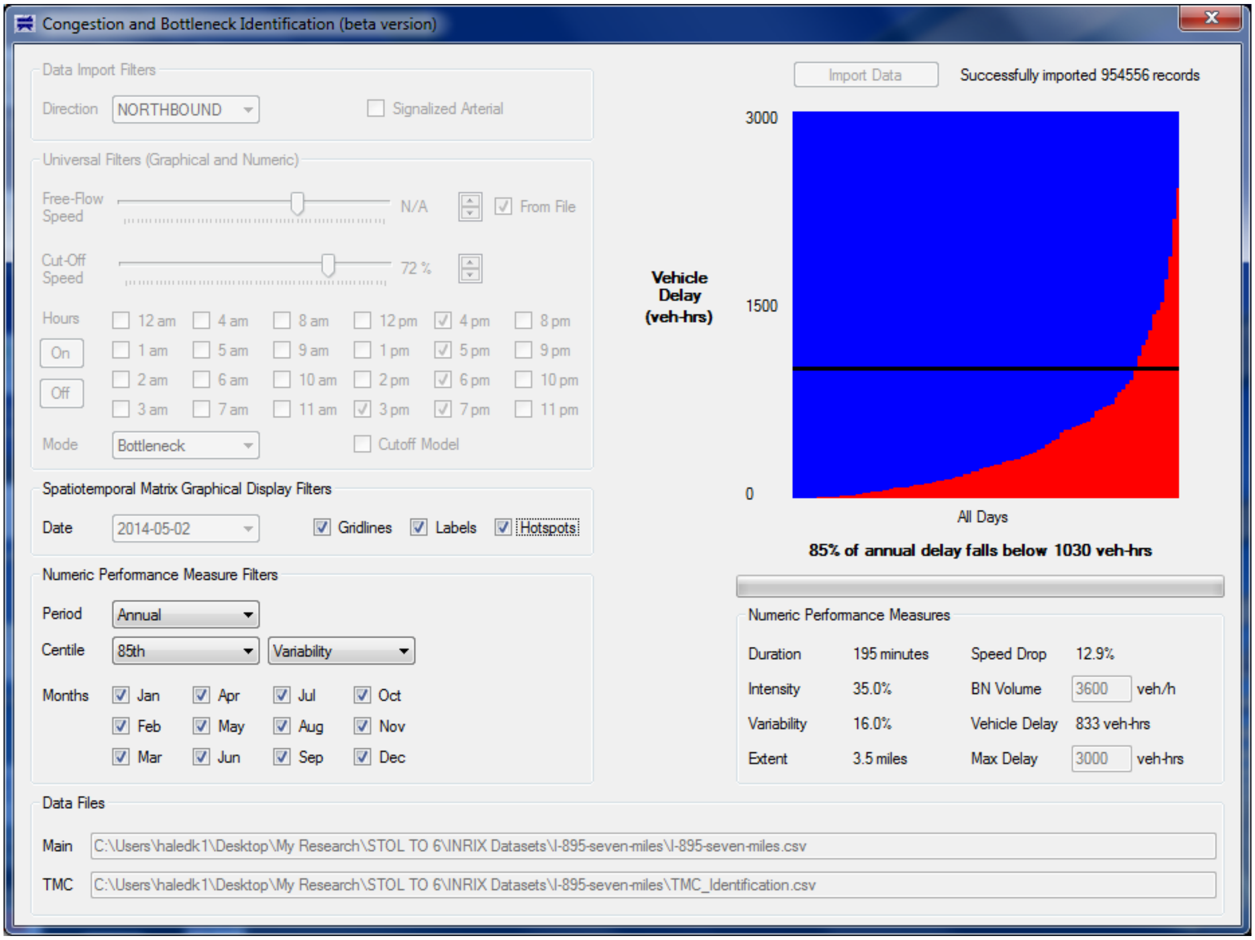


Figure 3. Screenshot. CBI software tool.

Once the bottleneck locations are precisely identified, the question becomes how to prioritize them for subsequent mitigation. The bottleneck identification process provides an initial indicator of the likely prioritization through performance measures that reveal their operational impacts. However the bottleneck diagnoses and solutions also affect prioritization because they shed light on the true benefit-cost (B/C) ratio. This report does not delve deeply into bottleneck diagnosis issues. Instead, it provides a comprehensive set of solutions, which should be analyzed by engineers to determine whether or not they are compatible with their bottlenecks.

Bottleneck solutions in this report are divided into three categories. First, there is a playbook of 70 bottleneck solutions divided into 7 categories. Second, there is a set of low-cost bottleneck solutions that currently have real-world implementations but are likely underdeployed in the United States. Third, there is a set of low-cost innovative solutions that are not yet deployed in the United States. The report focuses on potentially underrated strategies as opposed to popular strategies, like ramp metering, which have been extensively researched and implemented in recent decades. Moreover, this report focuses on low-cost solutions as opposed to solutions requiring excessive infrastructure investments or advanced vehicle technologies. While it is certainly hoped that connected and automated vehicles will produce significant congestion relief in the upcoming years, the magnitude and/or timeliness of this relief cannot be taken for granted, which motivates the pursuit of alternative solutions. This report focuses on solutions involving dynamic lane use, contraflow or reversible lane use (e.g., dynamic reversible left-turn (DRLT) lanes at signalized diamond interchanges, as illustrated in figure 4 and figure 5), hard shoulder lane use, lane width reduction, and modest extension of auxiliary lanes. These solutions produce significant operational benefits with only minor modifications to existing infrastructure. This report also provides design guidance on signing, signalization, and striping for these strategies, with a follow-on human factors study for two of the strategies. Finally, this report provides microsimulation and a B/C analysis for the strategies.

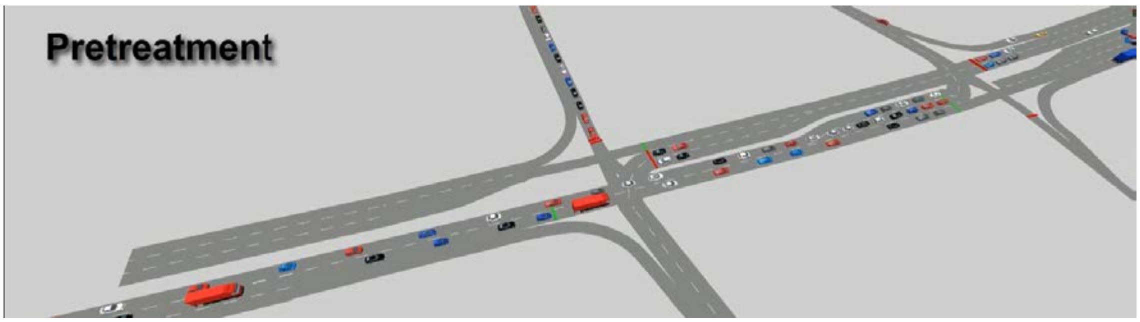


Figure 4. Screenshot. Conventional lanes at a signalized diamond interchange.

Figure 5. Screenshot. DRLT lanes at a signalized diamond interchange.

In summary, this report presents the results of a significant research project devoted to bottlenecks. It provides a modernized view of U.S. bottleneck mitigation by (1) introducing new methods of robust bottleneck identification, (2) describing new bottleneck mitigation strategies, and (3) presenting new research on the operational effectiveness and B/C ratios of these methods. The new methods of bottleneck identification are made possible by recent developments in high-resolution data collection technologies. Promising new mitigation strategies were identified and studied by a research team familiar with traffic operations state of the practice. Operational effectiveness of the bottleneck mitigation strategies was primarily assessed through simulation studies, and B/C ratios were estimated for five specific strategies.

CHAPTER 1. INTRODUCTION

It is evident from currently available literature that no standard definition of congestion is used consistently across the industry; one must differentiate between the terms “congestion” and “bottleneck.” The Federal Highway Administration (FHWA) addresses this directly by stating this tenet: “A bottleneck may cause congestion, but congestion is not always the result of a bottleneck.”(7) Moreover, supporting the contention that a bottleneck is a subordinate component of congestion (and not vice versa), FHWA has long-opined that bottlenecks comprise a significant portion of the total congestion causes (see figure 1).

Bottlenecks, as the name implies, denote an area of significant breakdown in flow. The question then becomes how significant must this breakdown be to warrant the bottleneck classification as opposed to mere congestion. Although this question is addressed by the HCM and by traffic analysis tools, ideally, this breakdown definition could be customized for local needs when appropriate, such that the guidance sources would simply advocate for specific default definitions.(6) Subsequently, once the bottleneck locations are identified and prioritized according to agreed-upon metrics, an effective array of bottleneck mitigation strategies should be considered. This report focuses on practical, low-cost methods of bottleneck identification and mitigation.

Bottlenecks historically have been categorized as either recurring (i.e., predictable and routine, as exemplified by that which occurs during commute peak hours) or nonrecurring (i.e., due to random events like incidents, weather, special events, and work zones). Recurring bottlenecks exist at locations that FHWA terms “operationally deficient” but only with the caveat that said locations reveal themselves only when they become overburdened by (typically) peak hour demand; the same location operates fundamentally sound otherwise.(8) Relief comes chiefly when the overburden falls away. Nonrecurring bottlenecks are random as to location, cause, and duration and typically are only relieved when the event disperses. Another key difference is that the recurring locations may have the opportunity to be corrected by redesign, whereas nonrecurring locations have less design-influenced solutions and more event-response opportunities to improve the condition.

There is a deficiency of cost-effective solutions for identifying and remedying oversaturated locations. To address this deficiency, this report is organized into the following four chapters:

- Chapter 2. Bottleneck Identification.
- Chapter 3. Bottleneck Mitigation Concepts.
- Chapter 4. Cost-Effective Solutions for Bottleneck Mitigation.
- Chapter 5. Innovative Solutions for Bottleneck Mitigation.

In addition, the appendix provides details of all bottleneck playbook solutions.

CHAPTER 2. BOTTLENECK IDENTIFICATION

PROBLEM STATEMENT

Precise bottleneck identification is one of the best ways that traffic engineers can demonstrate the need for, and the benefits of, investing in transportation improvements. However, conventional methods are limited and/or outdated, and improved methods are needed to make it more cost-effective, precise, and scientific. Regarding the conventional methods, they all have certain drawbacks. In the 1990s, most traffic engineers were modeling the peak 15-min period using the peak hour factor. However this paradigm did not take into account the significant factors that come into play throughout the year, such as seasonal demand volume fluctuation, weather, and incidents. Currently, improved procedures are available for analyzing the entire year; however, they are extremely data-intensive, which makes them difficult to use. Improved data sources such as INRIX® are available; however, some INRIX®-based bottleneck rankings have focused on traffic intensity without accounting for variability, reliability, or throughput.[(9)] This report describes some recent improvements in bottleneck identification, which leverage the latest advances in both research and ITS technology.

DEFINITION OF CONGESTION

According to *TTI's 2012 Urban Mobility Report*, in 2011, congestion in the top 85 U.S. urban areas caused 5.5 billion h of travel delay and 2.9 billion gal of excessive fuel consumption, totaling $121 billion in costs to the public.[(1)] This figure would be substantially higher if it accounted for the significant cost of environmental impacts associated with idle-related auto emissions and high gasoline prices.[(1)] The concept of congestion deals with the relationship between the quality and quantity of use of the transportation system. Congestion results when vehicles in a given road system exceed capacity, regardless of the size of the road system, which includes small (i.e., localized), medium (i.e., by facility or neighborhood), or large (i.e., regional). Congestion should be considered in two dimensions: spatial and temporal, including where (i.e., location, such as an intersection, roadway segment, or transit route) and when (i.e., time of day or year). Predicting either the where or when (or ideally, both) can be a huge first step in combating congestion, as agencies can mobilize appropriate dynamic responses to shortcut the delay. In the widest context, congestion results when traffic demand approaches or exceeds the available capacity of the system. However, traffic demand fluctuates significantly across seasons, days of the week, and time of day. Similarly, capacity changes dynamically based on the degree of weather (e.g., ponding, snow drifts, wind debris, etc.), degree of work zone interference, degree of traffic incident severity, and other nonrecurring events.

Although the cost of congestion to society is high[1], the precise definition of congestion is still vague, which muddies how the costs are calculated. Academia, engineers, and lay persons have different perceptions and expectations of how the system should perform based on whether they are in rural or urban areas, peak/off-peak periods, or prior experiences on that road. For example, the three prior groups might use such terms as volume/capacity (v/c), vehicle-hours, or buffer time needed, respectively. To capture the heterogeneity between people/agencies in defining

[1]Some costs of congestion that are measured are emissions, vehicle idling (fuel lost), time lost, increased shipping costs passed through, etc.

congestion, Bertini conducted an extensive survey to identify the different definitions of metropolitan traffic congestion.[10] The survey included the following four qualitative questions:

1. How do you define congestion in metropolitan areas?
2. How is congestion in metropolitan areas measured?
3. How accurate or reliable are traffic congestion measurements?
4. How has metropolitan traffic congestion been changing over the past two decades?

Figure 6 summarizes the responses for more than 500 participants from different professional backgrounds.[10] Respondents were provided an opportunity to comment on congestion in general to establish the future research guidelines. One of the survey comments noted that "If we want to reduce congestion we need to be able to define it and quantify it."(pg. 4)[10] Some people defined congestion as "anything below the posted speed limit" or below some "speed threshold (e.g., < 35 mi/h)."(pg. 4)[10] Others noted that "congestion is relative," "a perception," and "I know it when I see it."(pg. 4)[10] Therefore, in addition to addressing the question of how significant a breakdown must be to warrant bottleneck classification, there are also other questions about how to assess congestion at all levels. This report focuses on methods of bottleneck identification and mitigation as opposed to methods of identifying and mitigating congestion at all levels.

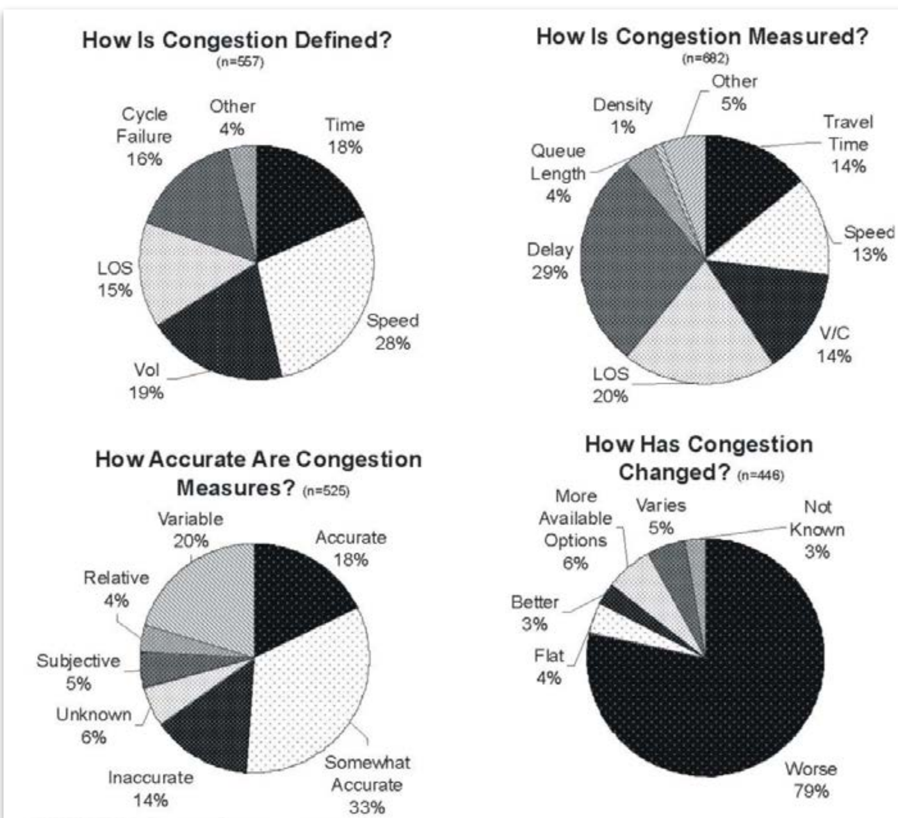


Figure 6. Graph. Survey results for defining congestion.[10]

The following four major dimensions or attributes can be used to quantify bottlenecks:

- **Duration**: The amount of time that breakdown conditions persist before returning to an uncongested state.
- **Intensity**: The relative severity of breakdown that affects travel, such as v/c ratios, that relate the different levels of congestion experienced on roadways.
- **Variability**: The changes in breakdown conditions that occur on different days or at different times of day.
- **Extent**: The number of system users or components (e.g., roads, bus lines, etc.) that are affected by the breakdown.

This report uses the acronym D.I.V.E. to reference these attributes.

CLASSIFICATION OF BOTTLENECKS

Skabardonis et al. summarized several approaches to define and measure both recurrent and nonrecurrent bottlenecks.(11) They reported that recurrent bottlenecks are caused by demand fluctuations, roadway operation strategies, and physical geometries. Luo reported that recurrent bottlenecks are predictable from excessive peak-hour demands, while the 2010 HCM notes that recurrent demands follow day-of-week and month-of-year patterns.(12,6) Nonrecurrent bottlenecks are said to be caused by random and unpredictable events, including accidents and bad weather. Cambridge Systematics aggregated actions to relieve both recurrent and nonrecurrent bottlenecks, as shown in table 1.(5) The report indicates that both forms of congestion can be quantified by travel time, throughput, and queue characteristics.

Table 1. Selected FHWA bottleneck relief sources.

Mitigation Strategy	Action	Example Resources
Mitigating nonrecurring bottlenecks	Traffic incident management	See references 13–17
	Work zone management	See references 18 and 19
	Road weather management	See references 20 and 21 and "Fundamentals of Road Weather Management" training course sponsored by FHWA
	Special events traffic management	See reference 22 and "Managing Travel for Planned Special Events" training course sponsored by FHWA and the National Highway Institute (NHI)
Mitigating recurring bottlenecks	Freeway management	See references 23, 24, and "Freeway Management & Operations" training course sponsored by NHI
	Arterial management	See reference 25 and "Access Management, Location and Design" training course sponsored by FHWA and NHI
	Corridor traffic management	See references 26 and 27 and HOV Facilities training course sponsored by FHWA
	Travel demand management	See reference 28

SPATIO-TEMPORAL MATRIX (STM)

Data-driven methods and models now make it possible to pursue a more precise quantification of bottlenecks. Big data are a hot topic in not just the transportation industry but in other industries as well. STM can be considered a step in the direction of big data compared to traditional traffic engineering analyses. STM facilitates monthly and annual analyses instead of just peak-hour analyses, providing a more comprehensive picture of what the traffic problems are. STM makes it possible to put a more precise price tag on transportation investments. As stated previously, bottlenecks can now be quantified according to D.I.V.E. STM is a fundamental prerequisite to generating these measures for congestion identification. Various types of measured data (e.g., from INRIX®, loop detectors, performance measurement system (PeMS), TomTom™, HERE™, etc.) could be used to generate STMs. Figure 7 illustrates that STMs can also be generated by traffic models, like micro-simulation, or HCM procedures.(6)

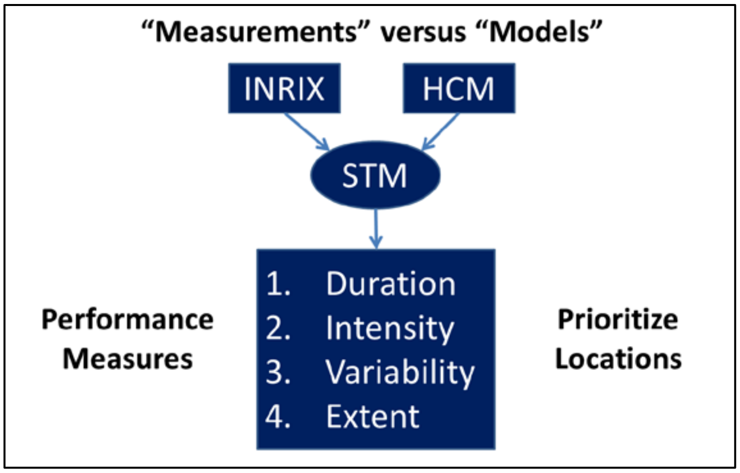


Figure 7. Flowchart. Generation of STMs via measurements or models.

In the case of HCM modeled data, traditional capacity analysis generates results for a single STM cell.(6) However, a recent HCM advancement is to generate STM results throughout the analysis year(s). The first step of the HCM procedure is to define a reliability analysis box. Figure 8 shows that two-dimensional heat charts, illustrating daily traffic performance on a space-time continuum, can be expanded into a third reliability dimension by modeling a significant number of days. Red cells within figure 8 represent congested conditions, yellow cells denote at-capacity conditions, and green cells indicate the uncongested regime. This HCM-generated STM may appear identical to an INRIX®-generated STM at first glance, although fundamental differences in how they were generated (i.e., projected versus measured conditions) should be carefully considered by the analyst. Regardless of whether derived from measurements (like INRIX®) or models (like the HCM), the STM is an important first step towards a more precise identification of congestion and bottlenecks.

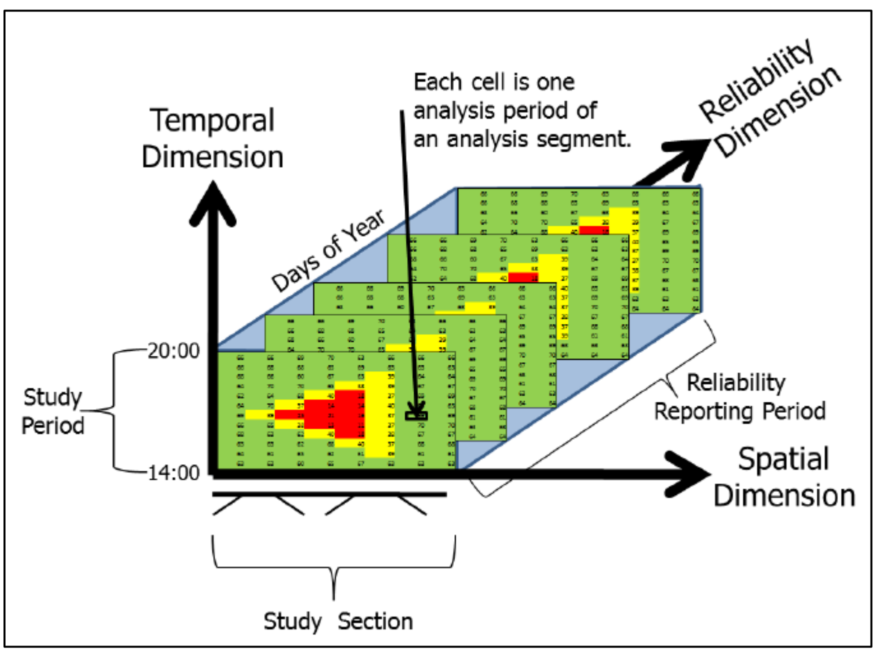


Figure 8. Graph. Reliability analysis box.(6)

CONVENTIONAL METHODS

Automatic identification of traffic congestion is an important first step toward identifying and ranking bottlenecks.(29) However, improved methods of bottleneck identification are needed. The conventional methods are either limited or incomplete. Bottleneck identification needs to be cost-effective, precise, and scientific. By comparing before-and-after scenarios in terms of their bottleneck measurements, precise bottleneck identification is one of the best ways that traffic engineers can demonstrate a solid return on investment in a time of tight budgets. When considering the conventional methods, it is possible to associate each one of them with one or more fundamental disadvantages. These disadvantages can be summarized into the following categories:

- **Peak period modeling**: In the 1990s, peak 15-min modeling was the norm and was recommended by the HCM.(30) However regardless of the time period duration being used, peak period modeling does not take into account significant factors that come into play throughout the year, such as seasonal demand volume fluctuation, weather, and incidents.

- **Annual reliability modeling**: Modeling frameworks are now available for analyzing the entire year and reflecting some of the elements that were not recognized by peak period modeling. The drawbacks of today's reliability models include extensive input data entry requirements, calibration requirements, and computer runtime requirements, which increase their risk for engineers who would use them.

- **Data-driven annual analysis**: In contrast to the 1990s, it is now common for traffic data providers (e.g., INRIX®, HERE™, PeMS, TomTom™, etc.) to deploy technologies that constantly measure traffic conditions in the field. However, some bottleneck rankings observed in the industry are relatively insensitive to traffic volume levels, congestion variability, and travel time reliability. Furthermore, data-driven analyses have typically been constrained to assessing past performance instead of predicting the future.

- **Experience and judgment**: Through extensive personal observation, traffic engineers can develop a sense for specific areas containing the worst traffic problems. However without precise annual measurements, it is more difficult to communicate the duration and extent of traffic problems to decisionmakers. When operational improvements are made, it is more difficult to verify the beneficial impact of these improvements.

DATA-DRIVEN METHODS

This section describes recent improvements in bottleneck identification, which leverage recent advances in transportation research and technology. The research project that produced this report also produced an INRIX®-based software tool for CBI, which displays a typical STM. One of the tool's main performance measures is called "bottleneck intensity." For example, in figure 9, the red area represents vehicles traveling below 45 mi/h, which means the cutoff speed used here is 45 mi/h. Bottleneck intensity can thus be computed as the red area divided by the total area.

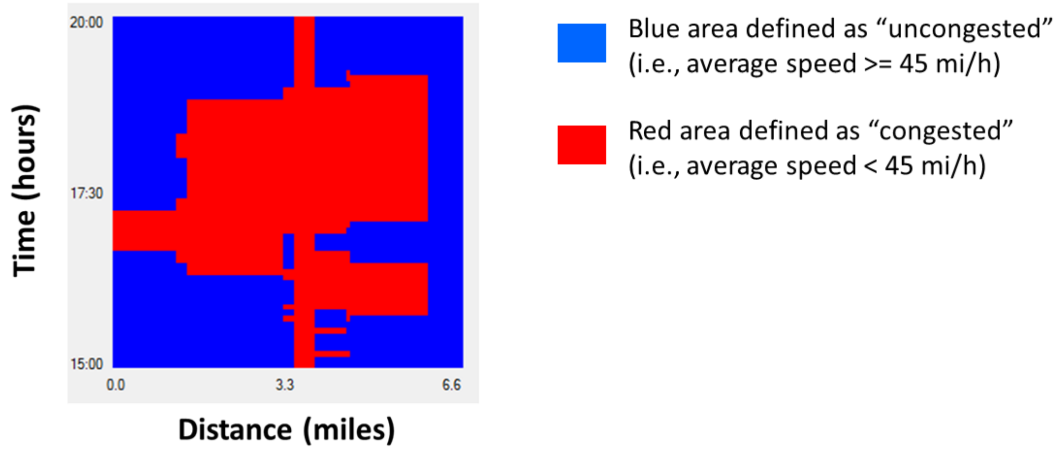


Figure 9. Graph. Concept of bottleneck intensity (percentage of analysis box that is congested).

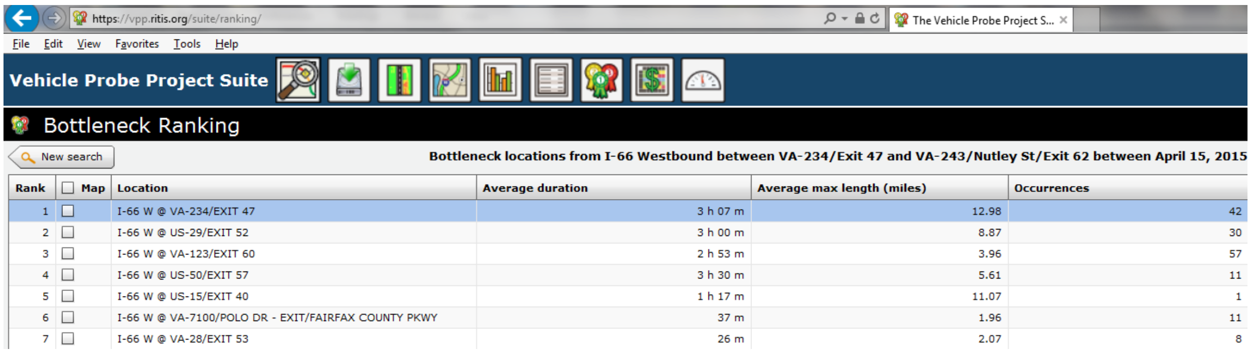
Although average vehicle speed is used in the example in figure 9, other measures (e.g., travel time, percentage of stopped vehicles, and density) could be used to identify congested sections. Obtaining the bottleneck intensity is a good first step in assessing bottlenecks, but it is not sufficient because it does not take into account annual variability or reliability.

The Regional Integrated Transportation Information System (RITIS) Web site has rich functionality for congestion identification.(9) It is capable of generating STMs for a number of different performance measures. It has a slider feature to easily adjust thresholds for different colors in the heat map and a second slider for specifying which hours of the day will be plotted. Figure 10 illustrates a sample RITIS heat map for congestion identification.

Figure 10. Screenshot. Congestion identification heat map from the RITIS Web site.(9)

BOTTLENECK RANKINGS

The RITIS Web site also provides bottleneck rankings (see figure 11) based on an impact factor.(9) The RITIS formula for computing impact factors is shown in figure 12. Assuming consistent thresholds (e.g., 45 mi/h) for differentiating between congested and uncongested conditions, the impact factor computed by this formula is essentially equivalent to the bottleneck intensity from figure 9 aggregated for all days of the analysis period (e.g., 1 year).



Rank	Map	Location	Average duration	Average max length (miles)	Occurrences
1		I-66 W @ VA-234/EXIT 47	3 h 07 m	12.98	42
2		I-66 W @ US-29/EXIT 52	3 h 00 m	8.87	30
3		I-66 W @ VA-123/EXIT 60	2 h 53 m	3.96	57
4		I-66 W @ US-50/EXIT 57	3 h 30 m	5.61	11
5		I-66 W @ US-15/EXIT 40	1 h 17 m	11.07	1
6		I-66 W @ VA-7100/POLO DR - EXIT/FAIRFAX COUNTY PKWY	37 m	1.96	11
7		I-66 W @ VA-28/EXIT 53	26 m	2.07	8

Figure 11. Screenshot. Bottleneck rankings from the RITIS Web site.(9)

$$IF = \sum_{n=1}^{N} (D_n \times L_n)$$

Figure 12. Equation. Impact factors for bottleneck rankings.

Where:

IF = Impact factor for ranking bottlenecks (dimensionless).
N = Number of days in the analysis period (e.g., 365 days).
n = Number of bottleneck occurrences.
D_n = Duration of congestion (minutes) (i.e., continuous vertical red distance in figure 9).
L_n = Length of congestion (miles) (i.e., continuous horizontal red distance in figure 9).

The RITIS impact factor is a reasonable method for comparing and ranking bottlenecks because it simultaneously accounts for the average duration and extent of congestion throughout the year. However, it does not account for the instability of congestion throughout the year. In statistics, instability is often quantified through the variance and/or standard deviation. In reliability modeling, which is a relatively new concept in transportation operations, instability is often quantified through the Travel Time Index. As stated in the Executive Summary, a problematic trend in this country is the decreasing reliability of surface transportation. When the Nation’s transportation facilities become less reliable, “we have to allow increasingly more time to ensure an on-time arrival.”(pg. 1)(31) In order to integrate the reliability concept into the comparison and ranking of traffic bottlenecks, the annual reliability matrix (ARM) was developed.

ARM

The ability to compute bottleneck intensity for every relevant day of the year, as shown in figure 9, is a prerequisite to ARM. All bottleneck intensities are plotted left to right onto a two-dimensional X-Y graph. The y-axis represents the bottleneck intensity value, which ranges between 0 and 100 percent. The x-axis represents all relevant days of the year, with the far left containing the best day of the year and the far right containing the worst day of the year. Once the annual values have been plotted in this manner, it becomes possible to visualize traffic bottlenecks in terms of both intensity and reliability.

Figure 13 and figure 14 compare STM to ARM. Whereas STM displays a bottleneck intensity for one day, ARM can display hundreds of bottleneck intensities throughout the year or across multiple years. By displaying and storing a distribution of bottleneck intensities in this manner, ARM facilitates visualization and computation of useful performance measures. For example, the red area under the curve in figure 14 is essentially equivalent to the impact factor from RITIS. However, the 85th percentile day of the year in the figure can now be visualized fairly easily as having an intensity of approximately 50 percent. The 85th percentile intensity would be a reasonable way to compare and rank bottlenecks because it reflects some of the most congested days of the year. By contrast, the 100th percentile intensity might reflect a fluke accident, and the 50th percentile intensity might not reveal how congested the worst days are.

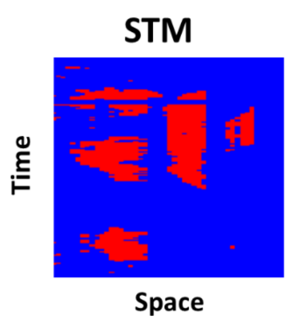


Figure 13. Graph. STM used to compute bottleneck intensity.

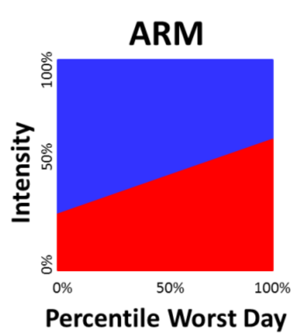


Figure 14. Graph. ARM based on bottleneck intensity.

As stated previously, ARM was developed to integrate reliability into the assessment of traffic bottlenecks. Figure 15 and figure 16 illustrate ARM diagrams for two known bottlenecks. Each ARM diagram has approximately the same amount of red area. According to the RITIS bottleneck ranking formula, both bottlenecks would produce approximately the same impact factor and would thus be considered approximately equal priority. However, ARM is intended to show that for bottleneck #2 on the right, roughly one-third of its days exhibit more delay than will ever be experienced at bottleneck #1. This means that a driver of bottleneck #2 will need to allow more time to ensure an on-time arrival in comparison to bottleneck #1. Bottleneck #2 is thus less reliable and should be considered a higher priority for mitigation efforts.

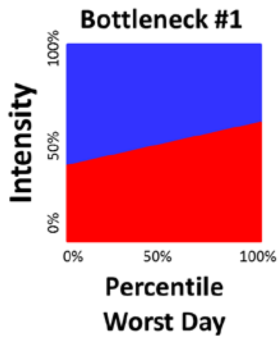


Figure 15. Graph. ARM illustrating relatively good bottleneck reliability.

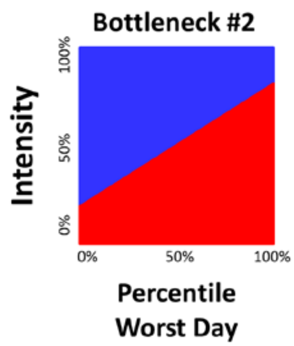


Figure 16. Graph. ARM illustrating relatively poor bottleneck reliability.

The ability to generate both STMs and ARMs was implemented within the CBI software tool. It was believed that the STMs would be valuable for inspecting bottleneck intensity on any given day, whereas ARM would be valuable for assessing annual intensity and reliability. One limitation that was noticed in the ARM concept was the inability to quantify speed drops. For example, if two bottlenecks exhibit a similar ARM shape, it is quite possible that one of the two bottlenecks may in fact be much more serious than the other one because their vehicle speeds in the congested red region might be much lower. The inability to quantify speed drop is apparently one of the fundamental disadvantages of bottleneck intensity as a performance measure. The first step taken to address this disadvantage was to compute daily and annual average speed drops in the CBI tool and to display these speed drop values as numeric performance measures. In this manner, the speed drop values could conceivably be used as a tiebreaker when comparing two bottlenecks with similar ARMs. One of the original versions of the CBI tool is shown in figure 17. Software controls on the left side of the screen are used to toggle between daily STM displays and annual ARM displays.

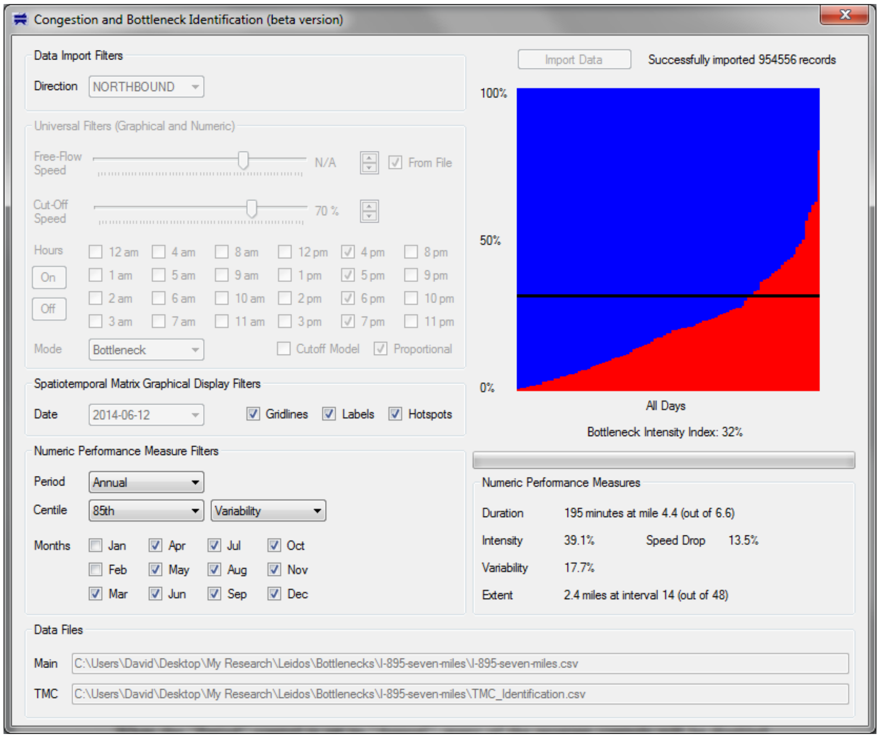


Figure 17. Screenshot. CBI tool with ARM (upper right) and speed drop (lower right).

CALIFORNIA METHOD—VEHICLE-HOURS OF DELAY

A series of workshops were conducted across the United States to share the concepts and information in this report. At a workshop in Oakland, CA, hosted on April 2, 2015, local participants shared a method for evaluating bottlenecks on the basis of vehicle-hours of delay. One of the main advantages of this method is the ability for bottleneck rankings to explicitly reflect traffic volume demands. For example, a four-lane freeway would tend to exhibit twice as much delay as a two-lane freeway and would thus become a greater priority for mitigation than the two-lane freeway. Indeed, both the RITIS impact factor and intensity-based ARM would potentially give similar priority ratings to a four-lane freeway and a two-lane freeway as long as they had similar traffic density levels. This provided an incentive for integrating this "California method" into the CBI tool. Vehicle delay is computed according to the *2013 Most Congested Freeways Report and Methodology*.(32) The equations in figure 18 and figure 19 illustrate California's use of the 35-mi/h cutoff speed. When integrating this method into the CBI tool, instead of assuming a fixed value of 35 mi/h, cutoff speeds were obtained from a flexible set of CBI software features.

$$\textit{Delay on each TMC for one vehicle} = \frac{\textit{Length of TMC}}{\textit{Average TMC Speed}} - \frac{\textit{Length of TMC}}{35}$$

Figure 18. Equation. Delay per vehicle on each traffic message channel (TMC).

$$\textit{Delay on each TMC (vehicle-hours)} = \textit{Delay on each TMC for one vehicle} \times \textit{Bottleneck volume}$$

Figure 19. Equation. Vehicle-hours of delay on each TMC.

The California method also requires specification of a bottleneck volume. According to an Iteris report, bottleneck volumes should be measured immediately downstream of the downstream end of congestion.(32) The CBI tool was upgraded to allow users to enter a measured or estimated bottleneck volume. In addition, the software was upgraded to display vehicle-hours of delay as a numeric performance measure. Vehicle delay is thus believed to be an important performance measure for comparing and ranking bottlenecks because it explicitly captures the effects of both speed drops and demand volumes. In contrast to the previous intensity-based ARMs in figure 15 through figure 17, figure 20 through figure 22 illustrate the same ARMs based on vehicle-hours of delay.

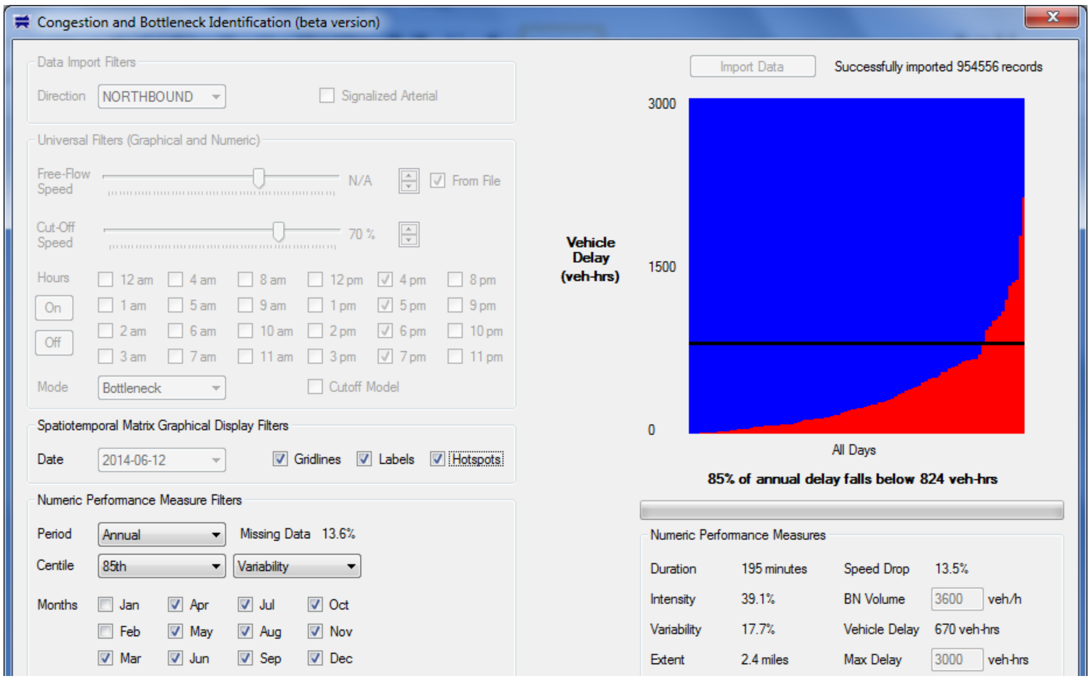


Figure 20. Screenshot. Newer CBI software tool featuring vehicle-hours of delay.

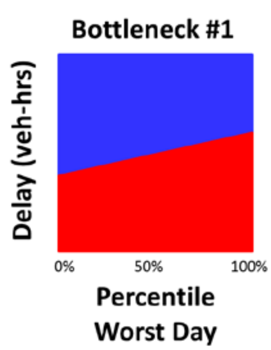


Figure 21. Graph. ARM illustrating relatively good bottleneck reliability based on vehicle-hours of delay.

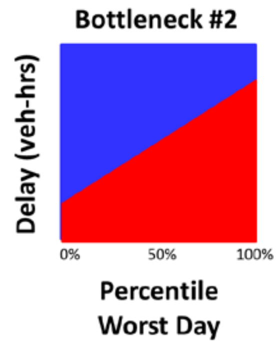


Figure 22. Graph. ARM illustrating relatively poor bottleneck reliability based on vehicle-hours of delay.

BOTTLENECK INTENSITY INDEX (BII)

Although the shape and size of delay-based ARMs are useful for simultaneously visualizing bottleneck intensity and variability, respectively, it was believed that a numerical performance measure was needed to quantify the ARM into a single number because matrices and graphs allow room for human interpretation or might be confusing to some engineers. By contrast, a single number could conceivably avoid any errors in interpretation. In other words, the highest congestion number could be much more easily interpreted as the worst number. In response, an ARM-based performance measure called the BII was implemented within the CBI tool. This value represents a delay level below which 85 percent of the ARM's red area exists. The BII is an attempt to capture the size and shape of the ARM into a single number. As shown in figure 20, the checkbox labeled "Hotspots" can be used to superimpose the BII as a horizontal black line across the ARM. If the "Hotspots" checkbox is turned off, this horizontal line will be hidden. Also in figure 20, the BII is reported as 824 vehicle-h because 85 percent of the annual red area falls below the 824-vehicle-h mark. Other numeric performance measures on the screen are daily values as opposed to annual values.

The 85th percentile BII appears to be more effective than 85th percentile delay for comparing bottlenecks. This is because the BII computation explicitly takes a summation of delay values throughout the year, whereas the percentile delay simply needs to be larger than a portion of other days' delays. This relationship of 85th percentile BII to 85th percentile delay is similar to the relationship of mean to median. The 85th percentile BII explicitly reflects 85 percent of the red area. The 85th percentile delay simply says that 85 percent of the days had a lower delay.

The 85th percentile BII appears to be more effective than the 100th percentile BII because the 100th percentile BII only reveals which bottleneck experiences the highest-delay day of the year, which may have been caused by a fluke accident or event. The 85th percentile BII appears to be more effective than the 50th percentile BII because the 50th percentile BII focuses on 50 percent of the best days of the year. The 85th percentile level provides a reasonable number for focusing on some of the worst days of the year without overemphasizing a small number of absolute worst days. Because of this, the BII is always an 85th percentile value regardless of what is selected in the CBI tool's centile control. By contrast, the standard D.I.V.E. performance measures all use the chosen percentile from the centile control.

Finally, the CBI tool encourages intelligent specification of cutoff speeds, which can be sensitive to a number of factors. Popular reliability performance measures, like the Travel Time Index, assume inflexible comparisons to the free-flow speed, but the perception of congested conditions depends on more than the free-flow speed. As pointed out by Elhenawy et al., drivers affected by severe weather and/or limited visibility conditions may not consider themselves delayed at speeds well below the posted limit.[(29)] In some areas, factors like lane width, lateral clearance, and pavement quality may not be properly reflected in the posted speed limit. In some areas, cutoff speeds should vary significantly on different segments within the same corridor. The important point is that engineering judgment must be used in properly defining cutoff speeds on a case-by-case basis. Without this judgment, a roadway severely impacted by weather could mistakenly be judged as having the same congestion as a corridor unaffected by weather.

Once the cutoff speeds have been properly chosen for each corridor, a fair comparison of known bottleneck locations can take place. As a prerequisite to a fair comparison, a consistent analysis box should be chosen for each bottleneck in terms of both time and distance. For example, each bottleneck could be analyzed for the same months of the year, days of the week, hours of the day, and interval duration. Most testing of the CBI tool was performed under 5-min interval durations. Larger interval durations of 15 or 30 min could be used, but this might compromise accuracy of the results. Smaller interval durations of 1 min could be used, but this might cause the software to become slow in performance. Once a consistent analysis box and interval duration have been chosen, the direct comparisons can occur. Figure 23 through figure 25 illustrate a comparison of three real-world bottleneck locations along three different freeway corridors. In this example, the I-695 bottleneck exhibits the worst score, with a BII of 3,840 vehicle-h.

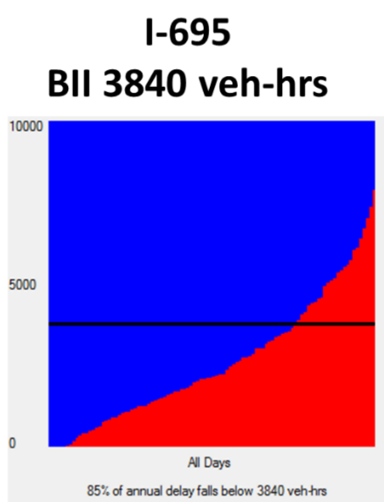


Figure 23. Graph. Comparison of bottleneck locations based on BII—I-695 example.

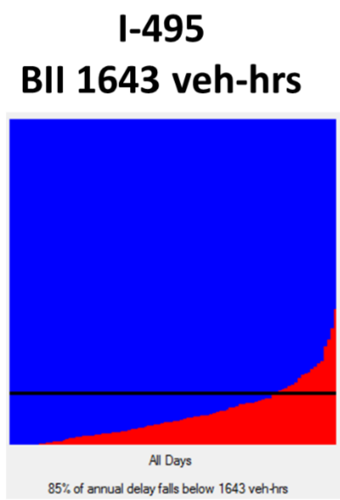


Figure 24. Graph. Comparison of bottleneck locations based on BII—I-495 example.

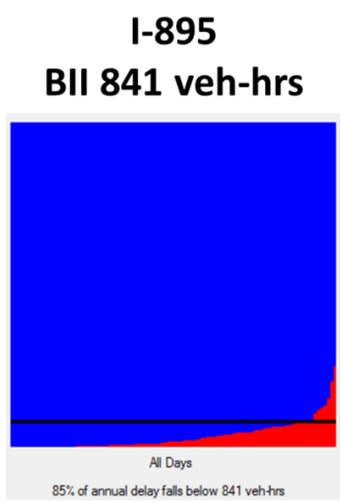


Figure 25. Graph. Comparison of bottleneck locations based on BII—I-895 example.

If mitigation strategies are going to be implemented, before-and-after use of the CBI tool could help to carefully quantify the improvements. It is also conceivable that the tool could compare multiple bottlenecks along the same freeway corridor to see which part of the corridor should be mitigated first or to determine whether upstream mitigation strategies would activate hidden bottlenecks downstream. However, the tool has not yet been tested for such analyses.

SIGNALIZED ARTERIALS

It is also possible to evaluate surface arterial corridors in this manner. Some researchers have expressed concern over the application of this type of congestion identification along signalized arterials. To be sure, the software tools can easily apply the same models in exactly the same way on both arterials and freeways. However, along signalized arterials, spotty patterns in the STM can be caused by the stop-and-go action at intersections. These spotty patterns make it more difficult to identify contiguous bottleneck locations caused by dense traffic. Another concern is that speeds and travel times might be harder to measure on signalized arterials. Regarding the ITS technologies used to measure traffic flows and speeds on surface arterials, there is a lack of standards and uniformity among various cities and States. This means that vehicle speeds will be measured more accurately in some cities and less accurately in others. Speed measurement technology will hopefully improve over time. In response to the concern about identifying arterial bottlenecks within the STM, FHWA researchers recommend the wavelet concept for filtering out portions of delay that appear to be unrelated to congestion.(33) In the CBI tool, clicking on the "Signalized Arterial" checkbox automatically invokes the wavelet method, which adjusts INRIX® speed readings during the import process.

Use of segment-specific free-flow speeds is also recommended for arterials. The free-flow speed slider control can be used to specify one free-flow speed for the entire analysis. Alternatively, it is possible to apply different free-flow speeds for each segment using the "From File" checkbox. When this checkbox is turned on, free-flow speeds from the INRIX® file are applied in the analysis. The "From File" checkbox should only be enabled if the INRIX® files contain reference speed data. When the "Reference Speed" checkbox is turned on, the cutoff speed slider will transform from absolute values into percentages. These percentages will be applied to free-flow speeds from the INRIX® file, which can differ for each segment. Percentage values will be displayed right next to the slider.

Figure 26 and figure 27 illustrate STMs for a signalized arterial before and after wavelet filtering. As shown in the leftmost figure, delays caused by recurring red phases produce a spotty pattern, making it more difficult to identify bottlenecks caused by traffic congestion. As shown in the rightmost figure, the number of false negatives and false positives has been reduced, making it easier to identify bottlenecks. As a result, the wavelet filtering mechanism is desirable for eliminating recurrent and unavoidable signal delays from the bottleneck identification process.

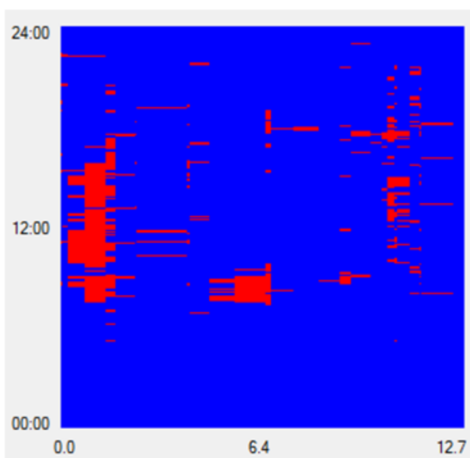


Figure 26. Graph. STM before wavelet filtering.

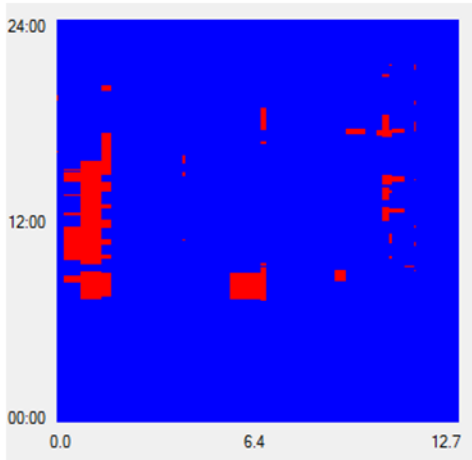


Figure 27. Graph. STM after wavelet filtering.

SUMMARY

The initial version of the CBI tool emphasized annual bottleneck intensities similar to the RITIS Web site. When the ARM concept was implemented, it integrated reliability (variability) into the comparison and ranking of traffic bottlenecks. However, it was noticed that the ARM concept was unable to quantify speed drops, so the overall bottleneck rankings could not be fully trusted. Next, this disadvantage was addressed by computing daily and annual average speed drops in the CBI tool and displaying these speed drop values as numeric performance measures. In this manner, the speed drop values could conceivably be used as a tie breaker when comparing two bottlenecks with similar ARMs. However, at the Oakland, CA, workshop, local participants recommended vehicle-hours of delay to explicitly reflect traffic volume demands, such that four-lane freeways would nominally become doubly higher priorities for mitigation than two-lane freeways. When the California method was integrated into the CBI tool, this allowed bottleneck rankings to explicitly reflect throughput impacts. Finally, along signalized arterials, some portions of delay might be completely unrelated to congestion. In response, the wavelet concept was incorporated for filtering out unavoidable portions of delay. Table 2 summarizes the evolution of the CBI tool in terms of the new elements that can now affect bottleneck rankings. These elements are operational in nature and do not reflect safety issues that could further affect bottleneck prioritization.

Table 2. New concepts affecting bottleneck rankings.

Type of Analysis	Before	After
Intensity	X	X
Variability		X
Speed drop		X
Throughput		X
Wavelet		X

Note: Blank cells indicate that this concept was not used to determine bottleneck rankings.

These methods provide a new level of robustness in identifying congestion and bottlenecks, both spatially and temporally. They also provide an important first step toward prioritizing problem areas and selecting countermeasures. Unlike subjective assessments of congestion that rely on experience and judgment, these automated methods offer scientific justifications for transportation investments. Unlike prior research, these methods reflect the impacts of important elements (e.g., weather, visibility, etc.), are more transferable to various regions, and are better for identifying important bottleneck characteristics. Once the bottleneck locations have been precisely identified and prioritized, an assessment of mitigation strategies can then begin.

CBI TOOL USE TIPS

The CBI tool has its own unpublished internal user's guide, which explains each feature in detail. Some notable recommendations for proper use of the CBI tool include the following:

- **Suggested cutoff speeds**: A model developed by Elhenawy et al. automatically computes cutoff speeds as a function of free-flow speed, weather, and visibility.[(29)] However, the model can also be applied under conditions of clear weather and ideal visibility. In this case, the model-recommended cutoff speed is generally 28 percent lower than the free-flow speed. For example, a free-flow speed of 62 mi/h produces a cutoff speed of 45 mi/h. For analyses where vehicle speeds are consistently and significantly affected by weather, visibility, or any factor ignored by posted speed limits, lower cutoff speeds should be used.

- **Unbiased comparisons**: When comparing and ranking bottlenecks, scope-defining parameters should be consistently applied to all bottleneck locations to achieve an unbiased analysis. These analysis parameters could include excluded days of the week, excluded months of the year, and resolution (e.g., 5 min) of the INRIX® data.

- **Bad sectors of data**: Some INRIX® datasets may contain corrupted or missing data. In these cases, the final performance measures might be less accurate. For example, the engineer could check for bad sectors of data by temporarily setting the cutoff speed equal to 75 mi/h and then performing a preliminary scan of STM diagrams throughout the year. When the cutoff speed is 75 mi/h, the analysis cube should be completely red. If large chunks of the STM are blue, this implies missing data, and such days should be excluded from the analysis. As illustrated previously in figure 20, a missing data diagnostic value was also added to the tool. This automatically reports the percentage of missing INRIX® data within the daily or annual analysis. In cases where the percentage of missing data is relatively high (e.g., greater than 10 percent), the analyst should be skeptical of any performance measures reported by the tool.

Figure 28 illustrates an analysis where all dates between January 1 and February 17 exhibited large chunks of blue in the STM, even when the cutoff speed was set to 75 mi/h. Thus, January and February were excluded from the annual analysis results, as shown by checkboxes in the lower left corner of the screen. The Second Strategic Highway Research Project (SHRP2) report *Establishing Monitoring Programs for Travel Time Reliability* recommends the use of imputation (i.e., the process by which voids in the data are filled by estimation based on data from nearby or similar detectors) to fill voids caused by missing data, but the CBI tool does not currently contain automated imputation methods.(34) This discussion of bad data sectors illustrates the importance of proper analysis practices, which can inform good decisions. It is not up to the software tools to make important decisions. Software is sometimes misused, and results are sometimes misinterpreted. There are usually more factors to consider than a software tool can incorporate. Moreover, the tool only analyzes operational performance. To effectively prioritize bottlenecks and select countermeasures, decisionmakers must consider additional factors (e.g., safety, impacts, costs, etc.).

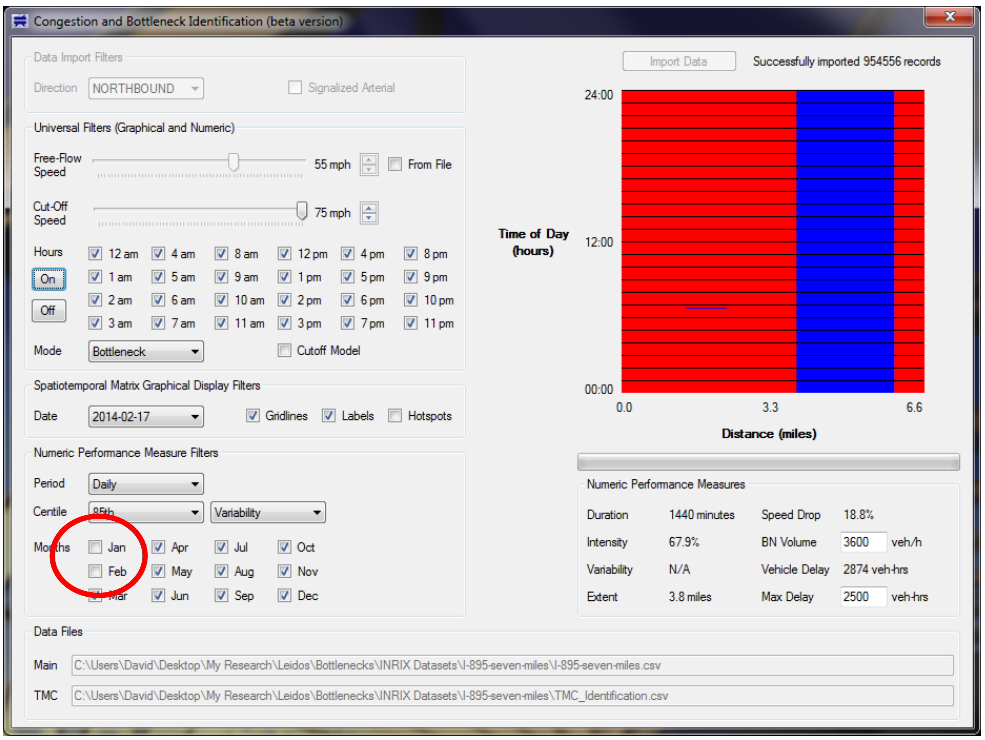


Figure 28. Screenshot. Example of missing data in the CBI tool.

Note that in figure 28, all graphical and numeric performance measures are applicable to the date February 17, 2014, as shown under the "Spatiotemporal Matrix Graphical Display Filters" box. However if the analysis period were changed from daily to annual under "Numeric Performance Measure Filters," subsequent numeric performance measures would reflect the selected months only.

CBI TOOL DISTRIBUTION

The CBI tool and its unpublished internal user's guide may soon be available on the FHWA Web site. Additionally, they can also be obtained by contacting the Saxton Transportation Operations Laboratory (STOL) at FHWA's Turner-Fairbank Highway Research Center (TFHRC).

CHAPTER 3. BOTTLENECK MITIGATION CONCEPTS

DEFINITION OF BOTTLENECKS

For at least 15 years, FHWA has stated that 40 percent of all congestion nationwide is attributable to bottlenecks, while another 5 percent is attributable to poor signal timing, which is also a tangible problem and not a random event.(5) These two causes together are said to define recurring congestion and account for 45 percent of congestion. While this number has been oft-quoted in FHWA literature and widely used by others, it is purportedly based primarily on a congestion study undertaken mostly for, or using data mostly from, the I-95 Corridor Coalition. (See references 24, 35, and 36–39.) Whether or not recurring congestion is 45 percent of all congestion is less the issue than having confidence that recurring causes were correctly identified and aggregated because it is also safe to say that said study did not break out, identify, or otherwise stratify the indicators.(5) In short, recurring congestion just exists, and all other congestion is considered nonrecurring (e.g., special events, bad weather, work zones, and traffic incidents) (see figure 1). Regardless, it is fair to say that no confirmation or additional study has been made since. There is little consensus on a bottleneck definition in the literature other than to say that bottlenecks, by whatever cause, result in the formation of queues upstream of the bottleneck and free-flowing traffic downstream. According to a theory proposed by Kerner, traffic is categorized into three phases: free-flow, synchronized flow, and wide-moving jams.(40) Free-flow is an unrestricted condition. Synchronized flow entails localized congestion. Kerner's synchronized traffic is fixed at a downstream front, which is a defacto bottleneck. Wide-moving jams are more or less systemic congestion, as the downstream traffic velocity moves, even as traffic may propagate upstream through bottlenecks.

The *Merriam-Webster Dictionary* defines a *bottleneck* as: (1) a narrow route or (2) a point of traffic congestion.(41) However, a road does not necessarily have to narrow for a bottleneck to exist (e.g., bottlenecks caused by a weaving condition, sun glare, rubbernecking, or a vertical climb). A layman's understanding of a bottleneck might be too many cars trying to use a highway at the same time. A bottleneck is distinct from congestion because it occurs at a specific location and not pervasively along the entire corridor. The term "traffic bottleneck" infers a localized congestion problem, not a systemic congestion problem. A physical bottleneck cause (e.g., a lane drop or other operationally deficient design on the roadway) only manifests itself when traffic demand overwhelms the available roadway capacity. Otherwise, the design operates safely (otherwise it would have drawn attention on that merit) and unobtrusively, and no one is delayed. The takeaway here is that in the case of a recurring bottleneck location, it takes both a deficiency in the design as well as traffic overburden for the delay to occur, whereas a nonrecurring event is the sole causal event that causes traffic.

ATTRIBUTIONS OF BOTTLENECKS

Many papers have referenced the definition used by Daganzo, which attempts to explain a bottleneck's signature; namely, a bottleneck is considered to be "active" if traffic is detected to be "queued upstream of the location and unqueued downstream."(pg. 106)(42) Geroliminis et al. provides another useful bottleneck definition, as follows: "A phenomenon where the full performance level (capacity) of an entire system cannot be realized due to an abnormality at a single component of the system. The performance at one location thus brings down the performance of the entire system."(pg. 5)[43] In supporting Geroliminis' contention, it is also FHWA's Office of Operation's observation that locally subordinate locations are often indeed the defacto "congestion" that is unfairly attributed to the entire facility (e.g., the highway is congested when in fact only the vicinity of eastbound (EB) Exit 12 is the problem.).[7] This is akin to one bad apple unjustly tainting the entire crop.

Severe congestion can be caused at locations where a physical reduction in roadway width (e.g., lane blockage or lane drop) occurs. This cause may be called a "physical bottleneck." Physical bottlenecks have been the focus of transportation improvements for many years for the simple reason that they are tangible and correctable by redesign in contrast to nonphysical causes of slowdown like rubbernecking, weather, or weaving. Regardless, motorists only care about the result, not the reason behind the bottleneck. Some of these locations are notorious to the extent that they have acquired colorful nicknames by the local motorists, such as the following:

- "Spaghetti Bowl" in Las Vegas, NV.
- "Malfunction Junction" in Knoxville, TN (since remedied).
- "Hillside Strangler" in Chicago, IL.
- "Mixmaster" in Dallas, TX.

NCHRP Report 3-83, *Low-Cost Improvements for Recurring Freeway Bottlenecks*, emphasizes that hidden bottlenecks occur when downstream demands are metered by upstream bottlenecks.[44] When hidden downstream bottlenecks are activated by improvements to upstream bottlenecks, it potentially causes negative system-wide impacts. This makes the case for using predictive tools and real-time data to drive bottleneck solutions even more compelling.

The degree of congestion at a bottleneck is related to its physical design. Some bottlenecks exist on roadways constructed many years ago using designs that were appropriate at the time but are now considered antiquated. Others have been built to sufficiently high design specifications but are simply overwhelmed by traffic. In the FHWA report, *Recurring Traffic Bottlenecks: A Primer Focus on Low-Cost Operational Improvements*, FHWA identified the worst physical bottlenecks in the country and examined the potential benefits that improving them could have for travel times, safety, emissions, and fuel consumption.[2] Ultimately, many definitions and attributes for bottlenecks arose from the literature. However, Margiotta and Spiller were able to summarize the different attributions of bottlenecks (see table 3).

Table 3. Traffic bottleneck attributes.[2]

Bottleneck Characteristic	Description
Occurrences	• **Recurring**: Predictable in cause, location, time of day, and approximate duration. • **Nonrecurring**: Random (in the colloquial sense) as to location and severity. Even if planned in some cases, like work zones or special events, these occurrences are irregular and are not predictably habitual or recurring in location.
Causes	**Recurring operational causes**: A facility determinate condition wherein a fixed condition (the design or function of the facility at that point) allows surging traffic confluence to periodically overwhelm the roadway's physical ability (i.e., capacity) to handle the traffic, resulting in predictable periods of delay.
Examples	**Recurring**: Ramps, lane drops, weaves, merges, grades, underpasses, tunnels, narrow lanes, lack of shoulders, bridge lane reduction, curves, and poorly operating signals.
Supplementary terms (applies to both recurring and nonrecurring bottlenecks)	• **Active bottlenecks**: When traffic released past the bottleneck is not affected by a downstream restriction (i.e., queue spillback) from another bottleneck. • **Hidden bottlenecks**: When traffic demand is metered by one or more upstream bottleneck(s) (i.e., either a lesser or nonexistent bottleneck that would increase or appear, respectively) if only unfettered.
Identification methods (applies to both recurring and nonrecurring bottlenecks)	Motorists typically refer to bottlenecks in terms of added time delay when compared to the same nondelayed trip, but engineers and agencies also measure performance data: average speed (travel time), lane densities, queue lengths, queue discharge rates, vehicle miles traveled, and vehicle hours traveled.
Measurement methods (applies to both recurring and nonrecurring bottlenecks)	Data are collected using manual techniques (e.g., floating cars, aerial photography, or manual counts from video recordings) or from dynamic surveillance (e.g., detectors, radar, video, etc.) collected in real time. Modeling, especially microsimulation, can be used to study the impacts of bottleneck remediation on upstream and downstream conditions.
Classifications	• **Recurring—type I**: Demand surge, no capacity reduction (typically at freeway on-ramp merges). • **Recurring—type II**: Capacity reduction, no demand surge (typically changes in freeway geometry; lane drop, grade, or curve). • **Recurring—type III**: Combined demand surge and capacity reduction (typically in weaving sections). • **Nonrecurring:** Usually classified by event type (e.g., incident or work zone) and impact severity (e.g., duration of the number of lanes lost, closed, or impassable).

Signature trigger	• **Recurring**: Bottleneck is due to overdemand of volume (i.e., peak-hour conditions). The bottleneck clears from the rear of the queue as volume declines. • **Nonrecurring**: Bottleneck is due to loss of capacity due to an incident, or short-term overdemand due to a spot event. The bottleneck clears from the front or rear of the queue, depending on whether the cause is incident-related (former) or volume-related (latter), respectively.
Dissipation criteria	• **Recurring**: When volume overdemand drops back to manageable levels for available capacity (i.e., when off-peak conditions return). • **Nonrecurring**: When dynamic event is removed, queue should dissipate thereafter.

BOTTLENECK IDENTIFICATION FRAMEWORK

Bottlenecks can have various and interrelated causes. This section focuses on identifying geometric or operational challenges associated with a bottleneck location and tailoring a solution or set of solutions to alleviate congestion. A given bottleneck location can have one or more causes. One cause can have multiple solutions, and one solution may address multiple causes, which is discussed further in the next section. Figure 29 shows a decision tree to help transportation agencies identify potential causes of an active bottleneck. The decision tree is divided into geometric and operational challenges. Further subdivisions within the two categories delve into exactly what aspects of geometry or operations may be causing bottleneck conditions.

The appendix contains a detailed description of each potential bottleneck cause in figure 29 and identifies potential solutions. Note that the numbers in the figure correlate to the numbers provided in the appendix. Each subcategory has a description or definition, a list of theoretical or empirical effects of that potential cause, and identification of existing and proposed solutions, where possible.

Recommended use of the bottleneck identification framework is as follows:

1. Try to classify the bottleneck according to the seven categories listed in figure 29.

2. Browse the available bottleneck causes listed under the chosen category.

3. Refer to the appendix to obtain additional information on associated mitigation strategy (or strategies) for the chosen bottleneck cause.

4. Consider the applicability of several mitigation strategies given in chapters 4 and 5 whose details are embedded within those sections.

For example, suppose a bottleneck is periodically identified on a section of freeway with none of the typical physical characteristics that tend to cause bottlenecks (e.g., lane drops, nearby ramps, horizontal or vertical curvature, etc.). If further study determines that this bottleneck tends to form during periods of poor weather, this could potentially be classified under the “Geometric—

Roadway Specific," "Operational Challenges—Agency Related," and "Operational Challenges—Driver Related" categories listed in figure 29. Regarding the "Geometric—Roadway Specific" bottleneck cause sun glare, which is a weather-related issue, one suggestion from the appendix is the installation of redundant traffic control devices. Regarding "responses to weather" under "Operational Challenges—Agency Related," the appendix lists options including education campaigns, reflective lighting, roadside lighting, roadside shelters for pulling over, signage (e.g., lights on when raining), salting or other material to increase traction, and restrictions during weather events. Regarding "unsafe vehicle conditions for weather conditions present" under "Operational Challenges—Driver Related," the appendix discusses vehicle inspection requirements and electronic enforcement. Regarding chapters 4 and 5, none of these solutions specifically target weather-induced, nonrecurrent bottlenecks. Therefore, the framework and playbook provided in figure 29 and the appendix produce a useful set of considerations and/or options in this case.

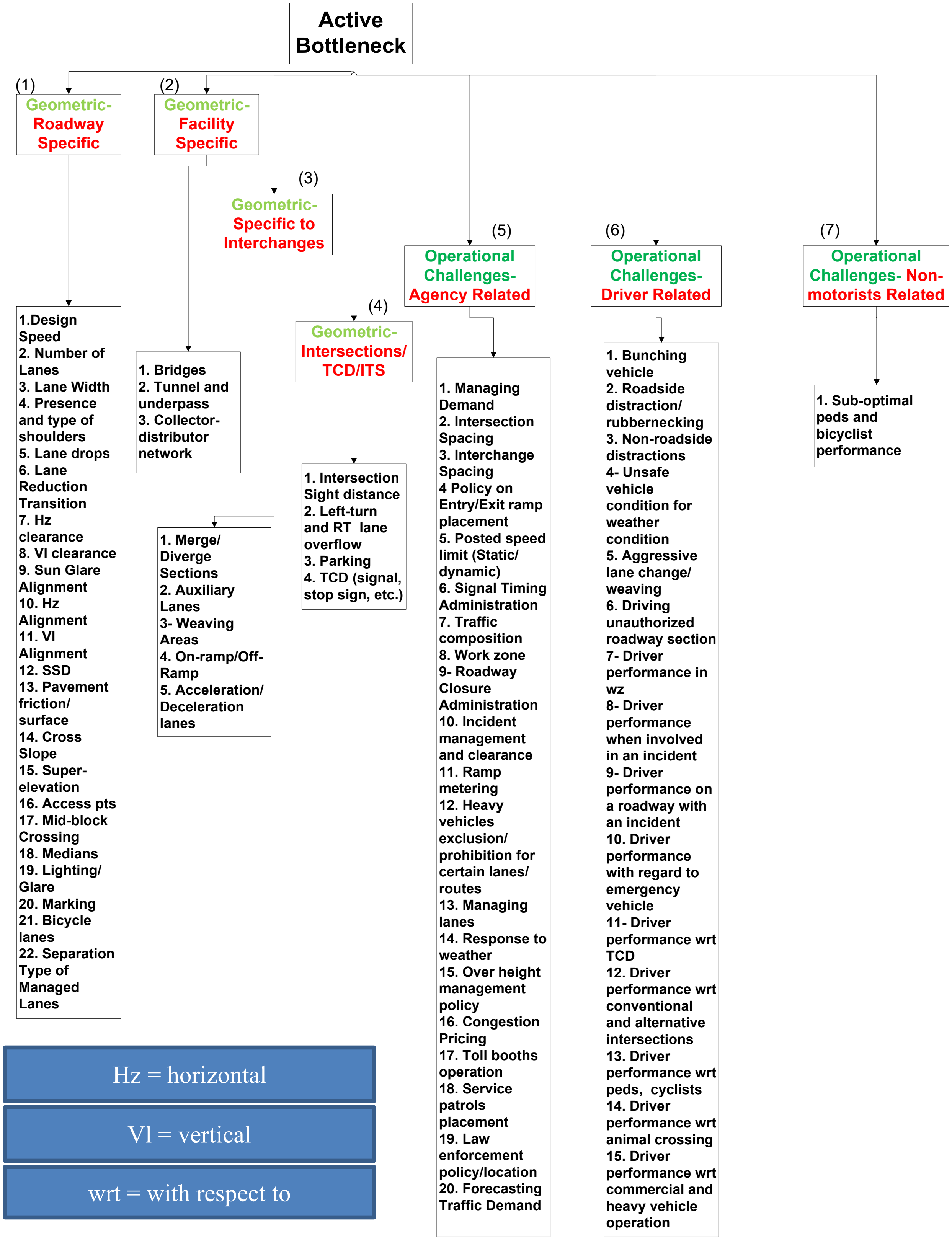


Figure 29. Flowchart. Framework to identify causes of bottlenecks.

CHAPTER 4. COST-EFFECTIVE SOLUTIONS FOR BOTTLENECK MITIGATION

In recent years, bottleneck mitigation has been attempted through strategies such as ramp metering and variable speed limits (VSLs). This section presents recent research on the following five mitigation strategies, which are expected to be cost effective:

- Dynamic lane grouping (DLG) (arterials).
- Dynamic merge control (DMC) (freeways).
- Changing acceleration lane lengths (freeways).
- Hard shoulder running (HSR) (freeways).
- Reducing lane widths to add new lanes (freeways).

Some of the unpublished simulation datasets used to evaluate these strategies can be obtained by contacting the STOL at FHWA's TFHRC.

ALTERNATIVE INTERSECTIONS AND INTERCHANGES

In addition to the five featured strategies, the following alternative intersection and interchange designs are recommended:

- Restricted crossing U-turn intersection (RCUT).
- Median U-turn intersection (MUT).
- Displaced left-turn intersection (DLT) and DLT interchange (DLTI).
- Diverging diamond interchange (DDI).
- Roundabout.

According to chapter 23 of the 2016 HCM, alternative intersections are created by rerouting one or more movements from their usual places to secondary junctions.[45] Often, the rerouted movements are left turns (LTs). Alternative intersection and interchange designs have significantly reduced travel times and delays in many areas compared to conventional intersection designs. Some designs have substantially reduced the number of conflict points between vehicles, thus increasing overall safety. In addition, the alternative designs can often be implemented with minimal disruptions to existing right-of-way. Given the relatively low cost of implementation for many of these designs, the combination of improved mobility and safety has produced outstanding B/C ratios within economic analyses. By moving or eliminating problematic LT movements, the alternative designs have efficiently mitigated congestion at surface street-freeway interchanges and at signalized intersections. For more information, refer to FHWA's *Alternative Intersection and Interchange Report* (AIIR) and *Alternative Intersection Design* Web site.[46,47]

According to the AIIR, DLT intersections tend to be more operationally efficient than RCUT and MUT intersections.(46) However, DLT intersections require a larger footprint than RCUT and MUT intersections and limit access along the displaced LT approaches. Regarding the alternative interchange designs (DDI and DLTI), DDI is more effective for unbalanced flows and heavy LT flows. DLTI is more favorable for heavy and balanced (opposing) through demands. It can handle also heavy LT volumes, although not as heavy as the DDI. FHWA's Capacity Analysis for Planning of Junctions spreadsheet can be used for sketch planning comparison of alternative intersection and interchange designs.(48) Roundabouts are more operationally efficient than traffic signals at isolated locations with moderate, balanced flows on at least three approaches. In addition, roundabouts can be significantly safer than conventional signalized intersections.

DLG

Conventional traffic signal control assumes the intersection's lane assignment configuration is fixed. DLG aims to adjust lane assignment based on fluctuating travel demand to further improve intersection capacity utilization. In previous studies, DLG has been proven effective for balancing lane-by-lane flows and for reducing intersection delays.(49,50) This report describes development of a software application, which automatically identifies candidates for DLT treatment based on intersection volume and signal timing. This report also compares identification criteria in terms of their ability to identify intersections that benefit the most from DLG. Pros and cons of the criteria are discussed along with suggestions about how the software should be applied. The candidate selection software is applied to a large number of intersections in the Northern Virginia area and is shown effective in identifying promising candidates for DLG treatment. Two case study intersections are then used to demonstrate the benefits of DLG. Figure 30 and figure 31 illustrate the potential efficiency created by changing lane use.

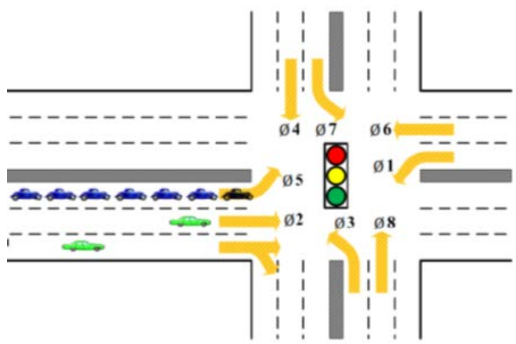


Figure 30. Illustration. Example of LT congestion prior to DLG.

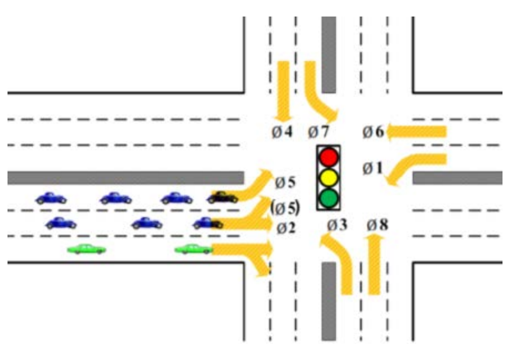


Figure 31. Illustration. Example of LT congestion after DLG.

DLG Screening Criteria

To find candidate intersections having the greatest potential of being improved by DLG, a set of screening criteria needs to be defined. The criteria were developed and tested across 30 signalized intersections within four urban traffic networks in Northern Virginia. Each of the 30 research intersections had four approaches, and each approach had two target movements (LTs and right turns (RTs)). The through (TH) movement is not a target movement because the necessary number of downstream lanes typically would not exist. Intersections were analyzed within eight typical time periods: weekday morning, weekday off-peak, weekday midday, weekday afternoon, weekend morning, weekend afternoon, Saturday peak, and Sunday peak. Four screening criteria were developed: lane geometry, volume change, volume/lane (v/l), and v/c.

Safe Turning Geometry

When identifying intersections eligible for DLG treatment, safe turning geometry is a prerequisite screening criterion. For example, to convert a TH lane to a dedicated LT lane, two criteria would need to be met: (1) a receiving lane exists for the additional LT lane and (2) turn radii for opposing LT movements would not cause head-on collisions within the intersection. Data collection for this research automatically accounted for number of receiving lanes but did not automatically perform conflict analysis. In most intersection designs, the number of exit lanes matches the number of incoming TH lanes. Because converting exclusive LT or RT lanes into shared or exclusive TH lanes is usually not feasible, this research focuses on converting TH lane(s) into LT or RT lanes. To achieve this, there must be at least two TH lanes. If there is only one TH lane, the only option is converting it to a single shared lane, which tends to produce limited mobility benefits. For those reasons, the following two lane geometry conditions were chosen to qualify an LT or RT lane for DLG:

- Enough receiving lanes.
- At least two TH lanes.

If the lane geometry criterion of an approach is not satisfied, that approach will be automatically excluded from DLG considerations.

Volume Change

The volume change screening criterion measures volume changes from one time period to another. Data collection for this research examined every possible combination of the eight time periods. If traffic volume patterns significantly differ between time periods, it implies one of the periods might qualify for DLG. Moreover, the absolute value of volume should be sufficiently high. For example, a volume increase from 10 to 20 vehicles/h is a 100 percent increase, but the absolute value of volume implies minimal benefit from a lane assignment change. In order to meet the volume change criterion, the equations shown in figure 32 through figure 34 should all be satisfied. Threshold values are set at 100 vehicles/h in figure 32 and 0.2 in figure 33 and figure 34, respectively. In the DLG software implementation, these values can be customized for local preferences.

$$max\ [V(LT \text{ or } RT \text{ in } TP1),\ V(LT \text{ or } RT \text{ in } TP2)] > 100 \text{ vehicles/h}$$

Figure 32. Equation. Minimum acceptable demand volumes for DLG.

Where:

V = Demand volume.
$TP1$ = Time period 1.
$TP2$ = Time period 2.

$$\frac{V(LT \text{ or } RT \text{ in } TP1) - V(LT \text{ or } RT \text{ in } TP2)}{V(LT \text{ or } RT \text{ in } TP2)} > 0.2$$

Figure 33. Equation. Minimum change in turn volumes for DLG.

$$\frac{V(\text{adjacent } TH \text{ in } TP1) - V(\text{adjacent } TH \text{ in } TP2)}{V(\text{adjacent } TH \text{ in } TP2)} < -0.2$$

Figure 34. Equation. Minimum change in TH volumes for DLG.

v/l

This criterion compares the v/l of a LT or RT movement versus v/l of the adjacent TH movement. A criterion containing v/l is suggested as a surrogate measure of flow ratio (volume over saturation flow) because saturation flow rate data are not always readily available. The goal is to identify adjacent movements having significantly different flow ratios, and the criterion is shown in figure 35. In a DLG software implementation, the value of 1.5 could be customized for local preferences.

$$v/l(LT \text{ or } RT) > 1.5 \times v/l(\text{adjacent } TH)$$

Figure 35. Equation. Different congestion in adjacent movements for DLG.

Unlike the volume change criteria, v/l does require data on existing lane groupings and further assumes that shared lanes (e.g., a shared LT and TH lane) are exclusive TH lanes. If there are no exclusive LT (or RT) lanes, the number of LT (or RT) lanes would be zero, and the v/l criterion would not be usable. However v/l is more sensitive to congestion levels than the volume change criteria.

v/c

The v/c criterion identifies how much capacity is utilized by each turning movement and should probably be used whenever signal timing information is available. DLG is most effective when v/c of the target LT or RT movement is sufficiently high, such that adding a TH lane significantly improves intersection operations. In addition, TH movement v/c should be sufficiently low, such that removing one lane does not generate undue congestion. The researchers found that when target LT or RT movement v/c exceeds 0.7, DLG tends to produce noticeable performance improvements. The TH movement should also be undersaturated (v/c lower than 1.0) after one lane is removed. Finally, if the target movement v/c is extremely high (e.g., over 1.3), it qualifies the approach for DLG regardless of TH movement v/c. Therefore, the criterion can be applied as follows: either the equations in figure 36 and figure 37 should both be satisfied or the equation in figure 38 should be satisfied.

$$v/c(\text{target } LT \text{ or } RT) > 0.7$$

Figure 36. Equation. Sufficient turn movement congestion for DLG.

$$v/c(\text{adjacent } TH) < \frac{NLanes(\text{adjacent } TH) - 1}{NLanes(\text{adjacent } TH)}$$

Figure 37. Equation. Minimal TH movement congestion for DLG.

Where:

NLanes = Number of lanes.

$$v/c(\text{target } LT \text{ or } RT) > 1.3$$

Figure 38. Equation. Severe turn movement congestion for DLG.

The simplified calculation of (*NLanes* – 1)/*Nlanes* assumes all TH lanes are equally utilized because lane-by-lane queue length analysis may not be practical or available, especially when automatically screening hundreds of intersections. In the four criteria, v/c requires the most amount of data: volume, lane grouping, and existing signal timing. During the data analysis portion of this research, capacity values were obtained from Synchro®. The four screening criteria were coded into a JAVA™ program, which imports input data from Synchro®. The JAVA™ program can then report which intersections were flagged by each criterion.

Data Collection and Analysis

The DLG screening software was tested across 30 intersections (from 4 urban traffic networks) in Northern Virginia. Intersection data were provided by the Virginia Department of Transportation (VDOT). Available at each intersection were traffic volume, signal timing, and v/c data for eight different time periods: weekday morning, weekday off-peak, weekday midday, weekday afternoon, weekend morning, weekend afternoon, Saturday peak, and Sunday peak. Each intersection had eight target LT and RT movements (two per approach). Thus, there were a total of 1,920 DLG candidates in the pool (30 × 8 × 8 = 1,920).

Table 4 displays the number of DLG candidates flagged by each of the screening criteria. Note that the number of candidates selected by lane geometry is not listed; this criterion was simply treated as a prerequisite to be met before consideration by the other criteria. Flagged candidates were then further studied within Synchro®. If no delay reduction was observed, that candidate was considered a false positive. Otherwise, the candidate was considered a true positive. The number of true positives is listed in parentheses in table 4. Some cells in the table have no parentheses, meaning those candidates were not analyzed due to time constraints. Table 4 shows that the v/c criterion flagged 24 candidates, and 22 of those were promising (true positive rate of 92 percent). True positive rates for the other two criteria were much lower. Within the OBN and TCN networks, true positive rates for the other criteria were lower than 20 percent.

Table 4. Number of candidates selected in each network by four criteria.

Selection Criteria	Traffic Network ID				Sum
	A	B	C	D	
Number of intersections	7	10	8	5	30
v/c criterion	6(5)	2(2)	1(1)	15(14)	24(22)
v/l criterion	4(1)	42(7)	22	68	136
Volume change criterion	4(2)	12(1)	15	30	61

Note: The number in parentheses indicates how many of the candidates could be improved by DLG.

Table 5 lists flagged candidates in the OBN and TCN networks, along with the criteria that flagged them, and the corresponding delay reductions from applying DLG.

Table 5. Analysis of the flagged candidates.

Candidate ID	Network	v/c (Yes/No)	V/L (Ye/No)	Volume Change (Yes/No)	Percent Delay Change	Percent Delay Change (Optimized)
1	TCN	Yes	Yes	No	-4.5	-4.2
2	TCN	Yes	Yes	No	-1.7	-0.7
3	TCN	No	Yes	Yes	-1.7	-2.0
4	TCN	No	Yes	No	-5.9	-0.6
5	TCN	No	Yes	No	-4.6	-4.6
6	TCN	No	Yes	No	-2.6	-3.1
7	TCN	No	Yes	No	-2.1	-2.1
8	OBN	Yes	No	No	-8.2	-8.2
9	OBN	Yes	No	No	-5.0	-10.4
10	OBN	Yes	No	No	-8.3	-10.5
11	OBN	Yes	Yes	Yes	-14.0	-20.6
12	OBN	Yes	No	No	-9.8	-20.7
13	OBN	No	No	Yes	+12.9	+9.2

The v/c criterion worked quite well in the OBN network, as its five flagged candidates showed delay reductions over 5 percent (candidate IDs 8–12). Candidate ID 11 was flagged by multiple criteria and exhibited the largest overall delay reduction. Candidate ID 13's delay increased because the opposing LT had protected-permissive phasing. When the number of subject-approach TH lanes decreased from two to one, available gaps decreased on the remaining TH lane. As a result, LT delay on the opposing approach increased, cancelling out delay reductions on the subject approach. The v/l criterion flagged the largest number of candidates in the TCN network. Candidate IDs 3–7 were not flagged because v/c was lower than the 0.7 threshold. However, these candidates had minimal delay reductions (less than 6 percent). As shown in table 5, intersection delay reduction was less than 6 percent for all flagged candidates in the TCN network. Together, the proposed criteria captured different aspects of the intersection pattern. For example, the northbound (NB) approach at Davis Drive and Church Road (in Sterling, VA) has a dedicated LT lane, two TH lanes, and a dedicated RT lane. The v/l criterion flagged all eight periods of the NB LT movement. Converting a lane from TH to LT reduced intersection delay for all eight periods. However, none of the eight periods were selected by volume change. In summary, the Northern Virginia data collection implied that v/c flagged the top candidates as having the most potential for improvement under DLG. Candidates missed by v/c but flagged by other criteria either produced minimal benefits (less than 6 percent), or the intersection performance became worse.

Case Studies

This section presents detailed results for two promising candidates, which were flagged by the screening software. These case studies demonstrate the analysis process and clarify the performance improvements possible under DLG treatment.

Case Study 1

At the intersection of Reston Parkway and Sunrise Valley Drive (R&S), the afternoon peak westbound (WB) RT lane was flagged by v/c and v/l. Weekday morning and afternoon peak-hour volumes are shown in figure 39 and figure 40.

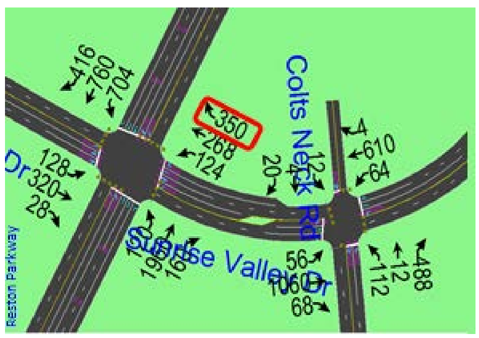


Figure 39. Illustration. Morning peak configuration for case study 1.

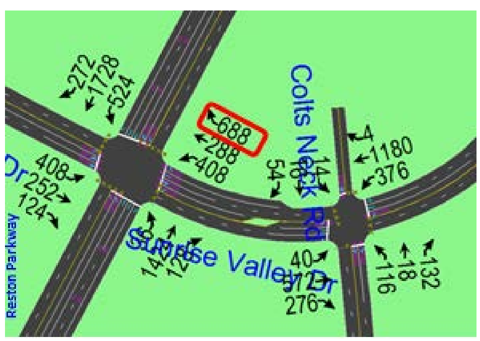


Figure 40. Illustration. Afternoon peak configuration for case study 1.

During the afternoon peak, there was heavy WB RT traffic (almost double the volume of the morning peak). Heavy RT volume exceeded capacity of the RT lane, causing a v/c of 1.56 and significant delays at the intersection. Moreover, a long queue on this approach degraded functionality of the adjacent upstream intersection (Colts Neck Road and Sunrise Valley Drive (C&S)). As a result, both intersections experienced significant delay during the afternoon peak. To improve the performance of these closely spaced intersections, the WB exclusive TH lane was converted to a TH+RT shared lane at R&S. Figure 41 and figure 42 illustrate the before-and-after lane configuration. The signal timing plan was then re-optimized to better handle this new intersection geometry. Both the existing and DLG lane assignments were simulated in SimTraffic™. Each lane assignment condition was simulated for five runs, having five sets of random number seeds. Each run simulated 1 h of traffic conditions.

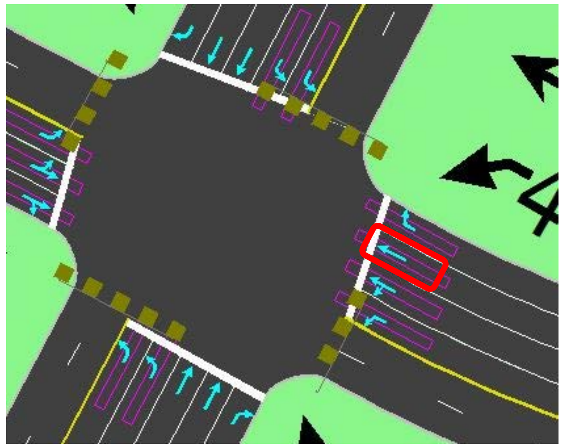


Figure 41. Illustration. Lane assignment before DLG for case study 1.

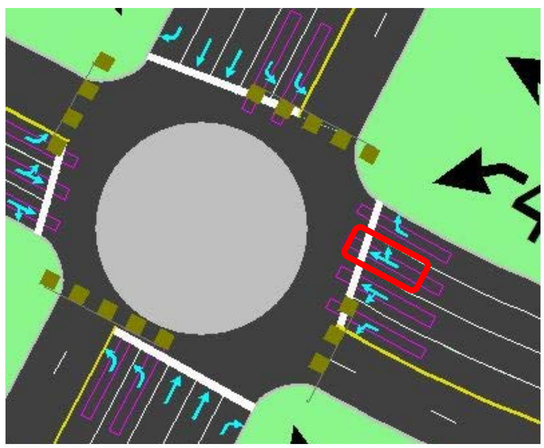


Figure 42. Illustration. Lane assignment after DLG for case study 1.

Table 6 lists the before-and-after delays at each intersection. It shows a reduction of total delay at the R&S intersection by 25.3 vehicle-h and at the C&S intersection by 44.0 vehicle-h during the 1-h simulation. Although the WB LT and TH movements performed worse at the R&S intersection under DLG, the WB RT improved, decreasing overall intersection delay. For the C&S intersection, performance of all WB movements was greatly improved. Based on the above analysis, DLG is suggested for weekday morning peak and afternoon peak hours.

Table 6. DLG benefits for case study 1.

Traffic Delay	R&S Intersection Existing	R&S Intersection DLG	C&S Intersection Existing	C&S Intersection DLG
WB LT delay/vehicle (s)	69.7	104.2	84.2	68.6
WB TH delay/vehicle (s)	71.4	107.9	207.4	82.7
WB RT delay/vehicle (s)	77.8	73.1	318.8	121.7
Total delay/vehicle (s)	167.3	138.3	206.4	129.2
Total delay (h)	253.0	227.7	136.6	92.6
Delay reduction (h)	25.3		44.0	

Case Study 2

For the intersection of Nutley Street and Lee Highway in Arlington, VA, whose turn movement volumes are shown in figure 43, the Sunday peak EB LT lane was flagged by all four screening criteria. The v/c ratio of this lane was 0.99, which is much higher than other time periods during the week. To improve the performance of this intersection, the left-most dedicated TH lane was converted to an exclusive LT lane. Under existing conditions, LTs move with permissive phasing. After the treatment, LTs can move with protected phasing. The timing plan was then reoptimized to better handle the new lane assignments. Each lane assignment condition was simulated for five runs having five sets of random seeds. Each run performed 1 h of traffic simulation.

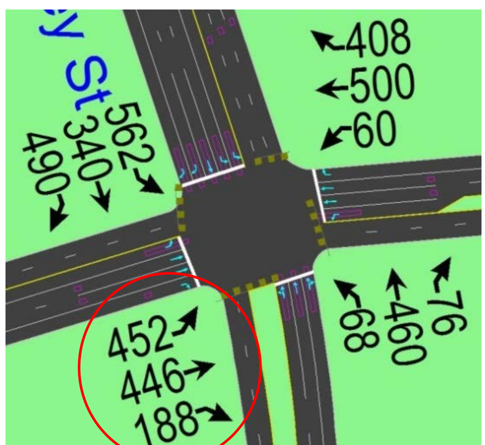


Figure 43. Illustration. Traffic demand for case study 2.

Table 7 lists the before-and-after delays for both intersections. The table shows a minor delay increase for the EB TH movement but a large delay reduction for the EB LT movement. This brings a total delay reduction of 15.9 h during the 1-h simulation. Therefore, dynamically converting a TH lane to a LT lane was beneficial during the Sunday peak hour.

Table 7. DLG benefits of case study 2.

Traffic Delay	Nutley Street and Lee Highway Intersection	
	Existing	DLG
EB LT delay/vehicle (s)	147.0	78.3
EB TH delay/vehicle (s)	24.7	26.8
EB RT delay/vehicle (s)	8.9	9.2
Total delay/vehicle (s)	60.4	47.3
Total delay (h)	70.1	54.2
Delay reduction (h)	15.9	

Implementation Example

The NB RT movement at the intersection of Shady Grove and North Frederick roads in Maryland has implemented a dynamic lane assignment strategy. Table 8 lists the discrepancy between morning peak and evening peak demands. The WB RT volume at 8 a.m. is 1,087 vehicles/h, while the same volume at 5 p.m. is only 678 vehicles/h. So during the morning peak, a dedicated TH lane is converted to RT.

A black-and-white dynamic message sign (DMS) over the dynamic lane at the intersection indicates whether the lane serves TH or RT traffic. When the white arrow is on, it means the lane is dedicated for RTs. Otherwise, it is a dedicated TH lane. Another sign 200 ft upstream of the intersection notifies drivers of dynamically assigned lanes ahead.

Table 8. WB turning movement counts at the Shady Grove intersection.

Time	Number of Vehicles by Turning Movement			
	LT	TH	RT	Total
6 a.m.	119	428	404	951
7 a.m.	370	477	926	1,773
8 a.m.	**445**	**569**	**1,087**	**2,101**
9 a.m.	350	831	1,395	2,576
10 a.m.	478	748	691	1,917
11 a.m.	488	933	743	2,164
12 p.m.	562	963	689	2,214
1 p.m.	567	1,096	820	2,483
2 p.m.	701	1,236	844	2,781
3 p.m.	660	1,070	568	2,298
4 p.m.	1,092	1,757	495	3,344
5 p.m.	**950**	**1,865**	**678**	**3,493**
6 p.m.	1,030	1,862	629	3,521

Note: Bold values indicate peak period demands.

B/C Analysis

When performing B/C analyses, determining the monetary value of travel time and delay reduction is a first step. The SHRP2 report, *Identification and Evaluation of the Cost-*

Effectiveness of Highway Design Features to Reduce Nonrecurrent Congestion, reviewed travel time monetization research.(51) One conclusion was that personal travel time (including commuter travel) should be valued at 50 percent of the prevailing wage rate. The report further cited a national wage rate of $21 per hour in 2009. Assuming an average occupancy of 1.6 persons per vehicle, hourly savings would be $17/vehicle-h as shown in figure 44.

$$\frac{\$21}{\text{h}} \times \frac{1 \text{ wage rate}}{2} \times \frac{1.6 \text{ persons}}{\text{vehicle}} = \frac{\$17}{\text{vehicle-h}}$$

Figure 44. Equation. Monetary savings for each vehicle-hour of delay reduction.

In case study 1, DLG saved 69.3 vehicle-h of delay. The closely spaced pair of intersections in case study 1 served approximately 6,800 vehicles during the peak hour. Therefore, assuming a cost of $17 per vehicle-h for 2015, total hourly time savings would be $1,178 as shown in figure 45.

$$69.3 \text{ vehicle-h} \times \frac{\$17}{1 \text{ vehicle-h}} = \$1{,}178$$

Figure 45. Equation. Total hourly time savings for the DLG example.

DLG would presumably be implemented where lane grouping is effective during the morning peak period but ineffective during the afternoon peak (or vice-versa). Thus, it seems appropriate to assume these cost savings would only materialize during one peak period per day. If the number of commuting days per year is derived by excluding weekends and holidays, then the annual cost savings attributed to DLG could be as follows:

$$\frac{\$1{,}178}{1 \text{ commuting day}} \times \frac{250 \text{ commuting days}}{1 \text{ year}} = \frac{\$294{,}500}{1 \text{ year}}$$

Figure 46. Equation. Annual cost savings for the DLG example.

Case study 2 revealed a more modest delay reduction of 15.9 vehicle-h. Repeating the cost calculations for case study 2 yields savings of $67,575 per year. Based on these results, it appears the DLG treatment may yield greater benefits at high-volume intersections (e.g., 6,800 vehicles/h) versus low-volume intersections (e.g., 4,100 vehicles/h).

Thus, in deciding whether or not to implement DLG strategies, annual benefits in the range of $68,000 (low-volume intersection) to $295,000 (high-volume intersection) would need to be compared against the costs of DLG implementation. These implementation costs would include the cost of installing and maintaining a DMS at the signalized intersection. It could also include the engineering analysis costs of applying the screening criteria (to identify candidate intersections) plus law enforcement (if any) for the new lane use.

Regarding the cost of installing and maintaining a DMS, the U.S. Department of Transportation (USDOT) Knowledge Resources Web site contains a list of unit cost entries for DMS.(52) These DMS costs vary widely from $10,000 to $200,000, but the signalized intersection DMS would presumably be less expensive than the freeway installations. Ideally, lifecycle maintenance and operation costs for the DMS would also be considered. A search for "DMS price" on the FHWA

Web site leads to the USDOT ITS Benefits, Costs, and Lessons Learned Database, and some of the relevant prices are summarized in table 9.(52)

Table 9. DMS capital and operating costs.

Type of DMS	Capital Cost (Thousands of U.S. Dollars)	Operating Cost (Thousands of U.S. Dollars)
DMS	47–117	2.4–6
DMS tower	25–120	
DMS—portable	18–25	1.2-2

Note: Blank cell indicates data were not found.

The USDOT ITS Benefits, Costs, and Lessons Learned Database further states the following:

> "The low capital cost for the DMS is for a smaller DMS installed along arterial. The high capital cost is for a full matrix, LED, 3-line, walk-in DMS installed along a freeway. These costs do not include installation. DMS Tower costs include a low capital cost associated with a cantilever structure. The high capital cost is for a truss structure that spans across 3–4 lanes. The portable DMS sign includes a trailer mounted DMS (3-line, 8-inch character display) and may include trailer, solar or diesel power."(52)

According to FHWA's *Operations Benefit/Cost Analysis Desk Reference*, many current B/C analysis tools use average annual benefits and costs in their analyses.(53) It was previously indicated that annual benefits would be in the range of $68,000 (low-volume intersection) to $295,000 (high-volume intersection). VDOT assumes a 15-year lifespan for DMS signs. In addition to the DMS (approximately $25,000) located above the intersection, it is helpful to have an upstream DMS (another $25,000) with its own mast arm (approximately $75,000). Assuming a signalized intersection DMS would have a total capital cost of $125,000, anticipated useful life of 15 years, and annual operations and maintenance (O&M) costs of $2,000 per year, the average annual cost and benefits would be as follows:

- Average annual cost = ($125,000 capital/15 years) + $2,000 O&M = $10,333 per year.
- Average annual benefit (low-volume intersections) = $68,000 per year (B/C ratio of 7:1).
- Average annual benefit (high-volume intersections) = $295,000 per year (B/C ratio of 29:1).

These are mobility-based B/C ratios not accounting for safety and environmental impacts. These ratios also assume that DLG is only applied to intersections flagged as good candidates for DLG treatment according to the screening process.

Design Guidance for Safe and Efficient Implementation of DLG

The following guidance is suggested for safe and efficient implementation of DLG.

Geometric Design

DLG is intended for implementation within existing roadways with no need for new curb cuts or alignment. However, as described previously, safe turning geometry is a prerequisite for DLG treatment. For example, to convert a TH lane to a dedicated LT lane, three criteria would need to be met: (1) a receiving lane exists for the additional LT lane, (2) turn radii for opposing LT movements would not cause head-on collisions within the intersection, and (3) the proposed lane grouping is compatible with any bicycle and pedestrian facilities present.

Signing

Black-and-white regulatory lane-use signs are essential to convey lane use, both at the intersection and in advance of the intersection, when the lane configuration is not defined by default rules of the road. The advance notification is especially critical in a dynamic situation where drivers may not expect a change. The use of traffic signal indications can supplement but not replace the regulatory signs. Figure 47 illustrates an example of upstream signage for DLG in the middle lanes. In this case, an overhead DMS is helpful because it sits directly above the affected lane. Regardless of which lane is dynamically controlled, similar DLG signage is needed at the intersection stop line to reinforce the message.

Figure 47. Photo. Upstream DLG signage in Maryland.

Striping

It is perhaps best for dynamically grouped lanes to not have painted turn movement arrows on the pavement, as this would introduce a contradiction with the DMSs and/or signal displays. The DLG treatment could theoretically benefit from electronic lighted striping where exclusive and shared-lane arrows could be dynamically superimposed on the roadway.

Signalization

During the DLG study that contributed to this report, the research team recorded hours of video at the Shady Grove intersection in Maryland. The videos indicated that some vehicles avoided the dynamically controlled RT lane and showed a strong preference for the statically controlled conventional RT lane. It was theorized that this could be in part due to the green ball signal display above the inside RT lane. When drivers see a green ball, they are conditioned to expect a TH movement. This signalization also conflicts with the DLG-based DMS above the inside lane, suggesting that an RT movement is allowed. When conflicting messages are present, drivers may have a tendency to obey the traditional signal display, as opposed to the less-common DMS. Thus, in cases like this, shared TH and RT arrow signalization could improve lane utilization and further reduce delay at the intersection. In addition, eliminating the contradictory messages (i.e., given by the green ball and the DMS) could help to improve safety. Similar benefits could be obtained for shared TH and LT cases. However, the nature of protected and permissive LT movements introduces new ramifications.

Summary

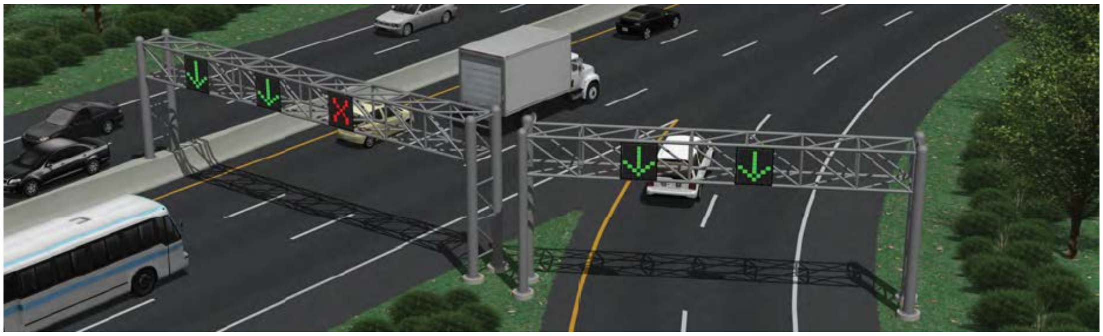
At arterial intersections, traffic demand may vary significantly by time, and the lane assignments designed for severe peak-hour traffic might not suit these different demand patterns. DLG can reduce the gap between supply and demand by dynamically changing lane assignments. This study developed a software tool with four screening criteria to quickly and easily identify intersections to benefit from DLG. It was found that v/c was shown to be the most effective criterion, with a true positive rate exceeding 90 percent and flagged candidates showing noticeable delay reductions. The volume change criterion appears helpful for identifying significant volume changes between time periods. The v/l criterion works similar to v/c but does not need signal timing data. In the two case studies, DLG reduced the overall intersection delay by approximately 20 percent. For agencies that manage intersections' signal timing and lane assignments, there is a B/C tradeoff. The v/c criterion requires the most data but can pinpoint the most promising candidates. If an agency wants to identify as many promising intersections as possible, it should use the full set of criteria and spend more time analyzing the flagged candidates. Future research will include enhancing the software tool to automatically select DLG treatments and estimate benefits accordingly. Another direction of future work is to develop a real-time DLG protocol capable of changing both lane assignments and signal timing based on predicted traffic volume data.

DYNAMIC JUNCTION CONTROL (DJC)

Research Background

In response to growing traffic congestion, an increasing number of active traffic management (ATM) strategies have been developed and implemented internationally. Among these strategies are HSR, VSLs, queue warning, dynamic truck restrictions, ramp metering, managed lanes, dynamic rerouting, traveler information, and DJC. These strategies are capable of monitoring traffic flow and can dynamically control speeds, reduce capacity drops, and inform road users of network conditions to optimize traffic and safety performance.

Of these strategies, DJC represents a component of the ATM system. It can be used at freeway off- and on-ramps to dynamically change lane allocation for interchanges. This report focuses on DJC and other potentially underrated strategies as opposed to popular strategies like ramp metering, which have been extensively researched and implemented in recent decades. The rationale for use is that in some traffic conditions or during certain times of day, it may be more effective to use existing downstream or upstream lanes for one type of movement or for traffic coming from the main lanes. At other times of day, it may be more effective to use the TH lanes to accommodate ramp merging, as illustrated in figure 48.[54]



Source: Schneider Electric and RK&K.

Figure 48. Illustration. DMC.[55]

One component of DJC strategies is DMC. When ramp congestion is high and mainline congestion is medium, hesitation and gap-finding delays may be disruptive to ramp capacities and flows, thus creating rear-end collision potential on the ramp. Alternatively, lane drops in the merge area can easily cause a bottleneck, leading to upstream congestion on both roads. In this situation, closing a lane on the lower-volume road would reduce or eliminate the friction caused by “weaving” (gap acceptance) movements in the merge area. As such, the DMC strategy is hypothesized to reduce traffic congestion at the merge of two freeways.

DMC is currently used in Germany and the Netherlands. Lane control signs are installed over both approaches upstream of the merge. Priority is given to the higher volume entrance. This strategy produces a more uniform traffic flow, with fewer conflicts and safer maneuvers.[56] There is no current implementation of DMC in the United States. In the 2014 *Greater Richmond Safety and Mobility Study*, DMC was considered for the two interchanges between I-95 and I-64 (Bryan Park and Shockoe Valley).[57] B/C analysis, however, revealed that deployment at Bryan Park would have adverse impacts and should not be implemented. Conversely, DMC implementation at Shockoe Valley was expected to have positive net impacts if planned modifications at the Chamberlayne interchange were implemented. One study suggested certain conditions should be met before applying DMC, including: (1) level of service (LOS)[2] F on entrance ramps for 2 consecutive hours during morning and afternoon peak periods, (2) over 1,200 vehicles/h on a single-lane on-ramp or over 2,000 vehicles/h on a two-lane on-ramp, and (3) LOS D or better on the upstream mainline lanes following implementation of DMC.[58] Research by TTI suggests it is preferable to have at least 900 vehicles/h on the single-lane on-ramp and LOS E or better on the mainline lanes following DMC implementation.[59] This research investigated the efficiency of DMC for common freeway-to-freeway and freeway-ramp merge geometries.

[2]According to chapter 8 of the 2016 HCM, LOS can be used to communicate roadway performance to laypersons. For example, LOS A represents the best operating conditions from the traveler’s perspective, and LOS F represents the worst.[45]

Road Geometries Selection

One set of freeway-to-freeway experiments involved multiple two-lane sections merging into three lanes (route A). In a second set of experiments, a three-lane section merged into a two-lane section to become four lanes (route B). Scenarios closing mainline lanes and ramp lanes were investigated with traffic demand variations. Table 10 describes the lane configurations and traffic demand variations for each freeway-to-freeway lane closing alternative. Figure 49 through figure 52 show the basic geometries for all freeway-to-freeway lane configurations and lane closing alternatives.

Table 10. Freeway-to-freeway merge scenarios.

Lane Closing Route	Route A		Route B	
	Number of Lanes	Volume (vehicles/h)	Number of Lanes	Volume (vehicles/h)
Route A	2	1,000–2,200 (by 200)	2	2,400–4,600 (by 200)
	3	2,500–4,600 (by 300)	2	3,000–4,600 (by 200)
Route B	2	2,400–4,600 (by 200)	2	1,000–2,200 (by 200)
	3	4,200–6,500 (by 300)	2	1,000–2,400 (by 200)

Note: Volume was increased in increments of 200 or 300 vehicles/h to create a thorough set of testing scenarios (indicated by number in parentheses).

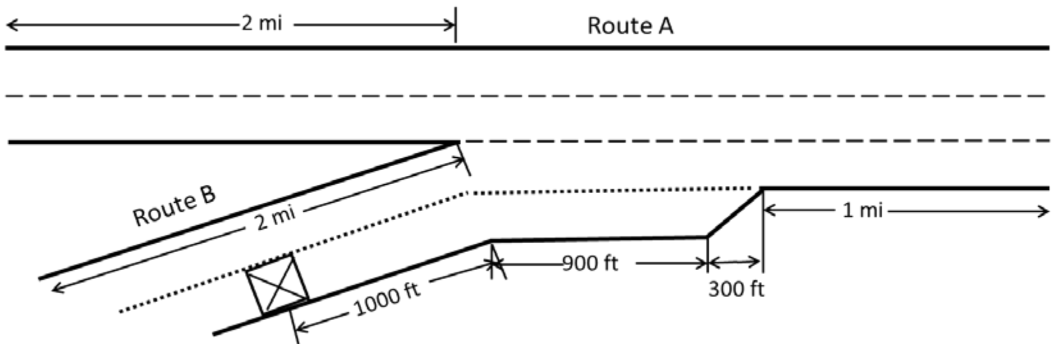


Figure 49. Illustration. Merging of multiple two-lane freeways (closure on route B).

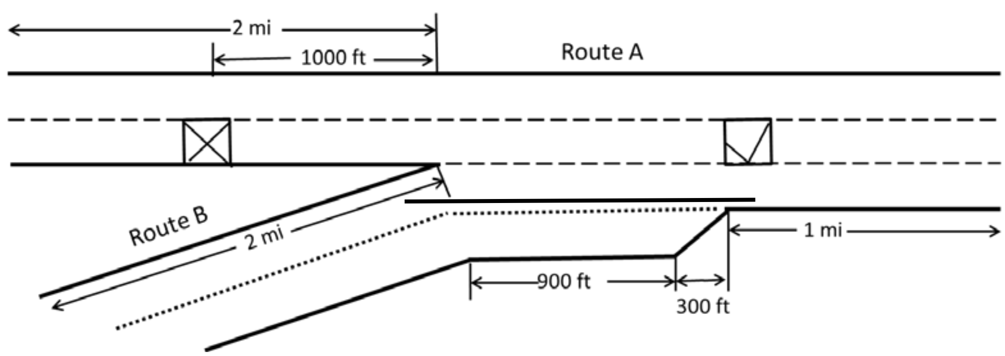


Figure 50. Illustration. Merging of multiple two-lane freeways (closure on route A).

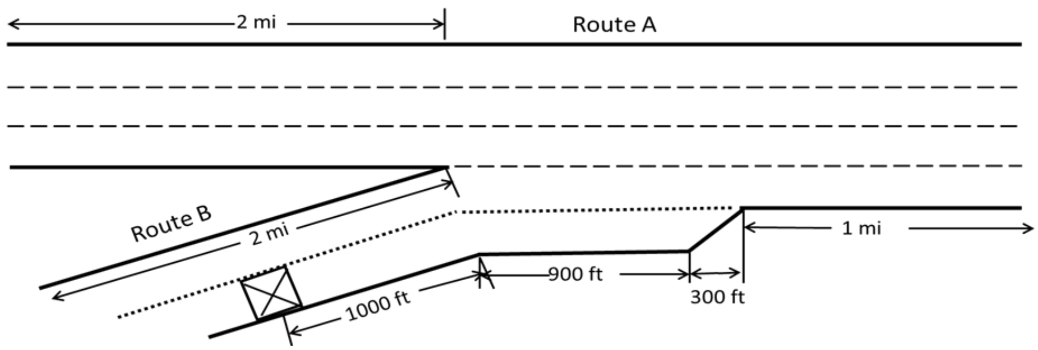


Figure 51. Illustration. Merging of three- and two-lane freeways (closure on route B).

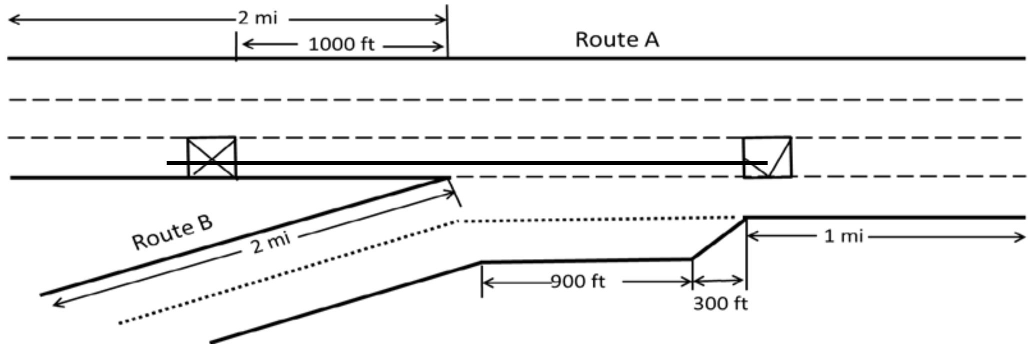


Figure 52. Illustration. Merging of three- and two-lane freeways (closure on route A).

Table 11 describes the lane configurations and traffic demand variations for freeway on-ramp merges. Figure 53 through figure 55 show basic geometries for the on-ramp scenarios considered in this research. The selected lane configurations covered the most common designs in the United States.

Table 11. Freeway on-ramp merge scenarios for DMC.

Mainline		On-ramp	
Number of Lanes	Volume (vehicles/h)	Number of Lanes	Volume (vehicles/h)
2	1,000–2,400 (by 200)	1	1,000–2,000 (by 200)
3	2,100–4,800 (by 300)	1	1,000–2,000 (by 200)
4	4,800–7,200 (by 400)	1	1,000–2,000 (by 200)

Note: Volume was increased in increments of 200 or 300 vehicles/h to create a thorough set of testing scenarios (indicated by number in parentheses).

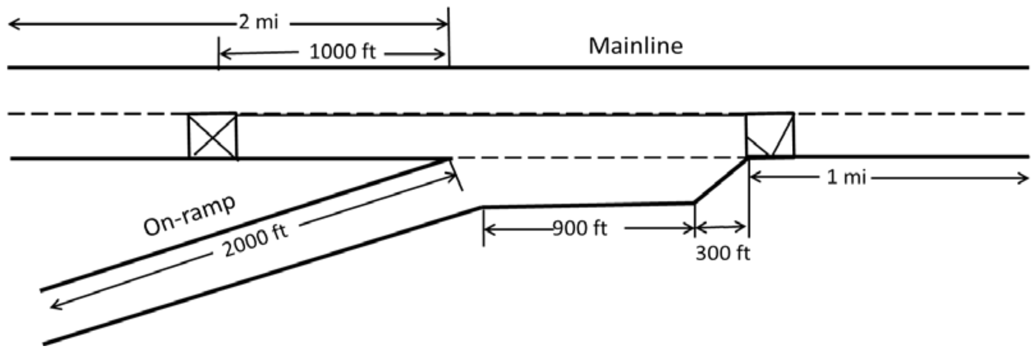


Figure 53. Illustration. Merging of one-lane ramp and two-lane mainline (closure on mainline).

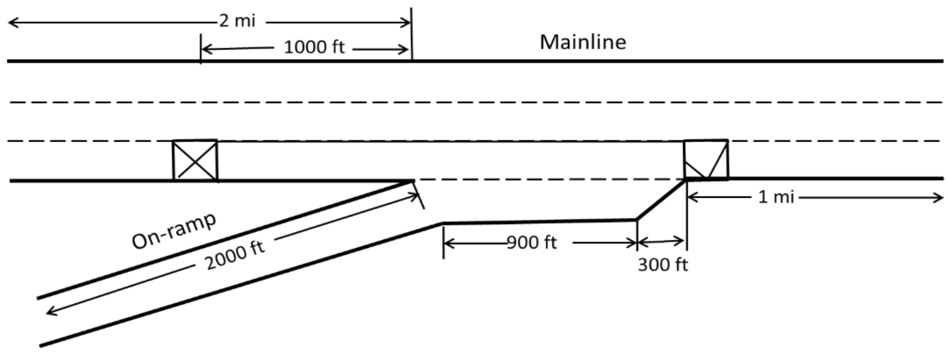


Figure 54. Illustration. Merging of one-lane ramp and three-lane mainline (closure on mainline).

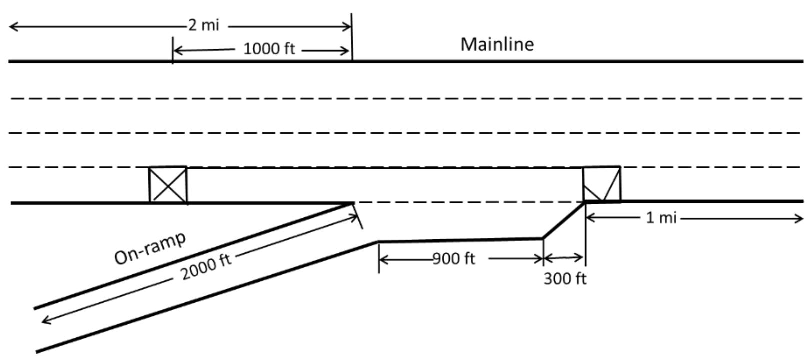


Figure 55. Illustration. Merging of one-lane ramp and four-lane mainline (closure on mainline).

Model Calibration

Micro-simulation software VISSIM ™ was used to simulate road network performance under various scenarios. The simulation period for each scenario was 8,400 s, including 1,200 s of warm-up time. In each scenario, traffic demands on both roads were assumed to be fixed over the entire simulation period. Each scenario was simulated 15 times with 15 sets of random number seeds. Preliminary tests indicated that, for most scenarios, simulation results would converge after approximately 9–10 runs. For DMC cases, drivers were notified of lane closures 2,500 ft upstream of the lane closing point. Vehicle characteristics and driving behaviors were based on a precalibrated network (VA 267 merging with I-66 in Northern Virginia), which represents a typical freeway-to-freeway merge. The calibration process is briefly summarized as follows:

- A total of 10 traffic sensors were installed.
- Two sensor types were used—remote traffic microwave sensor mounted on trailers and Wavetronix™, which are all side-fired radar systems with similar measurement accuracy and resolution.
- All sensors provided lane-by-lane data, which include speed, flow, and occupancy sampled every 30s.
- Calibration was based on the traffic flows measured at all 10 sensors.

The following three criteria were employed for calibration: (1) relative root mean square error (RRMSE) for flow, (2) FHWA Geoffrey E. Havers (GEH) criteria, and (3) accumulated flow.

RRMSE for Flow

It was assumed that a traffic state parameter had values from field data and simulated data, as shown in figure 56.

$$\overline{\mathbf{x}} = \left[\overline{x}_1, \overline{x}_2, \ldots, \overline{x}_n\right]$$

Figure 56. Equation. Simulated values and mean values over all simulation replications.

In the simulation experiment, 10 replications were used for each day. RRMSE is defined by the equation in figure 57 as follows:

$$RRMSE = \sqrt{\frac{1}{n}\sum_{k=1}^{n}\frac{(\overline{x}_k - x_k)^2}{x_k^2}}$$

Figure 57. Equation. RRMSE.

Where:

n = Total number of simulation replications.
k = ID number for the current simulation replication.
$\overline{x}_k$ = Mean performance measure value of all simulation replications.
x_k = Performance measure value of the current simulation replication.

FHWA GEH Criteria

The GEH statistic is calculated at each time point, *k*, as shown in figure 58 as follows:

$$GEH(k) = \sqrt{\frac{2(E(k)-P(k))^2}{(E(k)+P(k))}}$$

Figure 58. Equation. FHWA GEH criteria.

Where:

E = Expected performance measure value at time k as predicted by the traffic model.
P = Actual performance measure value at time k as measured in the field.

Then the percentage of aggregated time points which satisfy this condition can be calculated according to different traffic situations.

Accumulated Flow

RRMSE was not adequate to measure closeness of the model because if the error is biased, RRMSE could remain small, but accumulated error over time could be large. For example, if on-ramp or mainline upstream flows were persistently smaller than field-measured flows, many vehicles would be blocked outside the system, and the system would not represent real traffic. It was thus decided to use both RRMSE and accumulated flow for flow calibration. This is also applicable to the GEH criterion. Vehicles' parameter values and their variances or thresholds should come from measured data such as headway and its variance, vehicle acceleration-deceleration capabilities, and driver response time. For this research, parameters were selected by analyzing Next Generation Simulation and/or Berkeley Highway Lab data on headway, jam density, and other measures.(60,61) Table 12 shows the driving behavior parameters that were employed in this study.

Table 12. Calibrated driving behavior parameters.

Parameter	Freeway	On-Ramp	Merge
Maximum look ahead distance (ft)	692.65	820.21	1,169.26
Number of observed preceding vehicles	2	2	2
Average standstill distance (ft)	4.30	9.61	9.19
Headway at a certain speed (s)	0.90	1.28	1.23
Longitudinal oscillation (ft)	11.12	6.14	7.12
Start of the deceleration process	8.00	8.00	8.00
Acceleration behavior at 50 mi/h (ft/s^2)	4.92	4.92	4.92
Maximum deceleration for leading vehicle (ft/s^2)	6.04	9.97	6.59
Reduction of 3.28 ft/s^2 deceleration per distance interval for leading vehicle (ft)	62.53	100.83	120.40
Accepted deceleration for leading vehicle (ft/s^2)	1.48	6.76	3.22
Maximum deceleration for following vehicle (ft/s^2)	5.09	4.10	3.54
Reduction of 3.28 ft/s^2 deceleration per distance interval for following vehicle (ft)	16.07	31.04	27.33
Accepted deceleration for following vehicle (ft/s)	4.49	2.92	0.95
Minimum headway (ft)	3.94	5.77	1.84
Safety distance reduction factor	0.14	0.20	0.23
Maximum deceleration for cooperative braking (ft/s)	11.35	11.81	13.75
Maximum speed difference for cooperative braking (ft/s)	15.99	12.62	10.04
Maximum collision time for cooperative braking (s)	7.22	6.80	9.25

Performance of DMC for Freeway-to-Freeway Merges

This section presents the DMC simulation sensitivity analysis results for freeway-to-freeway merge area scenarios. The freeway with heavier demand is called route A (i.e., route B has lighter demand than route A). In some of the experiments, route A had two mainline lanes. In other experiments, route A had three mainline lanes. In some of the experiments, lane closure was applied to route A. In other experiments, lane closure was applied to route B. Therefore, this section presents results for four combinations of mainline lane numbers and lane closure.

Lane Closure on Route B—Three Lanes Merging with Two Lanes

Traffic demands were varied on route A from 4,200 to 6,600 vehicles/h in increments of 300 vehicles/h and on route B from 1,000 to 2,400 vehicles/h in increments of 200 vehicles/h. Preliminary analysis indicated negative effects of DMC when traffic demands on route A were 5,700 vehicles/h or less. A possible reason could be that the total volume of routes A and B did not exceed capacity in the merging area. To identify accurate thresholds, traffic demands of 5,800, 5,900, 6,400, and 6,500 vehicles/h on route A were tested against the same range of demands on route B (1,000 to 2,400 vehicles/h).

Table 13 presents the average vehicle delay (AVD) savings. Mean differences were obtained by subtracting AVD under DMC conditions from the corresponding base conditions. A positive value indicates a reduction of AVD by implementing the DMC strategy.

Table 13. AVD saving by the DMC strategy (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h) 5,800	5,900	6,000	6,300	6,400	6,500
1,000	AVD (s)	0.09	0.15	0.00	0.78	2.47	4.89
	Percent change	+1	+2	+0	+9	+23	+36
1,200	AVD (s)	0.14	0.15	0.08	2.21	3.81	17.72
	Percent change	+2	+2	+1	+22	+31	+67
1,400	AVD (s)	0.26	0.46	0.26	6.06	12.69	38.42
	Percent change	+4	+6	+3	+42	+60	+80
1,600	AVD (s)	0.31	0.69	1.26	13.97	41.35	71.68
	Percent change	+4	+8	+14	+62	+82	+88
1,800	AVD (s)	0.7	1.3	2.49	47.15	77.35	96.92
	Percent change	+8	+14	+24	+84	+88	+88
2,000	AVD (s)	1.41	3.65	9.63	83.27	98.09	99.56
	Percent change	+15	+30	+52	+87	+86	+78
2,200	AVD (s)	4.13	10.45	26.82	93.92	94.3	80.98
	Percent change	+31	+52	+73	+81	+73	+59
2,400	AVD (s)	3.35	17.01	40.65	62.95	46.29	26.02
	Percent change	+15	+45	+60	+48	+33	+18

Lane Closure on Route A—Three Lanes Merging with Two Lanes

Traffic demands were varied on route A from 2,500 to 4,600 vehicles/h in increments of 300 vehicles/h and on route B from 3,000 to 4,600 vehicles/h in increments of 200 vehicles/h. Paired *t*-tests were conducted to determine whether AVDs under DMC significantly differed from base conditions. These *t*-tests were based on results of 15 simulation runs for base and DMC conditions. Results indicated that all delay differences between base and DMC conditions were statistically significant. Table 14 shows that when route B traffic demands exceeded 4,000 vehicles/h (around 2,000 vehicles/lane), closing a mainline lane reduced AVD regardless of demands on route A. The table also shows that benefits increased as traffic on route B increased.

Table 14. AVD savings by the DMC strategy (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)							
		2,500	**2,800**	**3,100**	**3,400**	**3,700**	**4,000**	**4,300**	**4,600**
3,000	AVD (s)	0.48	0.49	0.49	0.53	0.51	0.57	0.58	0.53
	Percent change	9	9	8	9	8	8	8	6
3,200	AVD (s)	0.78	0.82	0.87	0.9	1.01	1.14	1.29	2.06
	Percent change	13	13	14	13	14	15	15	21
3,400	AVD (s)	1.22	1.23	1.28	1.39	1.59	1.87	2.42	4.6
	Percent change	18	18	18	19	20	22	25	36
3,600	AVD (s)	1.71	1.88	1.91	2.72	2.43	2.98	5.49	21.38
	Percent change	23	24	24	30	27	30	42	72
3,800	AVD (s)	2.74	5.88	7	8.65	6.88	26.48	42.69	88.95
	Percent change	31	49	52	57	50	78	85	91
4,000	AVD (s)	24.76	44.54	35.04	59.16	69.06	83.5	99.48	111.18
	Percent change	80	87	84	90	91	92	93	93
4,200	AVD (s)	83.46	91.21	89.06	102.04	101.81	107.54	112.65	119.1
	Percent change	92	93	93	93	93	93	93	93
4,400	AVD (s)	119.45	115.46	114.49	111.48	114.15	113.93	115.11	121.88
	Percent change	94	94	94	93	93	93	93	93
4,600	AVD (s)	124.42	122.58	119.66	117.89	115.97	116.14	119.01	122.02
	Percent change	94	94	93	93	93	93	93	92

Lane Closure on Route A—Two Lanes Merging with Two Lanes

Traffic demands were varied on route A from 1,600 to 2,400 vehicles/h in increments of 200 vehicles/h and on route B from 3,000 to 4,600 vehicles/h in increments of 200 vehicles/h. Table 15 shows that when traffic demands on route B exceeded 3,800 vehicles/h (around 1,900 vehicles/lane), closing a mainline lane improved AVD, average speed, and total throughput regardless of route A traffic demands. The table also shows that benefits increased as route B traffic increased.

Table 15. AVD savings by the DMC strategy (mainline lane closure) (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)				
		1,600	1,800	2,000	2,200	2,400
3,000	AVD (s)	-0.97	-7.53	-22.91	-30.25	-75.96
	Percent change	-12	-89	-254	-314	-734
3,200	AVD (s)	-0.53	-6.78	-21.41	-28.53	-72.99
	Percent change	-6	-71	-209	-260	-607
3,400	AVD (s)	-0.12	-5.71	-19.82	-26.25	-67.75
	Percent change	-1	-51	-168	-204	-464
3,600	AVD (s)	0.47	-4.08	-16.41	-22.68	-45.61
	Percent change	4	-30	-107	-139	-127
3,800	AVD (s)	11.07	6.31	1.72	46.12	41.79
	Percent change	45	23	5	54	33
4,000	AVD (s)	62.89	92.37	110.01	155.25	188.15
	Percent change	63	72	66	73	65
4,200	AVD (s)	119.91	109.23	147.89	194.82	179.98
	Percent change	42	36	44	52	43
4,400	AVD (s)	115.84	134.42	126.34	170.36	192.92
	Percent change	26	29	27	33	35
4,600	AVD (s)	109.13	99.38	123.61	138.10	155.04
	Percent change	18	16	20	22	23

Lane Closure on Route B—Two Lanes Merging with Two Lanes

Traffic demands were varied on route A from 4,200 to 6,600 vehicles/h in increments of 300 vehicles/h and on route B from 1,000 to 2,400 vehicles/h in increments of 200 vehicles/h. Results in table 16 indicate negative (or insignificant) effects of DMC when route A traffic demands were 3,600 vehicles/h or less. A possible reason could be that the total volume of routes A and B did not exceed merge area capacity. When combined volumes exceeded 5,600 vehicles/h and route A demand exceeded 3,600 vehicles/h, DMC may improve operations. For the two lanes by two lanes case, the delay difference tended to show improvements only for mainline demands above 3,600 vehicles/h. The most significant gains were when both mainline and on-ramp demands were close to their capacities in optimal conditions.

Table 16. AVD saving by the DMC strategy for two-lane by two-lane ramp closure (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)							
		3,000	3,200	3,400	3,600	3,800	4,000	4,200	4,400
1,000	AVD (s)	-0.12	-0.09	-0.07	-0.02	0.02	0.13	0.30	1.94
	Percent change	-2	-1	-1	0	0	1	3	14
1,200	AVD (s)	-0.18	-0.12	-0.08	-0.01	0.18	0.22	2.27	7.01
	Percent change	-3	-2	-1	0	2	2	17	36
1,400	AVD (s)	-0.22	-0.16	-0.11	-0.04	0.21	0.59	4.35	34.36
	Percent change	-3	-2	-1	0	2	5	27	72
1,600	AVD (s)	-0.28	-0.20	-0.03	0.12	0.55	2.17	17.90	109.07
	Percent change	-4	-3	0	1	5	17	60	87
1,800	AVD (s)	-0.40	-0.26	-0.16	0.34	1.37	8.42	73.31	195.07
	Percent change	-5	-3	-2	3	12	42	83	87
2,000	AVD (s)	-0.65	-0.52	-0.13	0.70	3.94	28.09	139.57	202.11
	Percent change	-8	-6	-1	6	26	67	85	68
2,200	AVD (s)	-1.49	-1.18	-0.47	0.97	8.38	57.93	151.97	177.24
	Percent change	-17	-12	-4	7	37	71	70	50
2,400	AVD (s)	-9.54	-13.40	-12.50	-21.55	-26.56	29.53	95.95	106.44
	Percent change	-100	-123	-100	-125	-70	22	34	26

Performance of DMC for On-Ramp to Freeway Merges

It should be noted that ramp metering effects may be comparable to DMC in terms of managing ramp volumes and downstream mainline volumes. One-lane on-ramps merging into two-, three-, and four-lane freeways were investigated. Considering the complexity of signal-controlled on-ramp traffic, the current research is limited to continuous on-ramp traffic. The major differences for the on-ramp merge study herein are as follows:

- Only one-lane ramps were considered.
- On-ramp driving behavior and desired speed varied from the freeway to freeway merging.

One-Lane Ramp Merging with Two-Lane Freeway

Traffic demands were varied on the mainline from 1,000 to 2,400 vehicles/h in increments of 200 vehicles/h and on route B from 1,000 to 2,000 vehicles/h in increments of 200 vehicles/h. Paired *t*-tests indicate that the AVD differences for all considered traffic demands were statistically significant at the 95 percent confidence interval. Table 17 indicates that when either mainline or ramp traffic demands were lower than 1,400 vehicles/h, closing a mainline lane did not have clear effects on network performance regardless of demand on the on-ramp. This is reasonable because with 1,400 vehicles/h or less on the mainline, on-ramp demands between 1,000 and 2,000 vehicles/h would not exceed merge area capacity. Moreover, traffic from either road would not interact very much in the merge area. Results also show that closing a mainline lane would not benefit network performance until total demands exceeded 3,600 vehicles/h.

Table 17. AVD saving by the DMC strategy (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)							
		1,000	1,200	1,400	1,600	1,800	2,000	2,200	2,400
1,000	AVD (s)	0.44	0.46	0.59	0.8	0.79	0.86	0.62	-13.73
	Percent change	7	7	8	9	8	8	5	-88
1,200	AVD (s)	0.54	0.66	0.74	1	1.47	2.42	4.28	-5.74
	Percent change	8	9	9	11	14	19	26	-24
1,400	AVD (s)	0.59	0.75	1.06	1.44	2.39	5.43	12.32	10.86
	Percent change	8	9	12	14	20	33	49	28
1,600	AVD (s)	0.68	0.86	1.22	2.2	5.15	13.97	19.28	35.97
	Percent change	8	10	12	19	33	55	59	56
1,800	AVD (s)	0.75	0.94	1.44	4.1	8.85	19.33	34.6	44.57
	Percent change	8	10	13	29	44	61	71	61
2,000	AVD (s)	0.75	0.98	1.9	6.56	12.93	21.46	35.77	53.78
	Percent change	8	10	16	39	54	64	72	66

One-Lane Ramp Merging with Three-Lane Freeway

Traffic demands on route A were varied from 2,100 to 4,800 vehicles/h in increments of 300 vehicles/h and on route B from 1,000 to 2,000 vehicles/h in increments of 200 vehicles/h. The reason for the maximum 2,000 vehicles/h was because the on-ramp traffic was controlled by surface signals where capacity rarely exceeded 2,000 vehicles/lane. Paired *t*-tests show that reductions of AVD were statistically significant at the 95 percent confidence interval for all tested traffic demand combinations. Table 18 shows that the DMC control could save each vehicle as much as 89 s (88 percent).

Table 18. AVD savings by the DMC strategy (s).

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)									
		2,100	2,400	2,700	3,000	3,300	3,600	3,900	4,200	4,500	4,800
1,000	AVD (s)	0.68	1.09	1.45	2.47	5.20	9.41	12.20	13.14	15.45	24.25
	Percent change	+17	+22	+24	+33	+48	+59	+63	+62	+63	+68
1,200	AVD (s)	1.09	1.74	3.33	6.89	10.85	12.15	13.99	15.07	21.38	55.17
	Percent change	+23	+29	+41	+56	+64	+65	+65	+65	+70	+83
1,400	AVD (s)	1.65	3.16	7.20	11.34	12.72	13.65	15.36	19.07	32.09	86.23
	Percent change	+29	+41	+58	+67	+67	+66	+66	+69	+77	+88
1,600	AVD (s)	3.56	6.63	10.79	13.31	13.41	14.93	17.60	26.89	44.55	89.00
	Percent change	+44	+57	+66	+68	+67	+67	+68	+75	+82	+88
1,800	AVD (s)	5.32	10.96	12.27	13.90	14.56	16.18	21.97	31.67	47.28	87.55
	Percent change	+51	+67	+67	+68	+67	+67	+72	+77	+82	+87
2,000	AVD (s)	8.34	13.16	14.00	13.93	15.05	17.72	22.88	32.91	44.33	84.62
	Percent change	+62	+70	+70	+68	+67	+69	+72	+77	+81	+87

One-Lane Ramp Merging with Four-Lane Freeway

Traffic demands on route A were varied from 5,000 to 6,200 vehicles/h in increments of 400 vehicles/h and from 6,300 to 6,800 in increments of 100 vehicles/h. Demands on route B were varied from 1,200 to 1,800 vehicles/h in increments of 200 vehicles/h and from 1,900 to 2,200 vehicles/h in increments of 100 vehicles/h. The grid density was changed to perform detailed analysis on conditions near capacity. Table 19 shows DMC benefits under nearly all demand combinations. Vehicle delay savings can be as high as 95 percent.

Table 19. AVD (s) savings by the DMC strategy (four-lane freeway.

Route B Demand (vehicles/h)	Experimental Result	Route A Demand (vehicles/h)									
		5,000	**5,400**	**5,800**	**6,200**	**6,300**	**6,400**	**6,500**	**6,600**	**6,700**	**6,800**
1,200	AVD (s)	10.9	12.0	12.3	15.00	16.36	17.39	19.75	22.87	32.81	53.17
	Percent change	59	59	57	59	61	61	64	67	73	81
1,400	AVD (s)	11.8	13.2	14.2	18.42	22.97	31.14	36.96	48.44	84.55	113.3
	Percent change	60	60	60	63	68	74	76	80	87	90
1,600	AVD (s)	12.4	13.4	16.2	29.26	42.19	52.11	86.80	118.8	191.5	228.5
	Percent change	60	60	62	73	79	82	88	91	94	95
1,800	AVD (s)	13.5	14.9	21.5	62.94	99.75	146.4	186.7	218.9	267.4	333.7
	Percent change	58	58	65	83	89	92	93	94	95	96
1,900	AVD (s)	14.0	15.7	28.0	110.2	151.6	190.3	235.7	290.7	327.4	373.6
	Percent change	50	52	65	87	90	92	93	94	95	95
2,000	AVD (s)	14.3	18.2	48.6	141.1	172.6	235.2	265.4	308.8	363.2	397.9
	Percent change	25	30	54	77	81	85	87	88	90	91
2,100	AVD (s)	14.2	18.9	37.3	142.6	173.3	217.8	258.4	309.9	342.8	407.9
	Percent change	12	16	28	61	66	71	74	78	80	82
2,200	AVD (s)	13.3	17.5	42.9	133.8	179.5	214.8	263.1	310.2	345.4	395.4
	Percent change	7	10	22	48	55	60	65	69	71	74

B/C Analysis

In the scenarios, DMC benefits were sensitive to the combination of merging demand volumes and number of merging lanes. Under certain volume combinations, DMC control produced no benefit. Under other volume combinations, DMC control produced medium-to-high benefits. Therefore, it seems that a B/C analysis of DMC should at least evaluate the best-case scenarios while keeping in mind that only the "best" volume combinations would achieve those benefits. The best-case scenarios can be summarized as follows:

- Three-lane merge with two-lane freeway (lane closure at ramp) saved 100 s per vehicle.
- Three-lane merge with two-lane freeway (lane closure at mainline) saved 124 s per vehicle.
- Two-lane merge with two-lane freeway (lane closure at mainline) saved 193 s per vehicle.

- Two-lane merge with two-lane freeway (lane closure at ramp) saved 202 s per vehicle.

In the prior scenarios, route B vehicles experienced the most benefit, whereas route A vehicles remained mostly unaffected. For the three-lane merge with two-lane freeway (lane closure at ramp) scenario, average delay decreased by 100 s for 2,000 vehicles/h on route B. If 2,000 vehicles save an average of 100 s due to DMC treatment, total hourly time savings would be 55.56 vehicle-h, as shown in figure 59.

$$2{,}000 \text{ vehicles} \times 100 \text{ s} \times \frac{1 \text{ h}}{3{,}600 \text{ s}} = 55.56 \text{ vehicle-h}$$

Figure 59. Equation. Total hourly time savings for the DMC example.

Assuming the same \$17/h from the DLG B/C analysis, total daily cost savings would be \$945, as shown in figure 60.

$$55.56 \text{ vehicle-h} \times \frac{\$17}{1 \text{ h}} = \$945$$

Figure 60. Equation. Total daily time savings for the DMC example.

For one-way freeways, it seems appropriate to assume DMC cost savings would only materialize during one peak period per day. If the number of commuting days per year is derived by excluding weekends and holidays, then the annual cost savings attributed to DMC could be \$236,130 per year, as shown in figure 61.

$$\frac{\$945}{1 \text{ commuting day}} \times \frac{250 \text{ commuting days}}{1 \text{ year}} = \frac{\$236{,}130}{1 \text{ year}}$$

Figure 61. Equation. Annual cost savings for the DMC example.

Applying similar cost calculations to all four scenarios yields the following best-case results:

- Three-lane merge with two-lane freeway (lane closure at ramp) saves \$236,000 per year.
- Three-lane merge with two-lane freeway (lane closure at mainline) saves \$673,000 per year.
- Two-lane merge with two-lane freeway (lane closure at mainline) saves \$1,002,000 per year.
- Two-lane merge with two-lane freeway (lane closure at ramp) saves \$477,000 per year.

These annual benefits could be compared against implementation costs, including the cost of installing and maintaining a DMS on the freeway. It could also include the engineering analysis costs of screening the volume combinations (to identify candidate merge areas) plus law enforcement (if any) for the new lane use. Engineering analysis would be critical because the above cost savings would only materialize under certain demand volumes.

Regarding the cost of installing and maintaining a DMS, the USDOT's Knowledge Resources Web site implies the costs vary widely from \$10,000 to \$200,000.(52) Freeway DMS prices would likely be on the high end of this range, and a DMS tower (approximately $100,000) could also be required. Ideally, lifecycle maintenance and operation costs for the DMS would also be considered. A search for "DMC price" on the FHWA Web site leads to the information, from the USDOT ITS Benefits, Costs and Lessons Learned Database shown in table 9.

The 2014 *Greater Richmond Safety and Mobility Study* predicted $83,000 annual benefits from improved safety and $341,000 total annual costs for the I-64 corridor in Richmond, VA. The improved safety was due to fewer merging conflicts, producing an estimated 12 to 36 percent reduction in crashes. The annual costs included a lane control system, DMSs, and vehicle detection. Earlier it was stated that annual benefits could range from $236,000 to $1,002,000. VDOT assumes a 15-year lifespan for DMS signs. Assuming a total capital cost of $250,000, anticipated useful life of 15 years, and annual O&M costs of $6,000 per year, average annual cost and average annual benefits would be as follows:

- Average annual cost = ($250,000 capital/15 years) + $6,000 O&M = $22,666 per year.
- Average annual benefit (worst case) = $236,000 per year (B/C ratio of 10:1).
- Average annual benefit (best case) = $1,002,000 per year (B/C ratio of 44:1).

These are mobility-based B/C ratios not accounting for safety and environmental impacts.

Design Guidance for Safe and Efficient Implementation of DMC

Dynamic and/or time-of-day lane closures on freeways are typically indicated by overhead DMSs. For DMC, the DMS are nominally positioned upstream of the merge gore. Decision sight distances could be obtained from the American Association of State Highway and Transportation Officials (AASHTO) and are sensitive to freeway design speeds.(62) This allows mainline drivers sufficient time and distance in which to vacate the rightmost merging lane. An example of DMSs used for DMC is shown in figure 48. DMSs are located downstream of the merge gore in this figure, but upstream DMSs (which cause mainline drivers to vacate the rightmost merging lane) are a higher priority. If the DMC is implemented in a predictable time-of-day fashion, static regulatory signs should be used to show hours of operation. Driver compliance could conceivably be improved by an automated gate, as illustrated in figure 62 and figure 63, similar to what is used when closing high-occupancy vehicle (HOV) lane entrances during off-peak periods. This option could improve on-ramp merge operations by completely emptying the rightmost mainline lane. However, there would be a possibility of mainline drivers occasionally stopping at the gate if they were unable to change lanes on time.

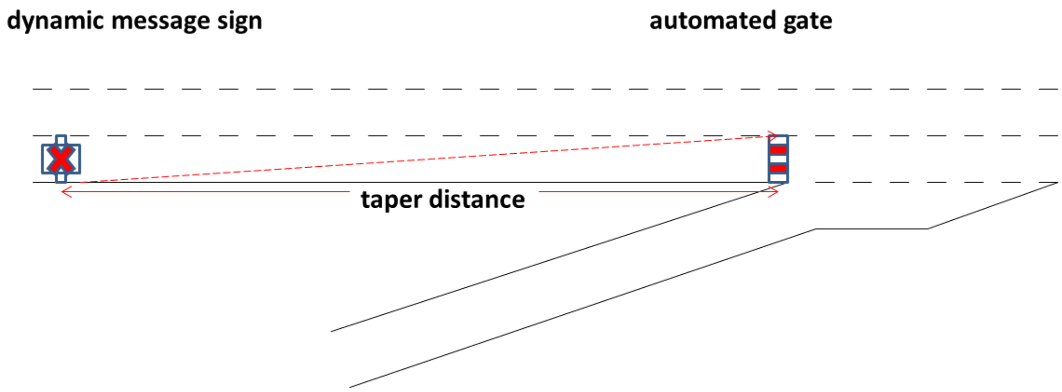


Figure 62. Illustration. Automated gate option for DMC.

Figure 63. Photo. Real-world image of an automated gate.[24]

Summary

The authors of this report investigated performance of the DMC strategy on a hypothetical road geometry involving a two-lane freeway merging with a two-lane freeway and dropping into four lanes. Traffic demands were varied on the minor road from 3,000 to 4,600 vehicles/h in increments of 200 vehicles/h and on the major road from 2,500 to 4,600 vehicles/h in increments of 300 vehicles/h for a total of 72 demand combinations. The major findings can be summarized as follows:

- Implementing the DMC strategy produced benefited at almost all demand combinations in terms of vehicle delays and speeds.
- DMC control reduced lane changes in the merge area, therefore increasing capacity and delaying the formation of bottlenecks.
- When traffic demand on the minor road reached 1,900 vehicles/h/lane, the benefits of applying DMC became both practically and statistically significant.

It is worthy to mention that a statistically significant improvement does not necessarily mean it is meaningful for practice. Agencies may set up their own criteria (e.g., average delay reduction of 10 percent or more) for justifying DMC implementation.

EXTENDING ACCELERATION LANE LENGTHS

On freeways, merging at the terminus of an acceleration lane often causes a severe traffic bottleneck. Acceleration lane lengths shorter than or equal to those recommended by AASHTO guidelines were tested to quantify delay changes when freeways of less-than-ideal design were upgraded.[(62)] In the Paramics™ simulations, a parallel design (see figure 64) consisting of acceleration lane length (see figure 65) and taper lane length was used. For a typical three-lane corridor, it was found that, on average, increasing acceleration lane length from 500 to 1,000 ft reduced delay by 14 percent, while increasing length from 1,000 to 1,500 ft (close to AASHTO recommendations) reduced delay by an additional 9 percent. Results were similar on a four-lane corridor with a parallel acceleration lane.

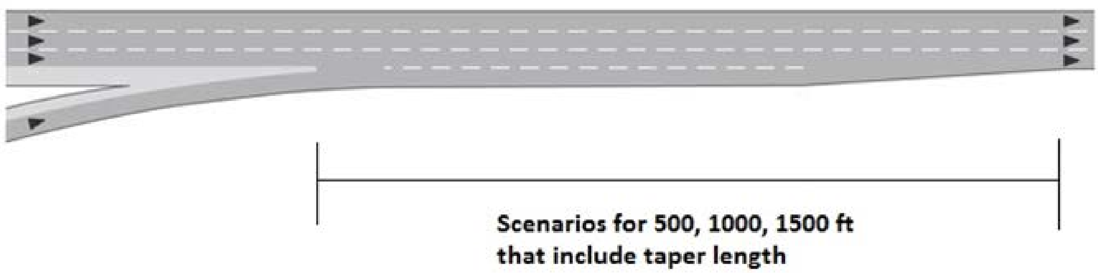


Figure 64. Illustration. Acceleration lane length schematic (three-lane configuration).

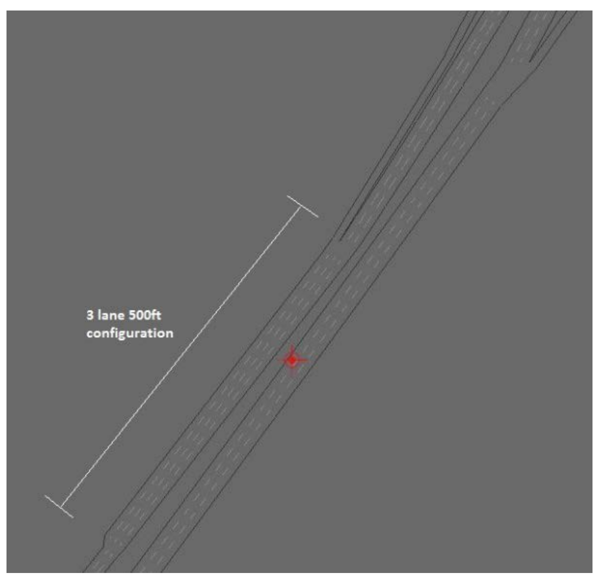


Figure 65. Illustration. Acceleration lane geometry.

In total, 12 geometric configurations were modeled (see table 20). Each of the geometric configurations was simulated under nine different demand scenarios (see table 21 and table 22). Based on engineering judgment, base condition demands were chosen to produce moderate congestion that would not extend beyond 1 mi upstream of the merge section. Specifically, two mainline configurations (three and four lanes), three acceleration lane lengths (500, 1,000, and 1,500 ft), and two ramp speeds were tested. In most cases, acceleration lengths were shorter than or approximately equal to recommended design lengths under AASHTO guidelines (i.e., exhibit 10-70 in the 2004 manual or table 10-3 in the 2011 manual) and are shown in the last column on table 20.[63,62]

All tabular simulation results shown were derived by aggregating detector readings. These detectors were placed at 500-ft intervals along the merge section and up to 1 mi upstream of the merge section to fully capture the spatial and temporal extent of delays. Detectors were also placed on the on-ramp at 500-ft intervals to capture on-ramp delays. Average speeds reported along the 1-mi corridor reflect average conditions up to 1 mi upstream of the merge section.

Table 20. Geometric configurations for testing acceleration lane lengths.

Configuration Number	Number of TH Lanes	Ramp Speed (mi/h)	Acceleration Length (ft)	Acceleration Length from the 2011 AASHTO Guideline[39] (ft)
1	3	40	500	780 + 300 taper
2	3	40	1,000	780 + 300 taper
3	3	40	1,500	780 + 300 taper
4	3	30	500	1,160 + 300 taper
5	3	30	1,000	1,160 + 300 taper
6	3	30	1,500	1,160 + 300 taper
7	4	40	500	780 + 300 taper
8	4	40	1,000	780 + 300 taper
9	4	40	1,500	780 + 300 taper
10	4	30	500	1,160 + 300 taper
11	4	30	1,000	1,160 + 300 taper
12	4	30	1,500	1,160 + 300 taper

Table 21 and table 22 present desired demands for each scenario under the three- and four-lane freeway configurations. The combination of geometric and demand scenarios presented in table 20 through table 22 resulted in 216 scenarios. Each scenario was run 10 times, and the average results are presented. Each geometric configuration was simulated with nine demand scenarios. On-ramp demands ranged from 578 to 1,978 vehicles/h/lane in the three-lane configuration and from 1,278 to 1,678 vehicles/h/lane in the four-lane configuration.

Table 21. Volume demands for testing acceleration lane lengths (three-lane configuration).

Demand Scenario	TH Demand (vehicles/h)	On Ramp Demand (vehicles/h)
1	4,847	1,278
2	5,047	1,278
3	5,547	778
4	4,147	1,578
5	3,547	1,978
6	5,797	578
7	5,347	1,078
8	4,547	1,428
9	4,047	1,678

Table 22. Volume demands for testing acceleration lane lengths (four-lane configuration).

Demand Scenario	TH Demand (vehicles/h)	On Ramp Demand (vehicles/h)
10	5,347	1,278
11	5,647	1,278
12	6,147	1,278
13	5,347	1,478
14	5,347	1,678
15	5,647	1,478
16	5,647	1,678
17	6,147	1,478
18	6,147	1,678

Figure 66 through figure 69 present total delay for each of the nine demand scenarios and for three different lane configurations. The red, green, and purple lines represent total delay for the 500-, 1,000-, and 1,500-ft acceleration lane lengths, respectively. As expected, the purple line (representing 1,500 ft) corresponds to the minimum amount of delay and has the lowest Y-coordinates. Where lines intersect, it means delays for different lane lengths were similar. This happens in simulations for demand scenarios 3 and 6, which had relatively low ramp volumes (below 800 vehicles/h/lane). For these two scenarios, the combination of ramp and mainline flow implies that increasing the acceleration length did not yield any reductions in delay.

The three-lane 40 mi/h configuration findings are as follows:

- Increasing acceleration lane length from 500 to 1,000 ft caused reductions in delay that varied based on demand pattern and were as high as 36 percent. The average delay reduction was 14 percent. Delay reductions greater than 15 percent were associated with demand scenarios 4, 5, 8, and 9, which had ramp volumes greater than 1,400 vehicles/h/lane.

- Further increasing the acceleration lane length from 1,000 to 1,500 ft also yielded significant reductions up to 19 percent, while the average was 9 percent. As expected, magnitude of the reduction was higher when going from 500 to 1,000 ft than when going from 1,000 to 1,500 ft. Again, the greatest delay reductions happened in scenarios where on-ramp flow exceeded 1,400 vehicles/h/lane.

The three-lane 30 mi/h configuration findings are as follows:

- Increasing acceleration length from 500 to 1,000 ft reduced delay up to 34 percent, with an average reduction of 15 percent. Higher reductions in delay were again associated with higher on-ramp flows.

- Further increasing the ramp length from 1,000 to 1,500 ft also reduced delay, but the reduction was smaller. The average reduction in delay was 12 percent, while the maximum was 18 percent.

The four-lane 40 mi/h configuration findings are as follows:

- For a ramp length increase from 500 to 1,000 ft, the average delay reduction was 14 percent.

- For a ramp length increase from 1,000 to 1,500 ft, the average delay reduction was 6 percent.

The four-lane 30 mi/h configuration findings are as follows:

- For a ramp length increase from 500 to 1,000 ft, the average delay reduction was 12 percent.

- For a ramp length increase from 1,000 to 1,500 ft, the average delay reduction was 6 percent.

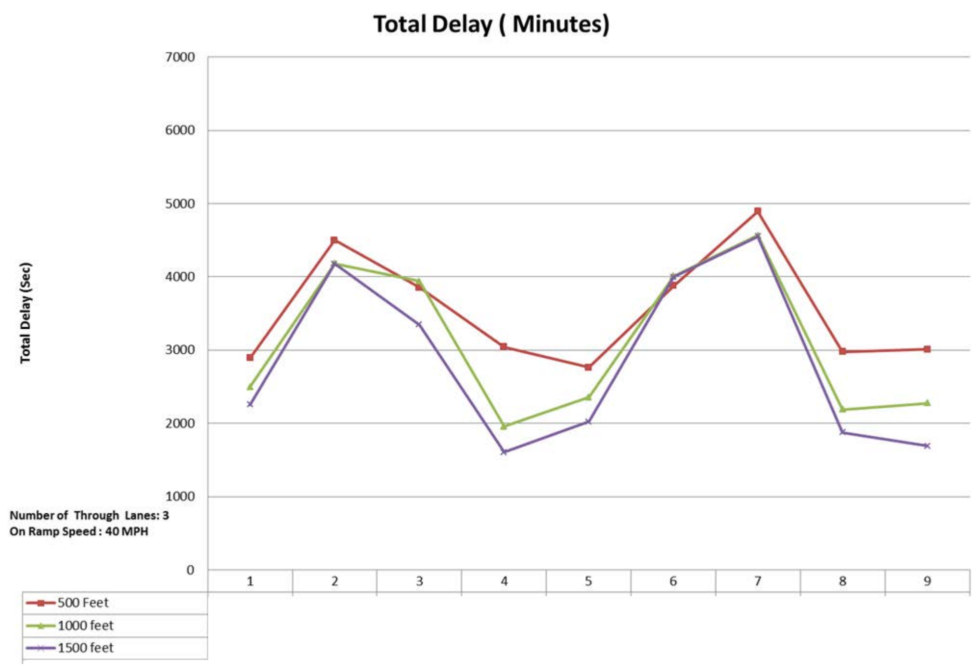


Figure 66. Graph. Total delay for three mainline lanes and 40-mi/h ramp speed.

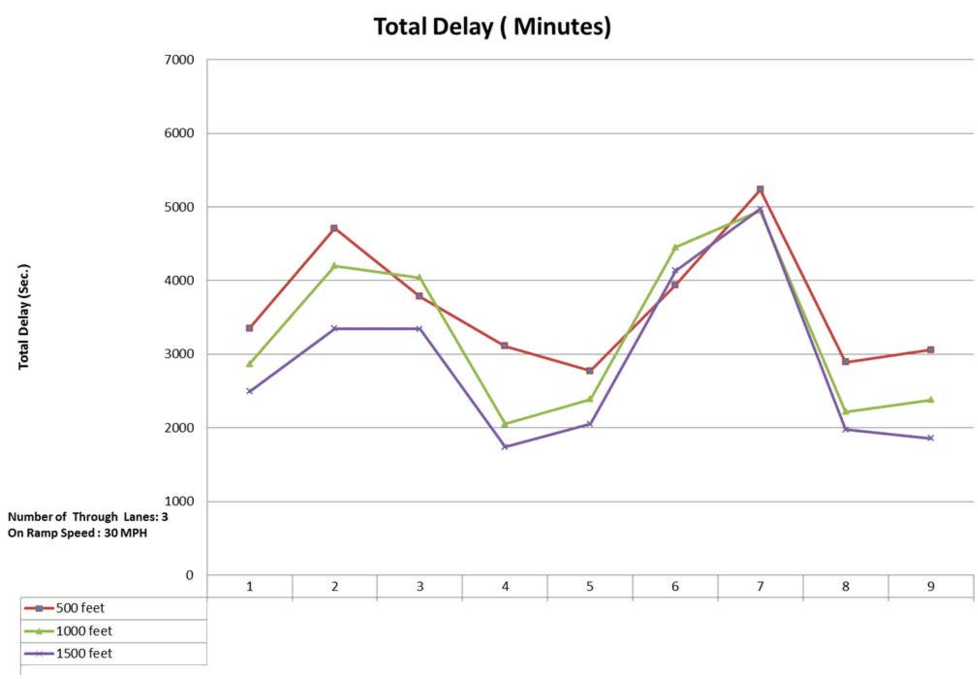


Figure 67. Graph. Total delay for three mainline lanes and 30-mi/h ramp speed.

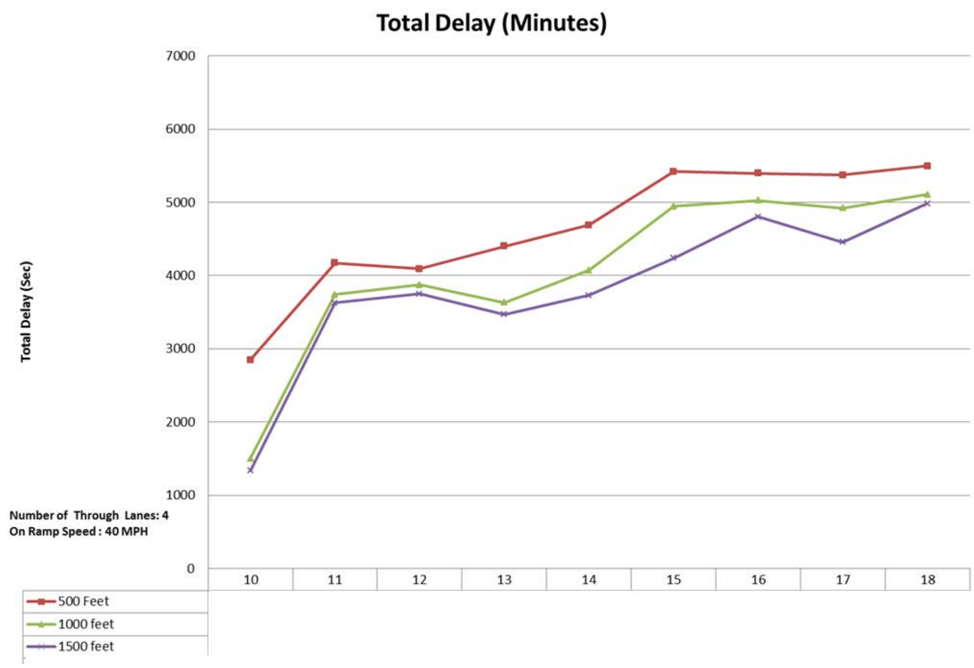


Figure 68. Graph. Total delay for four mainline lanes and 40-mi/h ramp speed.

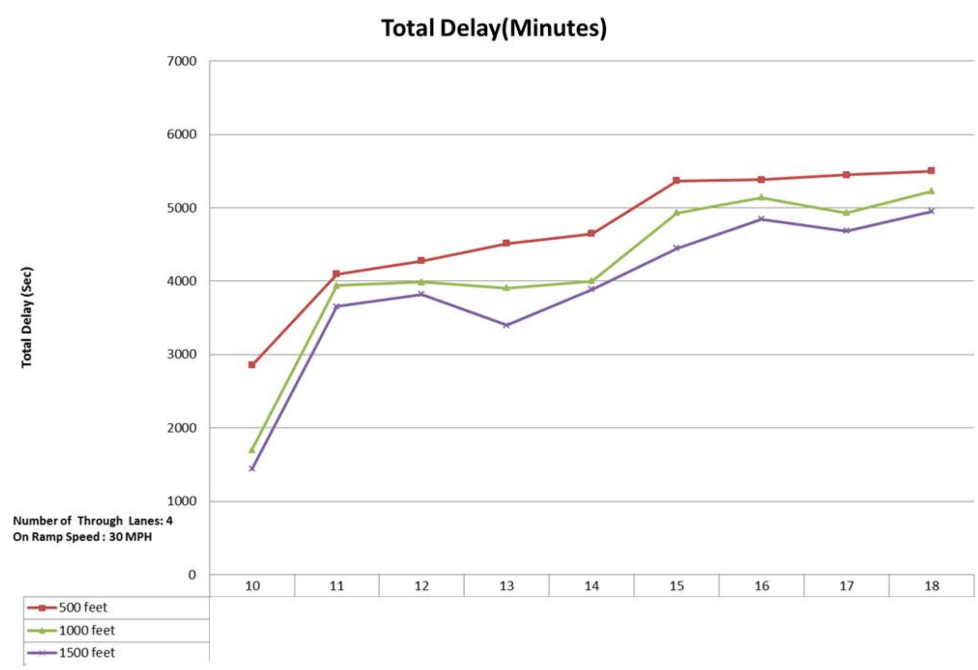


Figure 69. Graph. Total delay for four mainline lanes and 30-mi/h ramp speed.

B/C Analysis

According to the result tables, increasing acceleration lane length from 500 to 1,000 ft would save at least 395 min (i.e., 6.58 h) per day for the three-lane 40 mi/h configuration. Although additional vehicles beyond those simulated would likely benefit throughout the day, a conservative analysis can assume monetary benefits occur during the study period only. Assuming the same $17 per hour from the DLG B/C analysis, total daily cost savings would be $112, as shown in figure 70.

$$6.58\text{ h} \times \frac{\$17}{1\text{ h}} = \$112$$

Figure 70. Equation. Total daily time savings for extended acceleration lanes.

Adjusting for the number of commuting days, annual cost savings could be $27,965 per year, as shown in figure 71.

$$\frac{\$112}{1\text{ commuting day}} \times \frac{250\text{ commuting days}}{1\text{ year}} = \frac{\$27{,}965}{1\text{ year}}$$

Figure 71. Equation. Annual cost savings for extended acceleration lanes.

Increasing the acceleration lane length from 500 to 1,500 ft was shown to save 635 min (i.e., 10.58 h) per day, so this annual cost savings could increase to $44,965.

Table 23 illustrates the estimated cost savings for various scenarios.

Table 23. Estimated annual cost savings for increased acceleration lane length.

Scenario	Estimated Annual Cost Savings: 1,000-ft Acceleration Lane Length	Estimated Annual Cost Savings: 1,500-ft Acceleration Lane Length
Three lanes, 40 mi/h	$27,965	$44,965
Three lanes, 30 mi/h	$34,354	$61,058
Four lanes, 40 mi/h	$43,562	$67,717
Four lanes, 30 mi/h	$42,712	$78,838

Annual benefits between $45,000 and $79,000 for increasing from 500 to 1,500 ft, could be compared against construction costs. In addition, greater savings would be expected in scenarios of on-ramp flow exceeding 1400 vehicles/h/lane. According to TTI's mobility investment priorities, incorporating acceleration lanes by simple shoulder conversion can cost between $50,000 and $100,000 but can cost over $1 million for complex retrofits.(64,65) However, in 2013–2014, Arlington, TX, spent $640,000 to extend the Lamar Boulevard entrance ramp/acceleration lane on WB IH-30.(66)

The lifespan of a 1,000-ft acceleration lane area is uncertain but is expected to exceed the 15-year lifespan of DMSs. This analysis (1,000 ft added to the acceleration lane) assumes a useful life of 30 years and annual O&M costs of $1,200 per year.

When combining the worst-case benefits ($45,000 per year) with the worst-case costs ($34,500 per year), the B/C ratio of 13:10 implies that expected savings would exceed implementation costs by only 30 percent. However when combining the best-case benefits ($79,000 per year) with the best-case costs ($2,900 per year), the B/C ratio is 28:1. These are mobility-based B/C ratios not accounting for safety and environmental impacts.

HSR

The use of limited HSR for merge control when the complete hard shoulder lane is not opened for general traffic usage has the potential to provide better merge section operations. HSR requires extensive monitoring, servicing, maintenance, and incident reaction, as discussed in the B/C Analysis section. This section describes a number of HSR scenarios, which were studied by using the Paramics™ simulation software. HSR benefits were documented in terms of delay reductions. Results indicated delay reductions between 9 and 20 percent, depending on demand level, and propensity of drivers to use the HSR lane. This implies that agencies should consider dynamic use of hard shoulders for better merge performance without opening up the entire hard shoulder for general traffic usage. Figure 72 shows a schematic of the HSR configuration on I-80 between Atlantic Avenue and Douglas Boulevard in Roseville, CA.

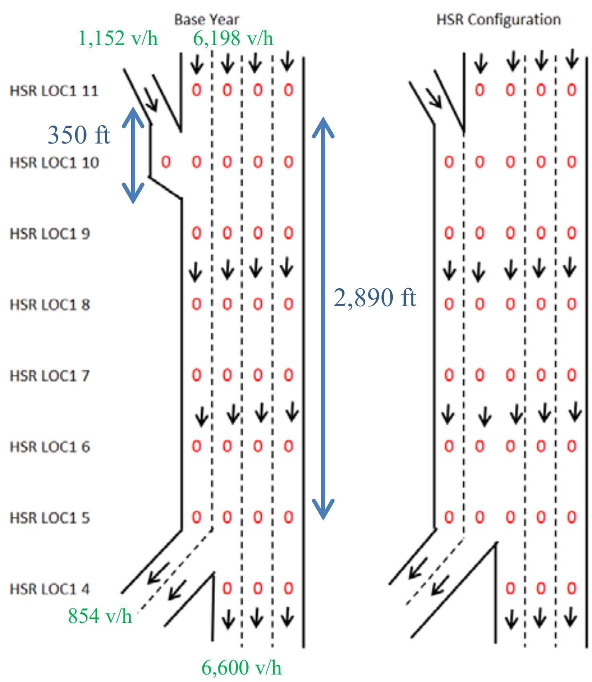


Figure 72. Illustration. HSR strategy schematic.

A series of scenarios were run to test the factors influencing facility performance. These factors include the following:

- **Demand**: Low, regular, or high.
- **HSR level of design**: Speed limit, lane width, and likelihood of use.

In total, 12 scenarios were analyzed: 4 with regular demand, 4 with low demand (95 percent of the regular demand), and 4 with high demand (105 percent of the regular). In table 24, non-HSR scenarios are bolded for easier comparison. Alternative scenarios were constructed by varying speed limit between 45 and 55 mi/h and by prohibiting 50 percent of drivers from entering the HSR lane, a scenario called “reduced likelihood of HSR use.”

In the model, all delays and speeds were reported for the hourly period between 7:30 and 8:30 a.m. Delay and speed calculations were based on detectors spaced at 500-ft intervals along a 1-mi-long corridor. Findings are as follows:

- Adding an HSR lane with a 55-mi/h speed limit reduced delay by 20 percent along the corridor. A 45-mi/h speed limit on the HSR reduced delay by 18 percent (scenarios 2 and 3 versus scenario 1).
- Even when a large percentage of drivers were unwilling to use the HSR lane, there was still a 13 percent reduction in total delay (scenario 4 versus scenario 1).
- HSR yielded similar reduction percentages under low and regular demands, but in absolute terms, delay reductions were lower under low demands.
- HSR yielded diminishing returns (benefits) under high demands.

Table 24. HSR summary results.

Simulation Number	Demand	HSR Type	Total Delays (min)	Average Speed (mi/h)	On-Ramp Volume (vehicles/h)	Upstream Volume (vehicles/h)	Downstream Volume (vehicles/h)
1	**Base**	**No HSR**	**8,186**	**35.0**	**1,156**	**6,902**	**7,380**
2	Base	55 mi/h HSR lane	6,522	39.4	1,159	6,889	7,369
3	Base	45 mi/h HSR lane	6,725	38.7	1,158	6,893	7,369
4	Base	Reduced likelihood of HSR use	7,081	37.8	1,156	6,904	7,381
5	**Low**	**No HSR**	**5,779**	**40.8**	**1,072**	**6,560**	**7,012**
6	Low	55 mi/h HSR lane	4,652	44.9	1,068	6,553	7,034
7	Low	45 mi/h HSR lane	4,677	44.8	1,069	6,549	7,033
8	Low	Reduced likelihood of HSR use	5,293	42.5	1,066	6,567	7,021
9	**High**	**No HSR**	**13,492**	**27.6**	**1,217**	**7,146**	**7,633**
10	High	55 mi/h HSR lane	11,948	28.3	1,225	7,105	7,638
11	High	45 mi/h HSR lane	12,187	28.0	1,226	7,126	7,640
12	High	Reduced likelihood of HSR use	11,526	29.2	1,221	7,185	7,695

Note: Bolded text indicates base case condition (without HSR).

Table 25. HSR delay reductions.

HSR Type	Base Total Delay (min)	Base Difference from No HSR (percent)	Low Demand Total Delay (min)	Low Demand Difference from No HSR (percent)	High Demand Total Delay (min)	High Demand Difference from No HSR (percent)
No HSR	8,186	0	5,779	0	13,492	0
55-mi/h HSR lane	6,522	-20.3	4,652	-19.5	11,948	-11.4
45-mi/h HSR lane	6,725	-17.8	4,677	-19.1	12,187	-9.7
Reduced likelihood of HSR use	7,081	-13.5	5,293	-8.4	11,526	-14.6

B/C Analysis

Table 26 illustrates the possible cost savings from HSR based on delay reductions from table 25. These savings assume the same $17/h and 250 commuting days per year from the DLG B/C analysis. These savings also assume 3 h of HSR benefits per day.

Table 26. Possible cost savings under various scenarios of HSR.

Simulation Number	Demand	HSR Type	Total Delay (h)	Annual Savings (dollars)
1	Base	No HSR	136.4	0
2	Base	55 mi/h HSR lane	108.7	353,100
3	Base	45 mi/h HSR lane	112.1	309,900
4	Base	Reduced likelihood of HSR use	118.0	234,600
5	Low	No HSR	96.3	0
6	Low	55 mi/h HSR lane	77.5	239,700
7	Low	45 mi/h HSR lane	78.0	233,400
8	Low	Reduced likelihood of HSR use	88.2	103,200
9	High	No HSR	224.9	0
10	High	55 mi/h HSR lane	199.1	329,100
11	High	45 mi/h HSR lane	203.1	278,100
12	High	Reduced likelihood of HSR use	192.1	418,200

These annual benefits would presumably be compared against construction and maintenance costs. Opening hard shoulders for limited times of operation often requires a DMS. During the earlier B/C analysis for DMC, it was seen that freeway DMS costs can exceed $200,000. Moreover, HSR requires extensive monitoring, servicing, maintenance, and incident reaction.(67) An extensive camera system is recommended "to ensure the shoulder is clear prior to opening."(67) Increased monitoring tends to bring increased service and maintenance costs. Finally, hard shoulder use by passenger cars adds new challenges for incident response. HSR could potentially add new costs associated with managing such incidents and controlling emergency vehicle access. The use of emergency pullouts at half-mile spacings is encouraged to provide refuge areas that could accommodate many of the disabled situations. Some areas have invested in service patrols, staged tow trucks, and other traffic incident management enhancements.

According to the Texas A&M toolbox for adding capacity, converting a shoulder to a shoulder lane costs $1.5 million for construction plus engineering and $12,000 per year for maintenance.[(68)] The toolbox further projects a B/C ratio of roughly 7:1.

The lifespan of a hard shoulder is uncertain, but this analysis assumed a useful life of 30 years. When combining the worst-case benefits under low demand volumes ($103,000 each year) with the worst-case costs ($1.5 million start-up and $12,000 annual maintenance), the B/C ratio of 1.7:1 implies that benefits would exceed costs by only 70 percent. However, when combining the best-case benefits under high demand volumes ($418,000 each year) with the best-case costs ($250,000 start-up and $6,000 annual maintenance), the B/C ratio is a much more favorable 29:1. These are mobility-based B/C ratios not accounting for safety and environmental impacts.

REDUCING EXISTING LANE AND SHOULDER WIDTHS TO ADD NEW LANES

In congested freeways with four lanes or more, reducing lane widths to 10 ft can create space for an additional lane. This can often be done without any construction or requisition of additional space. According to simulation results, the additional capacity obtained by adding a lane was effective at offsetting the reduced speeds caused by narrow lanes, eliminating bottlenecks, and producing higher corridor speeds. Specifically, reducing lane widths from 12 to 10 ft on the four-lane corridor and adding an additional 10-ft lane reduced overall corridor delay by 21 percent. Real-world implementations of 10-ft freeway lanes have occurred in California and Hawaii, as discussed later in the B/C Analysis section. Other reduced-width (11-ft) lanes were created on the I-75 and I-85 corridors in Georgia inside the I-285 perimeter. They were installed in combination with requirements that through trucks must bypass downtown Atlanta, GA, by using I-285. To assess operational benefits, a four-lane freeway segment was selected in Buffalo NY. This is four-lane facility has frequent on- and off-ramps whose mainline width is 48 ft (4 by 12 ft). The base year configuration was extended to four lanes along segments in which the roadway dropped to three lanes. In the base scenario shown in figure 73, a primary source of congestion was the friction caused by vehicles using various freeway ramps (I-290), especially near the on- and off-ramps (e.g., detector location 175 in figure 73).

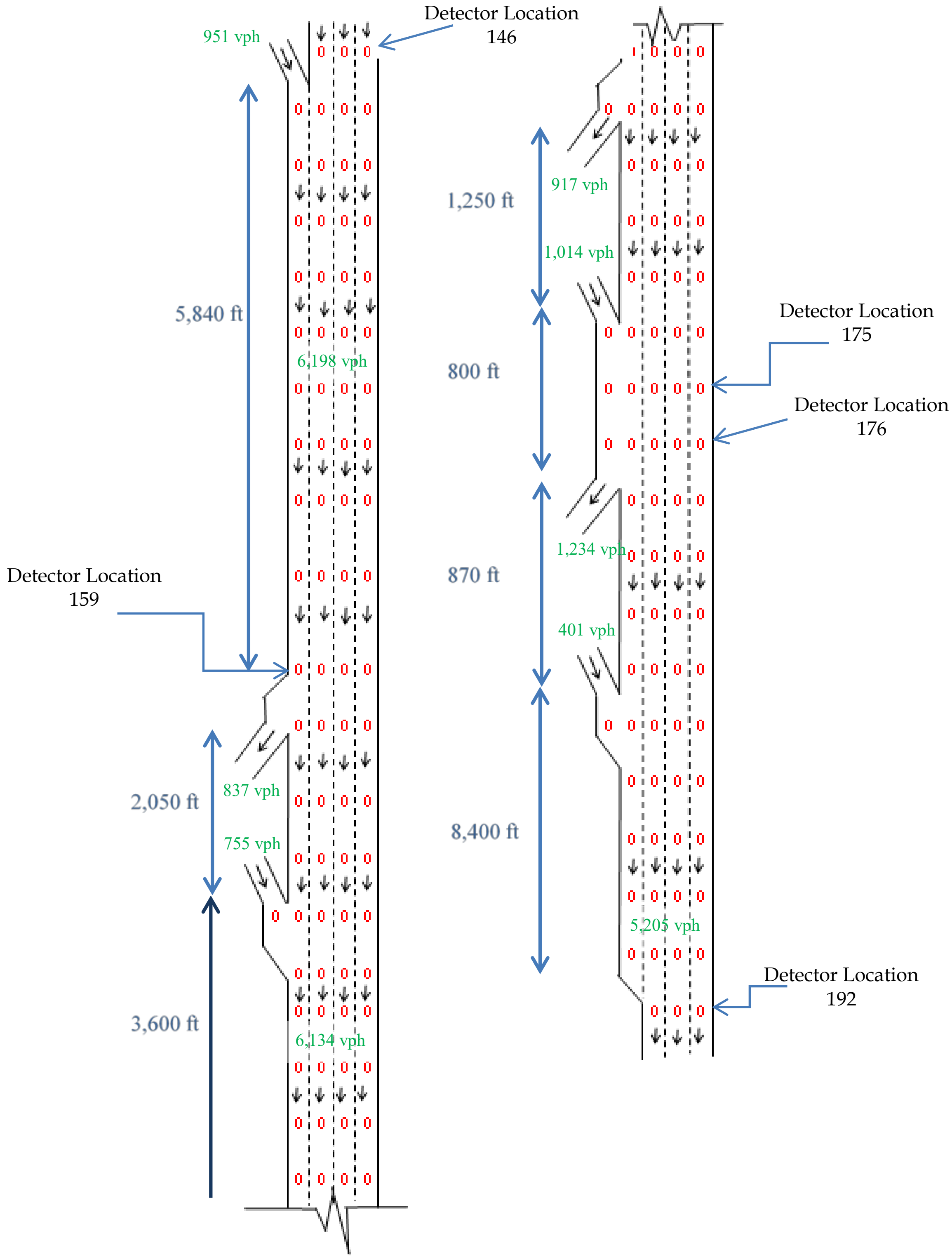


Figure 73. Illustration. Base year analysis of reducing lane widths to add new lanes.

As shown in figure 74, adding a fifth new lane and then reducing all lane widths to 10 ft ensured that the total roadway width differed only by 2 ft (48 versus 50 ft). It has been observed and documented in the 2010 HCM that drivers reduce their speed when driving on lanes narrower than 12 ft.[6] Specifically, it is expected that drivers reduce their speed by an average of 7 mi/h when driving on a 10-ft lane (see figure 75). Results indicate that speed reductions caused by lane-narrowing were offset by the additional capacity produced by an additional lane. These results are summarized in table 27 and table 28. In addition to the reduced lane-width scenarios, researchers also modeled a five-lane freeway with 12-ft lane widths to investigate the impact of building additional roadway as opposed to redistributing the existing roadway. In this experiment, demand was loaded from 5:30 to 9 a.m., while simulation results were collected between 6 and 9 a.m. Detectors were placed at 500-ft intervals along the corridor's 4.7-mi length to obtain speed and flow readings. Free-flow speed in the corridor was 70 mi/h.

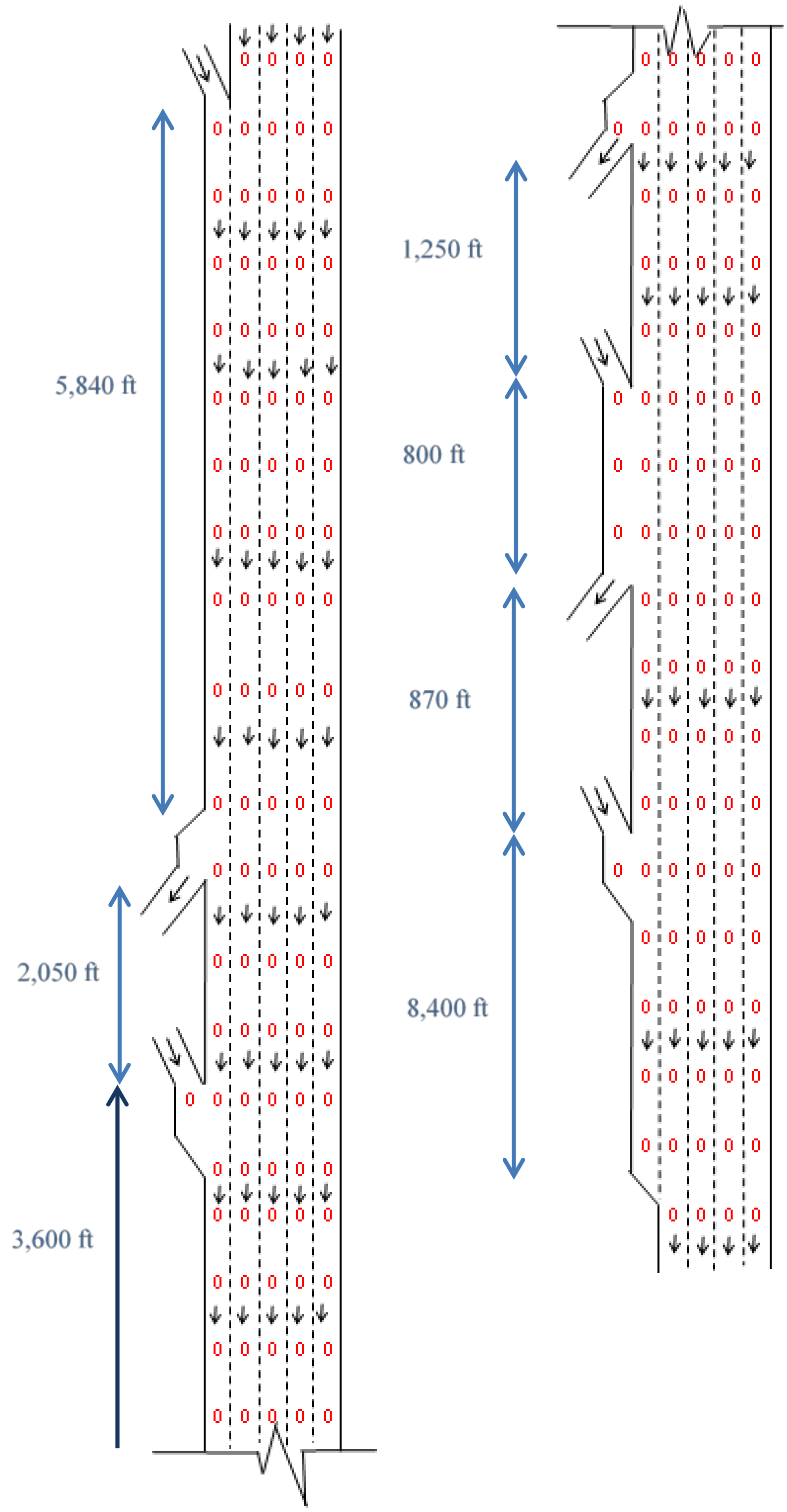


Figure 74. Illustration. Equivalent freeway scenario with five mainline lanes.

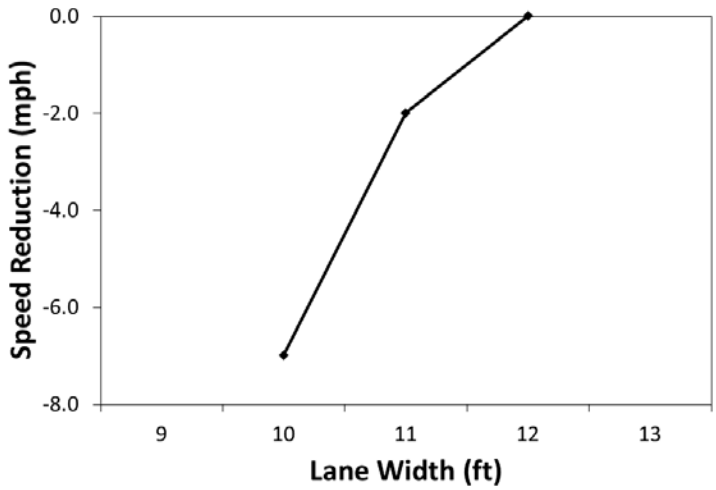


Figure 75. Graph. Speed reduction as a function of lane width.

Table 27. Summary results of reducing lane widths to add new lanes.

Scenario	Description	Total Delay from 6–9 a.m. (vehicle-min)	Reduction from Base Percent
Base	Modified existing roadway	50,715	0
1	One additional 12-ft lane	28,741	43
2	One additional 12-ft lane plus a 20-min incident on lane 5 starting at 8 a.m.	60,241	-19
3	One additional lane added by reducing all lane widths to 10 ft	39,899	21
4	One additional lane added by reducing all lane widths to 10 ft. A 30-min incident occurred on lanes 4 and 5 starting at 8 a.m.	68,019	-34

Table 28. Additional results from reducing lane widths to add new lanes.

Scenario	Mean Speed (mi/h)	Upstream Volume (vehicles/h)	Downstream Volume (vehicles/h)
Base	41	17,274	16,814
1	50	17,880	16,810
2	39	17,564	16,520
3	46	17,821	16,893
4	36	17,627	16,384

Table 27 summarizes delay results from the Paramics™ simulation environment, while table 28 summarizes the average speeds and throughput. The base year configuration was moderately congested, with an average speed of 41 mi/h between 6 and 9 a.m. Scenarios 1 and 2 involve maintaining existing lane widths and adding one more lane that is 12 ft wide to examine the benefits of new roadway capacity. In scenario 1, total vehicle delay dropped from 50,715 to 28,741 vehicle-min. This implies a 42 percent reduction in delay if one more 12-ft lane was constructed. Such reduction happens because the bottleneck from detector 176 to 159 in the base year was eliminated in scenario 1. A 30-min incident blocking the rightmost two lanes (4 and 5) resulted in 60,241 vehicle-min of delay (scenario 2), which is higher than the base year delay of 50,715 vehicle-min.

Scenarios 3 and 4 involved a five-lane freeway but with reduced lane widths that avoided any construction. Scenario 3 converted existing lanes to 10 ft and added a fifth lane, reducing delay to 39,899 vehicle-min. This is a 21 percent reduction, which is half the reduction obtained by adding new roadway capacity. For scenario 3, average speed over the 3-h period from 6 to 9 a.m. was 46 mi/h as opposed to 41 mi/h in the base year. However, speed contour plots (not shown) indicated that the scenario 3 speed was uniform, rarely dropping below 30 mi/h throughout most of the corridor. In fact, the main bottleneck along the corridor spanning from detector 156 to 176 disappeared. Although there were some low speeds at the end, these were due to capacity reductions outside the modeling area, and were carried over across all scenarios.

Scenario 4 copied the 10-ft configuration from scenario 3 and added a 30-min two-lane incident at 8 a.m. This increased delay from 39,899 vehicle-min in scenario 3 to 68,109 vehicle-min, a 41 percent increase. In both incident scenarios, delay was higher than the incident-free base year even though the incidents lasted for only 30 min. As expected, an incident on the 10-ft freeway (scenario 4) had a greater delay than the same incident on a 12-ft freeway (scenario 2). Table 28 contains information on the mean corridor speed and on volumes upstream and downstream of the simulation corridor (detectors 146 and 192 in figure 73).

B/C Analysis

According to the simulation results, increasing the number of lanes from four to five lanes (with narrower lane widths) would save 39,899 min (i.e., 665 h) during the 3-hour peak travel period. Although additional time savings would likely occur throughout the day, a conservative analysis can assume monetary benefits occur during the 3-hour peak period only. Assuming the same $17 per hour and 250 commuting days from the DLG B/C analysis, annual cost savings would be $2.83 million.

Cost information on restriping all mainline lanes to create a new lane without actually building a new lane has been relatively difficult to locate. Moreover, the cost of rerouting traffic is unclear. Unlike the other bottleneck mitigation strategies, lane narrowing for the purpose of adding a lane would require diverting each mainline lane at some point, causing greater congestion costs during construction. The *Honolulu Star Advertiser* cited a $200,000 project in 2012 to add a fourth lane on H-1 in Hawaii.[69] This restriping work was performed only during nighttime hours. However, in 1985, the *Los Angeles Times* predicted an $800,000 cost to increase lanes (via narrower lane widths) on a short section of Ventura Freeway.[70]

The lifespan of a fifth lane is uncertain, but this analysis assumed a useful life of 30 years. Assuming O&M costs of $6,000/year/lane-mi, and given that the Buffalo, NY, simulation took place over 22,810 ft (4.3 mi) as shown in figure 74, O&M costs were estimated at 6,000 × 4.3 = $25.9,000/year/lane-mi. Assuming construction and traffic diversion costs of $2 million, the estimated B/C ratio is 30:1. The average annual cost and average annual benefit are as follows:

- Average annual cost = ($2 million capital/30 years) + $25,920 O&M = $92,587 per year.
- Average annual benefit = $2,830,000 per year (B/C ratio of 30:1).

This is a mobility-based B/C ratio not accounting for safety and environmental impacts.

SUMMARY

In recent years, bottleneck mitigation has been attempted through strategies such as ramp metering and VSLs. This section presented research on the following five featured mitigation strategies, which are expected to be cost effective:

- DLG (arterials).
- DMC (freeways).
- Changing acceleration lane lengths (freeways).
- HSR (freeways).
- Reducing lane widths to add new lanes (freeways).

At arterial intersections, the travel demand may vary significantly by time, and the lane assignments, designed for the most severe peak hour traffic, might not suit these different demand patterns. DLG can reduce the gap between supply and demand by changing lane assignments dynamically. In the two case studies, DLG reduced overall intersection delay by up to 30 percent. Screening criteria can be changed to fit local agency needs. Estimated DLG B/C ratios ranged between 7:1 for low-volume intersections and 29:1 for high-volume intersections. These ratios also assume that DLG is only applied to intersections flagged as good candidates for DLG treatment according to the screening process.

Microscopic simulations of DMC of two-lane freeways merging with three-lane freeways and dropping into four lanes produced the following findings:

- When traffic demand on the minor freeway reached 1,900 vehicles/h/lane, benefits of applying DMC became significant.
- DMC reduced lane changes in the merge area, therefore increasing capacity and delaying the formation of bottlenecks.
- Estimated DMC B/C ratios ranged between 10:1 and 44:1.

On freeways, merging at the terminus of an acceleration lane often caused a severe traffic bottleneck. In microscopic simulations of a typical three-lane corridor, increasing acceleration lane length from 500 to 1,000 ft reduced delay by 14 percent, while increasing the length from

1,000 to 1,500 ft (close to AASHTO recommendations) reduced delay by an additional 9 percent. Results were similar on a four-lane corridor with a parallel acceleration lane. When increasing to 1,500 ft, estimated B/C ratios ranged between 13:10 and 28:1. B/C ratios were less favorable when increasing to only 1,000 ft.

The use of limited HSR for merge control when the complete hard shoulder lane is not opened for general traffic usage has the potential to provide better merge section operations. Results indicated delay reductions between 9 and 20 percent depending on demand level and drivers' propensity to use the HSR lane. This implies that agencies should consider dynamic use of hard shoulders for better merge performance without opening up the entire hard shoulder for general traffic usage. Under high-demand volumes, estimated B/C ratios ranged between 1.7:1 and 29:1. B/C ratios were less favorable under low demand volumes.

In congested freeways with four lanes or more, reducing lane widths to 10 ft can create the space needed to add an additional lane. This can often be done without any construction or requisition of additional space. According to simulation results, the additional capacity obtained by adding a lane was effective at offsetting the reduced speeds caused by narrow lanes, eliminating bottlenecks, and producing higher corridor speeds. Specifically, reducing lane widths from 12 to 10 ft on the four-lane corridor and adding an additional 10-ft lane reduced overall corridor delay by 21 percent. B/C ratios were estimated at 30:1.

In addition to the five featured strategies, the following alternative intersection and interchange designs are recommended:

- RCUT.
- MUT.
- DLT and DLTI.
- DDI.
- Roundabout.

For more information, refer to FHWA's AIIR and *Alternative Intersection Design* Web site.[47]

RECOMMENDATIONS FOR FUTURE RESEARCH

Regarding the automated methods for CBI, further model enhancements are sought for modeling traffic speed with more weather information, including wind direction and speed. The model will be validated using other data sets from other roadways. Another future enhancement of the model is replacing the symmetric normal distributions with nonsymmetric distributions, such as log-normal or gamma distributions.

DLG aims to adjust lane assignment based on changing demand to further improve intersection capacity utilization. Future software tools should automatically forecast delay reductions and other improvements to the performance measures at all locations flagged as DLG candidates. If such tools were available, subsequent sensitivity analyses could theoretically determine optimal threshold values for the screening criteria.

DMC is one of the major DJC strategies. The research presented was constrained to a specific set of geometric and traffic conditions, and the study was solely based on simulation work. Although DMC has not been implemented in the United States, field experiments are desired to validate the proposed strategy. Moreover, the current study closed a lane approximately 1,000 ft upstream of the merge gore and informed drivers 2,500 ft upstream. The next phase of this study will investigate varying lane closing locations and varying distances to inform drivers. Furthermore, the current research assumes a 100 percent compliance ratio when closing a lane. In the next phase, various compliance ratios will be evaluated.

The simulation of mitigation strategies showed beneficial impacts for a limited set of geometries. Some of these analyses could be expanded to include different geometry and flow patterns. The nationwide bottleneck analyses have revealed geometries of some of the most representative and problematic merge, weave, or diverge sections. These problematic geometries could be examined in future research.

Another important subject to investigate is the impact of autonomous or semi-autonomous vehicles on traffic flow. Limited research on this topic in the last 5 years indicates a beneficial impact of even a small percentage of semi-autonomous vehicles on traffic flow. However, the impact of various cooperative and autonomous driving strategies should be thoroughly investigated to understand how much each strategy benefits individual vehicles and the traffic stream.

CHAPTER 5. INNOVATIVE SOLUTIONS FOR BOTTLENECK MITIGATION

The cost-effective bottleneck mitigation strategies described in chapter 4 have already been implemented in many real-world locations. This project produced research on the following three strategies, which are not known to have been deployed in any U.S. locations:

- Dynamic HSR (freeways).
- DRLT lanes (surface-freeway interchanges).
- Contraflow LT (CLT) pockets (signalized intersections).

Some of the simulation datasets used to evaluate these strategies can be obtained by contacting the STOL at TFHRC and referencing the bottleneck project (i.e., STOL task order 6).

DYNAMIC HSR

Because of its success in treating recurrent congestion, it is natural to contemplate the possible effectiveness of HSR versus nonrecurrent congestion. For example, after an incident occurs, lane blockages may produce severe bottlenecks. In this situation, opening some section of the hard shoulder could be an effective way to increase capacity and help vehicles bypass the incident. This section proposes the concept of dynamic HSR as a traffic incident management strategy.[71] A dynamic HSR strategy can be explained using the example in figure 76, which illustrates a three-lane freeway segment and one hard shoulder lane on the far right. When an incident blocks the rightmost mainline lane, a section of the shoulder could be opened temporarily. This would allow vehicles to bypass the incident, thus avoiding the formation of excessive traffic congestion. Opening or closing shoulder sections could be accomplished preferably by smartphones, pushed auditory messages, or overhead gantries.

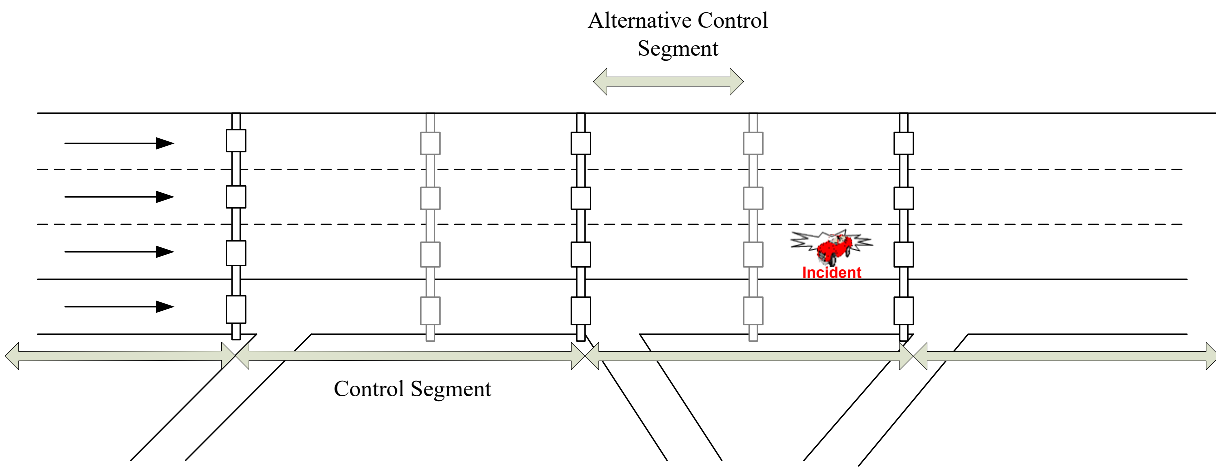


Figure 76. Illustration. Dynamic HSR for incident response.

Incident Types

Incidents are categorized as fatal and injury (FI) and property damage only (PDO). During an FI incident, more field operations are needed. Emergency vehicles tend to block more lanes (usually including incident lanes, lanes next to incident lanes, and shoulder lanes), and shoulders cannot be used after their arrival. Also, it is critical for emergency vehicles to arrive on time. Shoulders are usually reserved for these vehicles so that they can arrive as soon as possible. Therefore, dynamic HSR strategies are not recommended for FI incidents. For minor PDO incidents only, police arrive at the scene and push blocking vehicles to leave the highway and drive to the shoulder if vehicles do not voluntarily drive away or cannot leave the mainline. Traffic management centers usually have more flexibility in managing PDO incidents; thus, dynamic HSR can be more efficient for these types of incidents.

In general, dynamic HSR is useful in the following PDO scenarios:

- Incident vehicles can drive away or be towed away completely. Shoulders can be opened immediately upon incident detection, and remain open even after the incident is cleared.

- Incident vehicles can drive or be towed to the shoulder to prevent secondary incidents. Shoulders can be opened after incident detection but closed before the shoulder becomes occupied by incident vehicles. It is possible to reopen the shoulder after the shoulder is cleared.

Simulation Experiments

Experiments were conducted using VISSIM™ 6.0 microscopic traffic flow simulation tool. Incidents were modeled by adding two vehicles with a speed of 0 mi/h to the incident location at the incident occurrence time. This was realized through the VISSIM™ component object model interface. After the incident clearance time (TCL), the two vehicles were removed through the same interface. Shoulder lanes and general purpose lanes were assumed to be 11 and 12 ft wide, respectively. Based on the 2010 HCM procedures for basic freeway segments, lane widths affect the free flow speed according to the equation in figure 77.[6]

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22TRD^{0.84}$$

Figure 77. Equation. Free-flow speed from lane width and lateral clearance.

Where:

FFS = Free-flow speed (mi/h).
f_{LW} = Speed adjustment for lane width (mi/h).
f_{LC} = Speed adjustment for lateral clearance (mi/h).
TRD = Total ramp density (ramps/mi).

In order to model rubbernecking effects within a VISSIM™ three-lane section, Hadi et al. recommended the application of 20-mi/h speeds in adjacent lanes.[72] According to the 2000 HCM, this speed produces typical incident period capacities.[73] To capture rubbernecking effects on freeway segments with more than four lanes in one direction, Miller-Hooks also suggested the use of lane-specific speed reduction values.(74) Thus for these experiments, reduced speed areas in VISSIM™ were added 150 ft upstream of the incident.

The existence of ramps could be another key factor affecting dynamic HSR effectiveness. Because the excessive number of possible ramp-deployment scenarios could not be explicitly evaluated by microscopic simulation, this study considered ramp effects through the ramp-density concept in figure 77. The simulation experiments assumed a total ramp density of two ramps per mile and assumed typical real-world lateral clearances producing speed adjustment factors equal to zero.

In addition to the assumptions related to free-flow speeds and rubbernecking speeds, the dynamic HSR simulations required a second set of logistical assumptions. First, it was assumed possible to seamlessly and flexibly close or open any length of shoulder, upstream or downstream of the incidents, at any location. In practice, the necessary driver notification could be only possible through smartphones and connected vehicles. While the assumptions might apply to smartphone use, if overhead gantries were the sole source of driver notification, this would constrain shoulder length closures to the spacing of these gantries (typically 0.5 to 1 mi). For incidents occurring directly under gantries, the choice of which freeway sections to close would be clear. For incidents between gantries, traffic operations engineers can decide an opening strategy based on recommended values in this study. The second logistical assumption was that incidents occur on lanes closest to the shoulder. In an incident with two lanes blocked, the two lanes closest to the shoulder were assumed to be involved. This assumption was made to reduce the number of simulation runs to a manageable level.

Experimental Results

The following conclusions were derived after statistical analysis of the simulation results:

- Dynamic HSR strategies are more suitable for PDO incidents where traffic management centers have more flexibility in managing traffic.
- Only the part of the shoulder that is 0.5 mi upstream and downstream of an incident location needs to be opened to fully use the potential of shoulders for incident management.
- The opened shoulder section can be closed as soon as possible after the incident is cleared. Opening the shoulder for a longer time will not improve traffic flow conditions.
- The effectiveness of dynamic HSR is rather significant across different roadway geometry, traffic, and incident scenarios. Depending on the traffic condition and number of lane blockage, the average delay can improve by 30 to 80 percent and total traffic throughput by 15 to 40 percent, which are very significant considering only opening a certain section of the one-lane shoulder within a limited amount of time.

Given the results obtained from this study, some related research efforts are recommended for the future. First, insights on the effects of other general system parameters, such as vehicle composition and freeway grades, may need to be derived. Second, this strategy can be combined with other ATM strategies for better incident recovery. It might also be interesting to investigate a left shoulder scenario or shoulders on both freeway sides. Further, the concept of shoulder opening and closure can be extended to general mainline lanes, manipulating freeway capacity to achieve effects similar to speed harmonization. Last, it would be beneficial to investigate how information sharing with connected vehicle technology could play a role in dynamic HSR strategies.

Design Guidance for Safe and Efficient Implementation of Dynamic HSR

The following subsections suggest ways to safely and efficiently implement dynamic HSR.

Geometric Design

Ramp density could be a key factor affecting dynamic HSR effectiveness. Because of the excessive number of possible ramp-deployment scenarios, the dynamic HSR study assumed a ramp density of two ramps per mile and typical lateral clearances. A feasible but costly geometric design option would be the installation of refuge pull-outs at 1,600-ft intervals combined with emergency call boxes. However, communication to and from traffic management centers should be available via smartphones. Enhanced pavement texture and resurfacing could help to make drivers aware of the dynamic HSR functionality and could facilitate safer driving conditions. A dynamic HSR strategy should consider real-world roadway geometry, incident scenarios, and clearance strategies. In practice, a hard shoulder can be on the left, right, or both sides of a freeway mainline. The standard width of a hard shoulder can be anywhere between 10 and 12 ft. These different lane widths affect free-flow speeds and lane capacities. An incident may block a shoulder lane or any number of mainline lanes. Finally, some vehicles involved in accidents are able to drive away or drive to the shoulders. However, in other cases, emergency vehicles must tow incident vehicles to the shoulder or tow them away completely. Dynamic HSR research concluded that only the part of the shoulder 0.5 mi upstream and downstream of an incident location needs to be opened to fully use the potential of shoulders for incident management. If overhead gantries are the sole source of driver notification, this will constrain shoulder length closures to the spacing of these gantries (typically 0.5 to 1 mi). Enhanced accident monitoring and emergency response is desirable in all HSR implementations.

Signing

The dynamic HSR treatment is applicable to minor traffic accidents, in which vehicles come to a stop in one or more mainline freeway lanes. As such, speed reduction advisories are recommended to prevent unsafe travel speed discrepancies between lanes. Opening and closing shoulder sections could be accomplished by smartphones, pushed auditory messages, or overhead gantries. FHWA has conducted research on optimum message sign formats and messages. In a report titled *Travel Time Displays at Freeway Entrance Approaches*, researchers indicated that highway agencies have increasingly made freeway travel time information available to motorists.[75] The project included computerized laboratory and field survey studies to assess participants' comprehension, decisionmaking, strategies, preferences/acceptance/ satisfaction, and behavioral responses to a variety of sign formats and messages.

Signalization

Dynamic HSR advisories or controls are proposed to be implemented by smartphones, pushed auditory messages, or overhead gantries as opposed to signalization.

DRLT LANES

The diamond interchange with its signal control has been successfully used in numerous freeway-to-arterial connections. However, the operation of a conventional signalized diamond interchange design is now becoming a challenging issue, particularly in an urban or suburban environment where heavy traffic volumes must be served and right-of-way is restrictive.(76) Figure 78 shows a typical diamond interchange with two closely spaced signalized ramp intersections, and the LT lanes on the internal link are back-to-back.

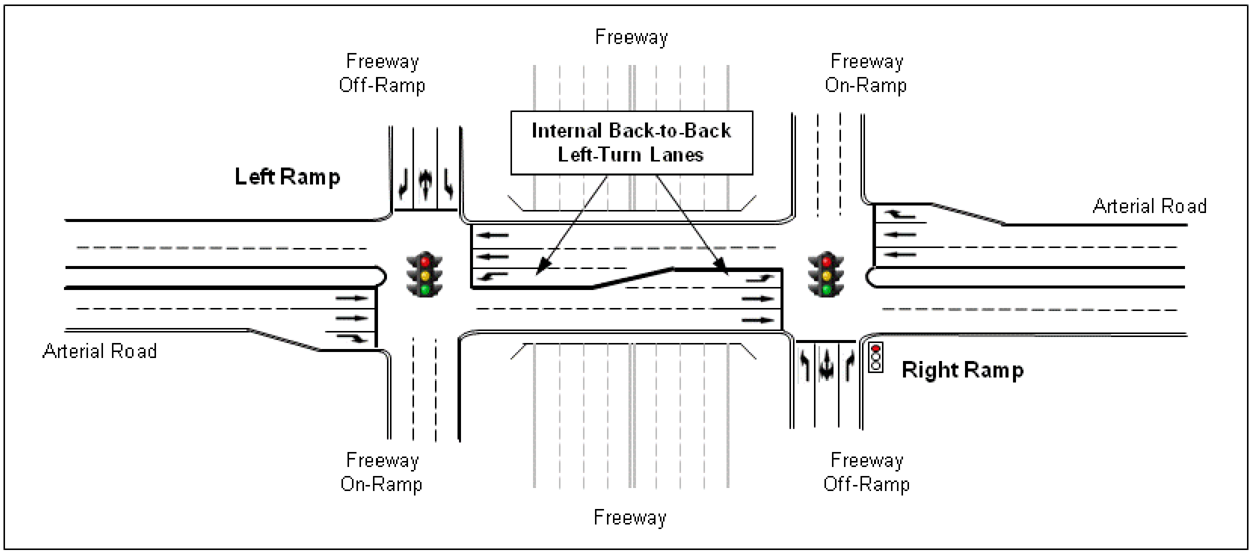


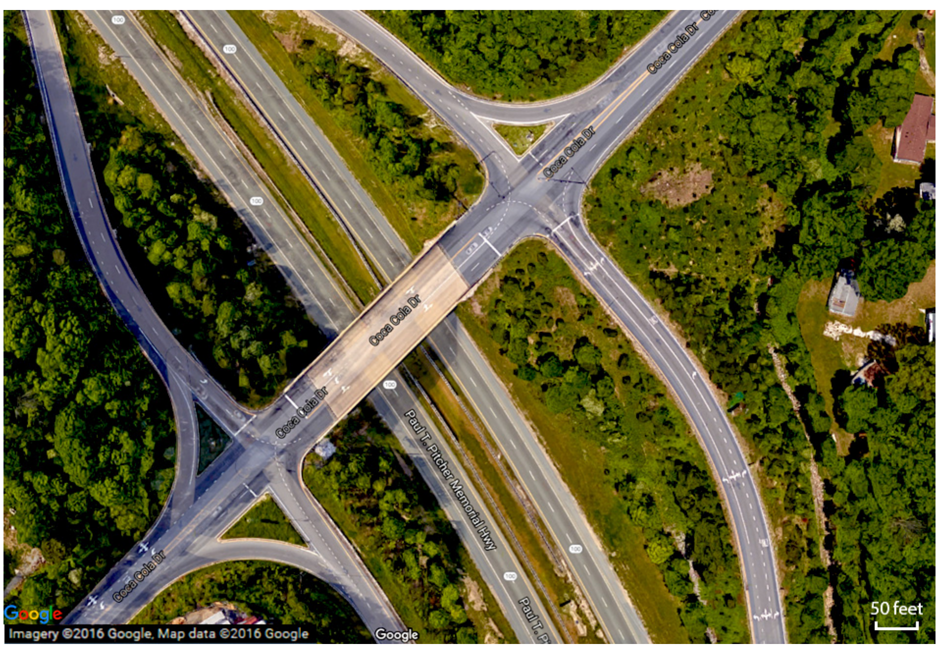
Figure 78. Illustration. Typical signalized diamond interchange with back-to-back LT lanes.

Several design concepts have been applied to improve the operational efficiency and mitigate recurring traffic congestion at diamond interchanges, such as signal control treatments and interchange design improvements. The idea of a DRLT lane was put forth to deal with many of the traffic congestion issues in diamond interchanges.(77) By removing the internal center back-to-back LT lanes and creating reversible lanes, the signal timings can be set to give each LT movement use of the full-length lane during each of the opposing LT movement. The cost to such a design is a TCL which necessitates a completely free lane for the opposing lane's LT movement. The signal timing strategy must be such that the LT vehicles cleared before shifting the green light to conflicting TH traffic. Expanding an overpass (or underpass) or realigning intersection configuration design may eliminate the limitations, but it would result in substantial costs and footprint impacts. This section presents the DRLT interchange design and puts forth scenarios in which this design is advantageous to more traditional design scenarios.

The reversible lane concept can be applied to signalized diamond interchanges. Previous numerical analysis has been done on the DRLT lane diamond interchange.(77) Since there is limited capacity between the two intersections of the diamond interchange, the center lane (LT lane for each approach) is turned into a full-length LT lane when it is that approach's turn for a green phase. These lanes are referred to as DRLT lanes in this section. LT on-ramp vehicles can enter and traverse through these DRLT lanes from both directions but only one direction at a time, which is dynamically controlled by a signal. Overhead traffic signals are necessary to notify drivers when the reversible lanes are open or closed to driving.

Maryland Case Study

The diamond interchange of Paul Pitcher Memorial Highway (Route 100) and Coca Cola Drive in Howard County, MD, was selected as a real-world case study for operational study. Figure 79 displays the aerial view of the selected diamond interchange. The geometry of this diamond interchange was used as a base case for simulation. Its geometry is that of a conventional four-lane diamond interchange with a single LT lane and can be representative of numerous diamond interchanges. Due to the short distances between the intersections, the conventional three-phase signal design was used. A four-phase design would not allow for sufficiently short cycle times. The center LT bays are back-to-back and occupy the length of the region between the two signalized intersections. This interchange was chosen for its delay and congested conditions during peak periods. Specifically, there are heavy LT on-ramp movements that cannot be accommodated by the existing LT storage lanes in recurring conditions. This is largely due to a major business district northeast of the intersection that must use this interchange to get to the highway during the afternoon peak.



©2016 Google®

Figure 79. Map. Selected signalized diamond interchange case study.(78)

A microscopic traffic simulation package, VISSIM™ 6.0, was used to model several geometries of signalized diamond interchanges and compare the operational performances of the base case conventional signal design and the alternative DRLT design concept. Three geometries of cross streets were evaluated in order to simulate the following traffic volumes: (1) four-lane arterial road and five-lane bridge with single LT lane, (2) four-lane arterial road and six-lane bridge with double LT lanes, and (3) six-lane arterial road and six-lane bridge with double LT lanes. The only difference between the base case and the alternative is that the LT bays in the conventional signal design were switched to be reversible lanes for the alternative design. For each of the six total geometries, two traffic demands are considered. An interchange's traffic demand is varied by the v/c ratios: 1.0 and 1.3. The input interchange traffic volume for each scenario is as follows:

- **Scenario 1—four-lane arterial road with single LT lane**: 4,200 vehicles/h (v/c = 1.0) and 5,460 vehicles/h (v/c = 1.3).
- **Scenario 2—four-lane arterial road with double LT lane**: 6,000 vehicles/h (v/c = 1.0) and 7,800 vehicles/h (v/c = 1.3).
- **Scenario 3—six-lane arterial road with double LT lane**: 6,800 vehicles/h (v/c = 1.0) and 8,460 vehicles/h (v/c = 1.3).

For each interchange traffic volume, it was assumed that freeway off-ramp traffic was 70 percent of the arterial road traffic. For instance, the interchange traffic volume of 6,800 vehicles/h accommodated 2,000 vehicles/h on each direction of arterial road and 1,400 vehicles/h on each freeway off-ramp. Three traffic patterns shown in figure 80 were tested. An interchange's traffic pattern characterized by the degree of balance in both the arterial road and freeway ramp traffic movements included balanced, unbalanced (heavy off-ramp LT), and unbalanced (heavy on-ramp LT) patterns. The proportions of turning movements (LT:TH:RT) on arterial roads were 1:1:1 for balanced and 2:1:1 for unbalanced patterns, and those on freeway on-ramps (LT:RT) were 1:1 for balanced and 2:1 for unbalanced patterns. For the conventional diamond interchange scenarios, the optimal signal cycle length and the amount of green time required for each approach were calculated by the HCM procedure using Synchro® software. The cycle length of both signalized intersections was assumed to be the same for coordination purposes. However, it was impossible to deal with the DRLT lane control using the same package. The cycle lengths and minimum green times required on each approach of the DRLT diamond interchange scenarios were set the same as those of conventional scenarios for a fair comparison. Manual refinement was performed in order to meet the signal timing and TCL warrants.

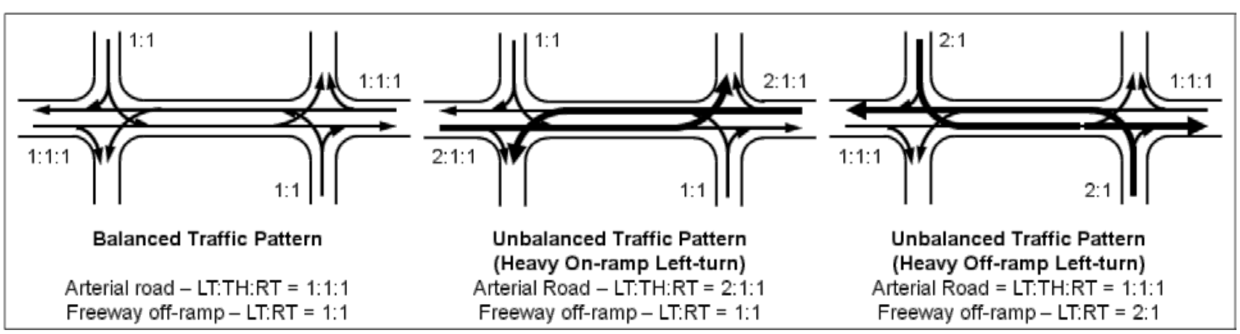


Figure 80. Illustration. Interchange traffic patterns.

The signalization can be rather confusing at first glance, so it can be best understood via a simulation video. This video can be obtained by contacting STOL at FHWA's TFHRC. The simulation was created in VISSIM™ by simply stacking LT lanes on top of one another and then creating a conflict area over the length of the segment for the two lengths of roadway. This way, if the intersection does fail, all the vehicles for the oncoming reversible lane would stop at the beginning of their approach and simply would not be able to move forward, causing the cycle to fail. This is a realistic interpretation of what may occur in a real-world situation. When one approach fails, the phasing must go through an entire cycle before the opposing LT movement can be evacuated.

Operational benefits of the DRLT design, relative to the conventional diamond design, were as follows:

- The simulations showed increased LT on-ramp movement throughput, reduced network travel time, and reduced delay for all three selected geometries. The effects were most prominent when the number of TH lanes was proportionally higher than the number of LT lanes.

- For balanced base case condition scenario 1 at 1.0 v/c (geometry 1), no operational benefits to the DRLT lane were found. There was actually a 1 percent reduction in throughput and 21 percent increase in delay for this case. Clearly, if the turning movements are balanced, and the intersection is small with near saturation volumes, the DRLT design should not be implemented. However, this changes when the interchange size increases. For scenario 2, throughput increased by 2 percent. For scenario 3, the throughput increased by 13 percent. In general, the dynamic LT design becomes more attractive as the interchange has greater number of TH lanes. Also, the less balanced the turning movements (higher number of LTs), the better the DRLT design performs, ranging from no benefit at low LT movements to 29 percent throughput improvements at high volumes and high LT rates.

- For DRLT design, the entry ramp LT movement travel time was lower for all simulations runs. However, this was counteracted by the longer TH movements as compared to the conventional three-phase design.

- These results showed that the best implementation of the new interchange design may be during certain times of day where the LT on-ramp volumes are high. Throughout the day when the turning movements are balanced, the intersection can remain a conventional three-phase design. However, when the movements change (possibly during afternoon peak) the DRLT design can be beneficial and implementable.

Virginia Case Study

The diamond interchange of Georgetown Pike and Interstate 495 in McLean, VA, was selected for a second case study, whose experimental design was similar to the Maryland case study. Plan views for the before-and-after lane configurations are illustrated in figure 81 and figure 82. Table 29 demonstrates very significant traffic flow improvements as measured by delays, speeds, and vehicle stops. Specifically, the researchers noted that southbound (SB) off-ramp traffic improved significantly in the morning peak. Further, benefits in the afternoon peak were even more significant, mostly due to major improvements in WB on-ramp flow. In the base case, there had been a very long queue, which cleared almost completely in the DRLT case.

Figure 81. Screenshot. Georgetown Pike interchange with conventional lanes.

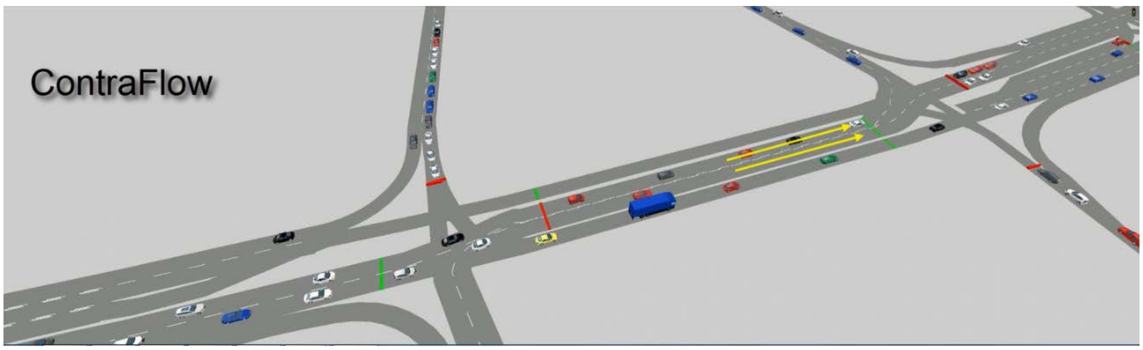


Figure 82. Screenshot. Georgetown Pike interchange with DRLT lanes.

Table 29. Case study simulation results for DRLT lanes.

Scenario	Morning Peak Delay (s/vehicle)	Morning Peak Speed (mi/h)	Morning Peak Number of Stops	Afternoon Peak Delay (s/vehicle)	Afternoon Peak Speed (mi/h)	Afternoon Peak Number of Stops
Base case	140.5	9.7	1.9	343.4	4.7	5.7
DRLT	54.7	16.8	1.1	65.2	16.4	1.2
Improvement (percent)	61.1	73.8	40.7	81.0	249.4	78.5

Conclusions for Virginia and Maryland Case Studies

This DRLT design is not applicable for all diamond interchange scenarios, but for high volumes with geometries of low number of LT lanes, its application may greatly reduce travel time and increase interchange throughput. These conclusions apply only for the selected geometries and traffic conditions modelled and simulated. Finally, it should be noted that this is a theoretical evaluation of a new design concept that remains unused in the real world. Safety concerns should be fully evaluated before serious consideration for implementation.

Design Guidance for Safe and Efficient Implementation of DRLT Lanes

The following guidance is suggested for safe and efficient implementation of DRLT lanes. A follow-on human factors research project began in April 2016. It is expected that final reports and Techbriefs for the human factors study will become available in 2017 or 2018.

Geometric Design

In the DRLT strategy, it was important that on-ramp vehicles did not get stuck on the reversible lane after the signal turned red. The entire platoon of vehicles should be served during each green phase. According to traffic flow theory, when a signal turns green, platoons of closely spaced vehicles begin to disperse as they travel downstream. If the distance between intersections is too far, platoon dispersion may lower the probability of all vehicles arriving on green, even when signal timings are optimized. Because of this, a relatively small distance between intersections (i.e., 650 ft or less) was recommended for the DRLT treatment. In the case study of Paul Pitcher Memorial Highway (Route 100) and Coca Cola Drive presented previously, the distance between intersections was approximately 400 ft, and the operational benefits of DRLT were shown (in simulation experiments) to be significant.

Figure 83 through figure 85 illustrate three layouts of DRLT signalized diamond interchanges used in this study: (1) four-lane arterial road and five-lane bridge with single DRLT lane, (2) four-lane arterial road and six-lane bridge with double DRLT lanes, and (3) six-lane arterial road and six-lane bridge with double DRLT lanes. The highlighted bold lines in the figures indicate an elevated curb, with suggested dimensions of 2 by 2 inches. This would discourage adjacent lanes' vehicles from entering the reversible lanes during a time when the opposing traffic has the right-of-way. Due to the possibility of emergency situations where vehicles get stuck in reversible lanes after the signal turns red, the elevated curb height should not exceed 3 inches. In these situations, the stuck vehicles will need some way to vacate the right-of-way so that reversible lane flows can continue to operate normally. In other words, the stuck vehicles would change lanes by driving over the elevated curb even though the elevated curb is designed to discourage lane changing under normal circumstances. It is also recommended that the elevated curbs be extended further beyond the STOP bar to discourage off-ramp vehicles from entering the reversible LT lanes.

Simulation studies found that DRLT increased on-ramp throughput, reduced network travel time, and reduced delay for all geometries.(76) However, these operational benefits were most prominent when the number of TH lanes was proportionally higher than the number of LT lanes.

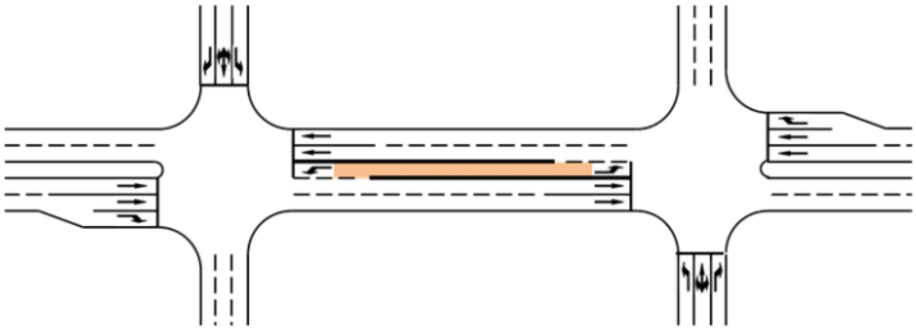


Figure 83. Illustration. DRLT design for a four-lane road with a single LT lane.

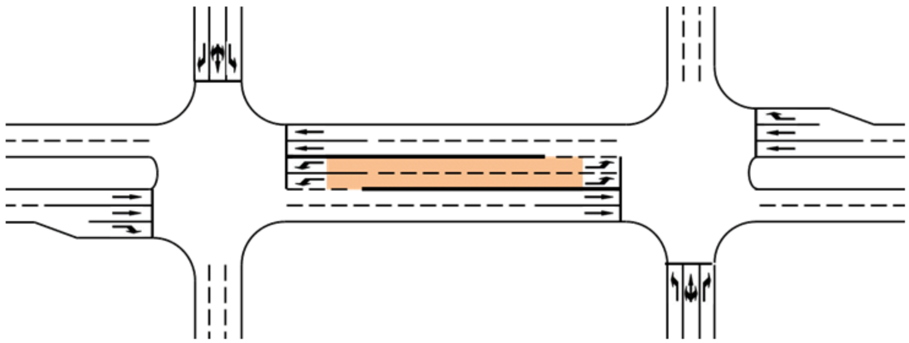


Figure 84. Illustration. DRLT design for a four-lane road with dual LT lanes.

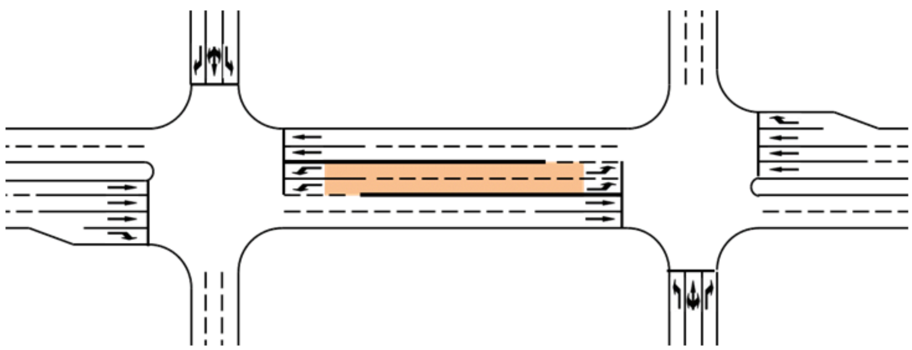


Figure 85. Illustration. DRLT design for a six-lane road with dual LT lanes.

Signalization

Overhead traffic signs and signals can inform drivers about whether reversible lanes are open or closed at any given time. Figure 86 through figure 88 illustrate the proposed signal and sign display for a four-lane arterial and a five-lane bridge. The interior roadway has a single DRLT lane. When the DRLT displays a green signal during the peak period (see figure 86), the overhead changeable message sign (CMS) transforms into a guide sign for accessing the freeway. The large green arrow indicates that the reversible lane is open to SB TH traffic. Traffic signals for the other two TH lanes display circular green signal indications. The middle lane displays a black arrow, indicating that only TH movements are allowed from that lane. The size and spacing of overhead DMS should reflect reversible lane widths and should conform to MUTCD standards.[79]

During off-peak periods when reversible lane use is disallowed (see figure 87), drivers must use the LT pocket to access the downstream on-ramp. The overhead CMS guide sign is put into blank mode. A large red cross is displayed on the other CMS, indicating that the lane is not open for traffic. However, the other two TH lanes are open, as indicated by circular green signal indications (see figure 87). Figure 88 shows what the SB displays would look like when NB traffic is moving. Traffic signal heads in the rightmost TH lanes display a circular red signal indication, and the overhead DMS displays a large red cross.

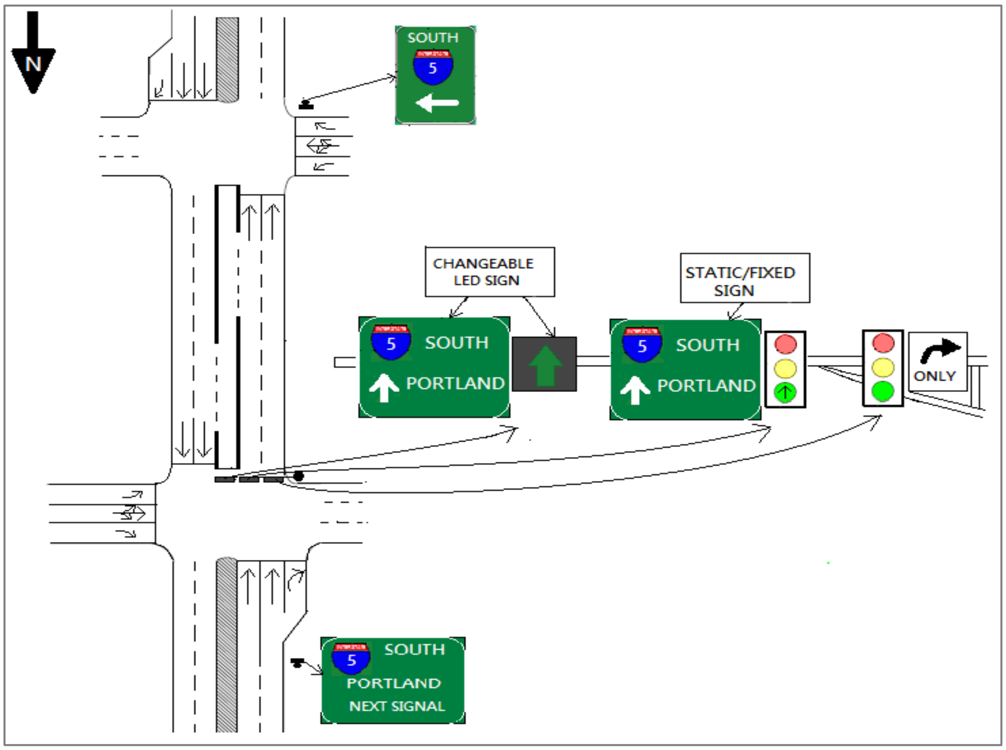


Figure 86. Illustration. Proposed single-lane DRLT signalization during peak-period green phases.

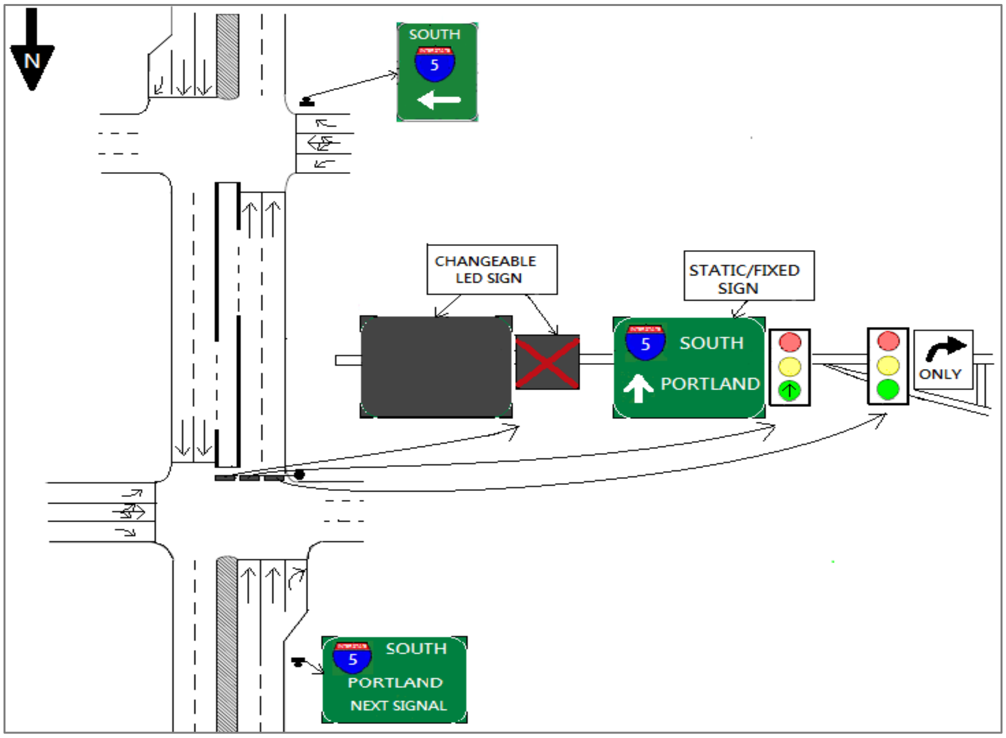


Figure 87. Illustration. Proposed single-lane DRLT signalization during off-peak-period green phases.

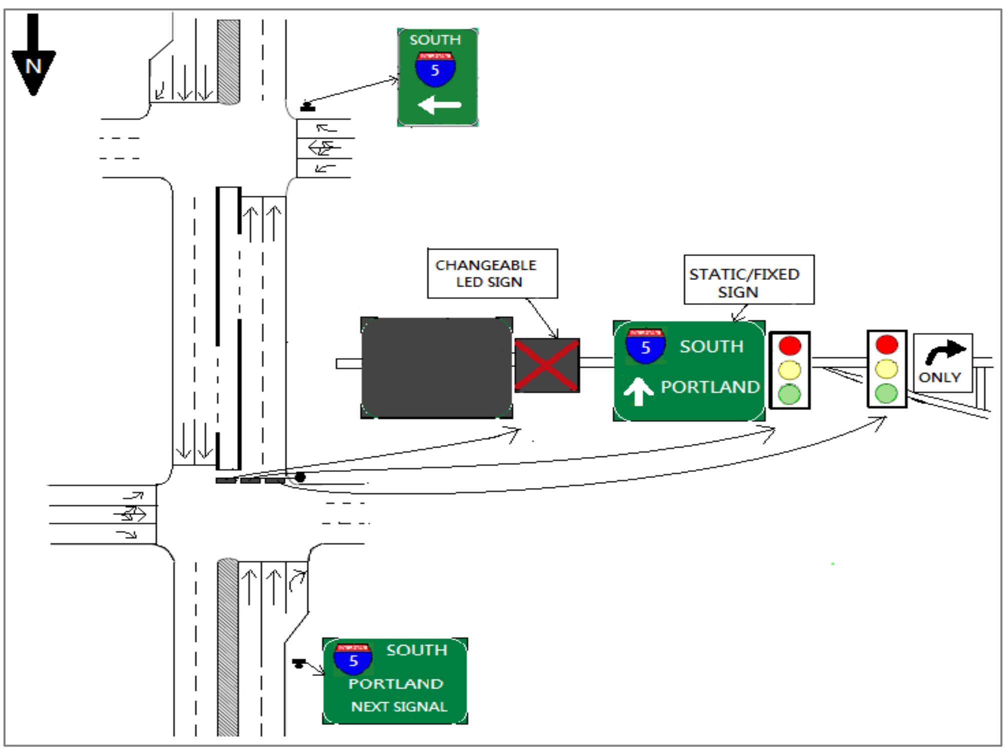


Figure 88. Illustration. Proposed single-lane DRLT signalization during red phases for all periods.

For dual DRLT lane scenarios, additional signs were needed to properly guide drivers. Figure 89 illustrates the proposed signal and sign display for a six-lane arterial and a six-lane bridge. Design guidance for the dual-lane DRLT includes the following:

- To avoid weaving issues at the intersection and to guide drivers into the appropriate downstream lanes, overhead dynamic lane marking signs are displayed for each lane.

- Similar to the single DRLT design, traffic signals for middle TH lanes display circular green signal indications with a black arrow, indicating that only TH movements can be made from those lanes.

- A roadside advance guide sign can be placed upstream of the interchange.

Similar design guidance can be followed for a four-lane arterial road with a six-lane bridge having dual DRLT lanes. However in this scenario, a LT storage lane should be placed within the upstream median, if possible. This would allow LT storage lane drivers to proceed directly into the left-most reversible LT lane during peak-period green phases. Figure 90 illustrates a set of dual-lane DRLT guide signs for use during the peak and off-peak periods, respectively. The designs are mostly consistent with similar signs from section 2b of the 2009 MUTCD.[(79)] These DMS designs could potentially be programmed within a single physical sign and illuminated when necessary. The signs could be post-mounted or placed overhead.

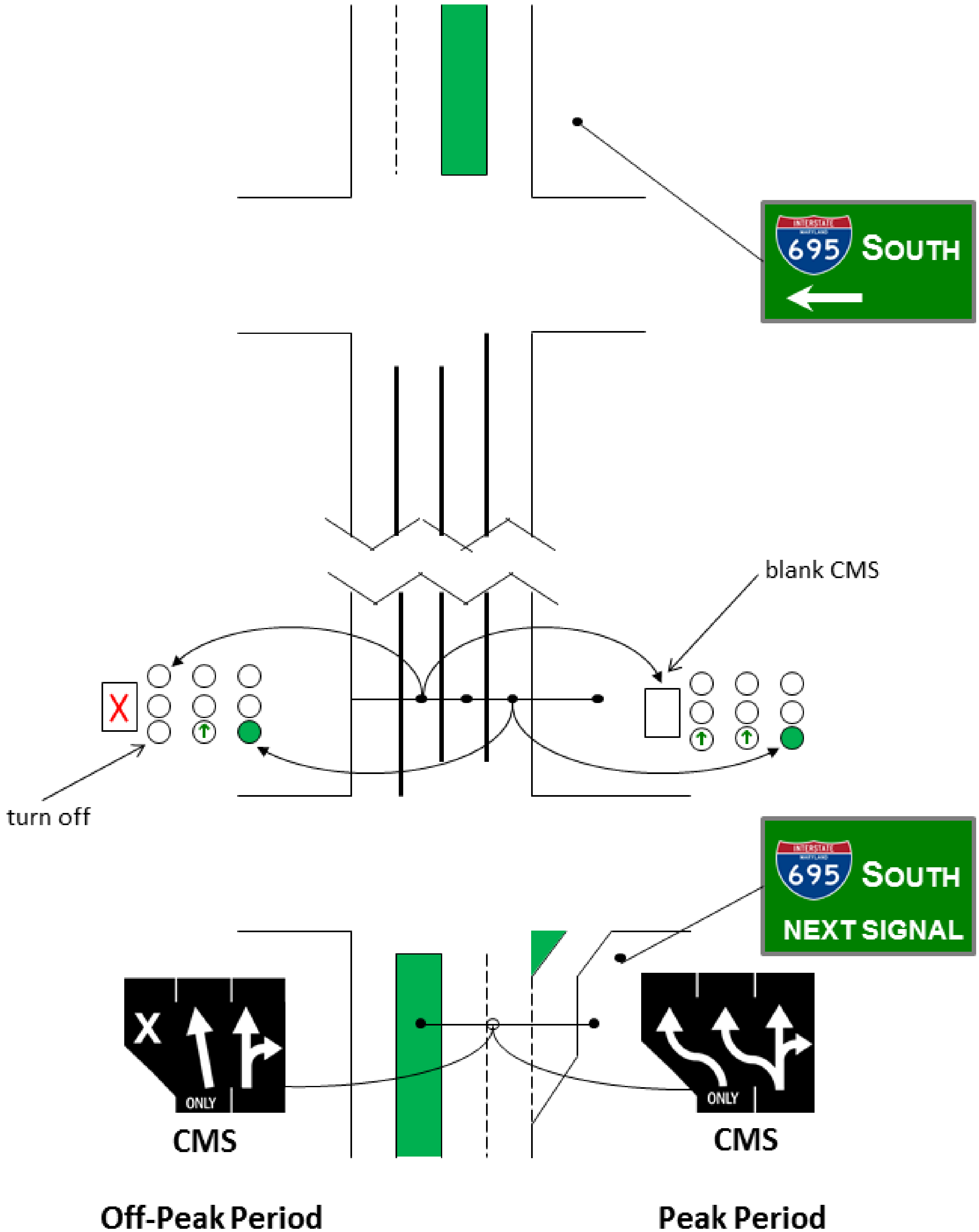


Figure 89. Illustration. DMSs for dual DRLT lanes during peak and off-peak periods.

Four-phase signal timing plans are required for DRLT operation. Figure 90 shows the phasing diagram and ring structure of two signalized intersections for the DRLT diamond interchange proposed in this study.

Two signals were treated as one four-phase signal control. Each of the four phases indicates the movements from four external approaches, EB phase (Φ2), SB (Φ4), WB (Φ6), and NB (Φ8), and Φ1 and Φ5 are for internal LT movements. The signal sequence is typical of the conventional four-phase control.

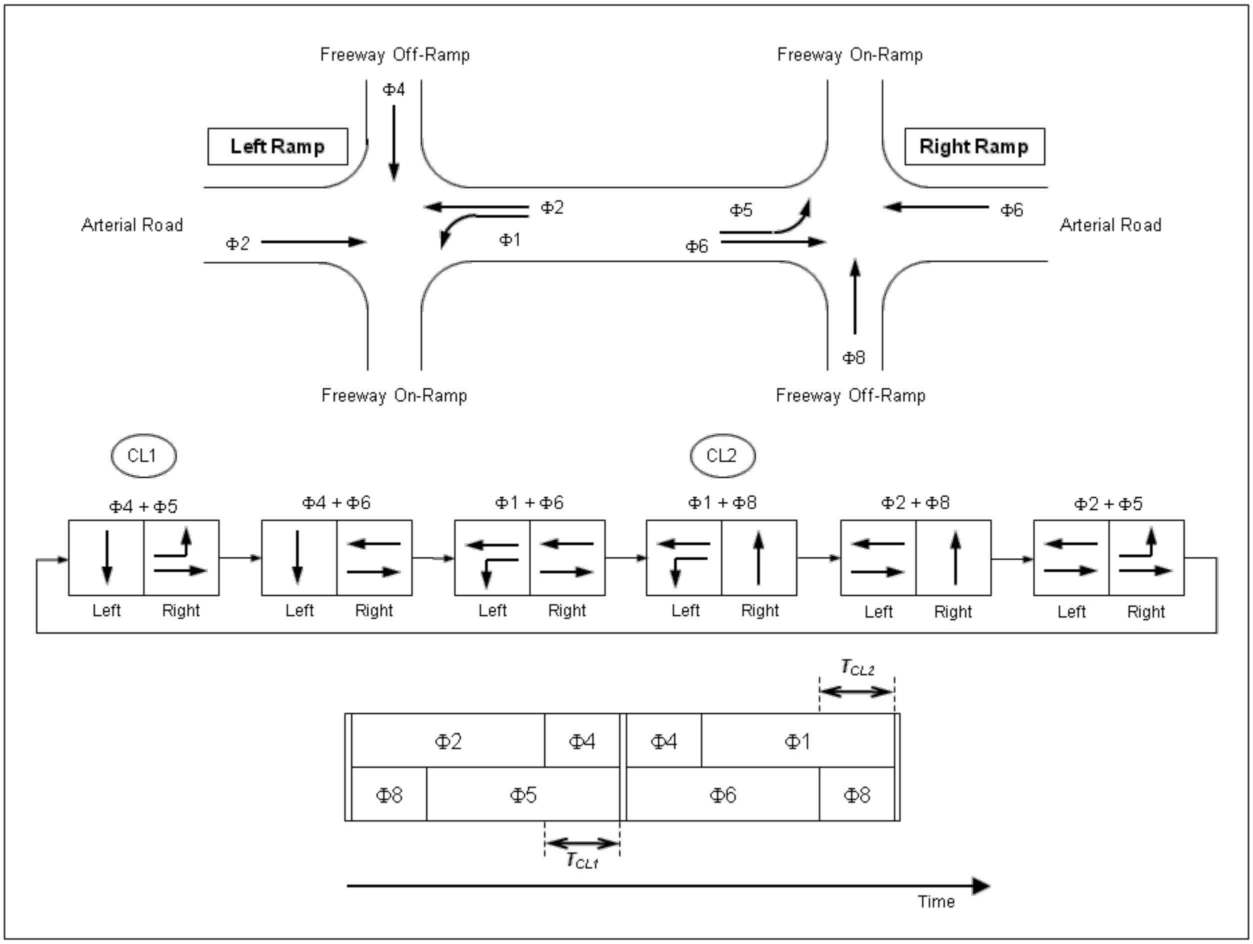


Figure 90. Illustration. Signal phasing and ring structure for proposed DRLT diamond interchanges.(76)

To create DRLT movements for two opposite directions, the TCLs, denoted as *TCL*1 and *TCL*2, (as shown in figure 90) were proposed to ensure there was no conflict between two opposite movements. TCL is the minimum green time required for LT on-ramp vehicles to depart from the upstream signalized intersection, fully progress along the DRLT lane, and complete their LT movement at the downstream signalized intersection without stopping on the LT lanes. If this value is given an insufficient allotment of time, then the next cycle will fail, and no cars will be able to turn left from the opposite approach. It is therefore imperative that this TCL is sufficient for all vehicles on the network.

TCL ensures that the last left-turning vehicle from the upstream intersection can make an LT at the downstream intersection during one green phase. In other words, vehicles from Φ2 (or Φ6) should complete their movement before the end of Φ5 (or Φ1). There should not be any vehicles remaining on the DRLT lanes so that the opposing LT vehicles can enter the DRLT lanes from the opposite direction. Mathematically, TCL should be at least longer than travel time between two intersections, as expressed in the equation in figure 91.

$$TCL1 \ (\text{or } TCL2) > \frac{\text{Interchange spacing}}{\text{Vehicle speed}}$$

Figure 91. Equation. Calculation of DRLT signal TCL.

For example, if the interchange spacing is 300 ft and the vehicle operating speed on the arterial road is 25 mi/h, then the TCL should be at least 8 s.

It should be noted that traffic movements on Φ5 and Φ1 occupy the same physical space of the reversible lane from opposite directions; hence, these two phases cannot occur at the same time. For this alternative to be feasible, some safety precautions must be taken. The dynamic reversible lane could implement sensing cameras in order to make sure the lane is cleared before the phasing is switched. There could conceivably be a broken-down vehicle in the DRLT lane in which equipment must be able to respond to by disallowing entrance to the lane via signalization.

Signing and Pavement Marking

Pavement markings and striping generally need to reflect off-peak operations. In the case of DRLT, this means the striping would indicate location and length of LT pockets facing in each direction. However, in the peak period, signalization, and DMS information (shown in the previous section) would take precedence over the striping.

An advance warning sign to notify drivers of the upcoming dynamic lane treatment should be considered. This advance warning sign could contain a flashing yellow light, as shown in figure 92, similar to warning signs for DLG. The flashing yellow light could then be illuminated at times of the day when dynamic lane reversals are in effect. To provide drivers enough reaction time for smooth lane changing, it is recommended that the upstream signage be installed 1,000 to 1,500 ft away from the intersection.

Figure 92. Photo. Upstream advance warning sign for DRLT.[80]

In the proposed signal phasing and ring structure (see figure 90), when off-ramp vehicles turn left onto the arterial, reversible lane vehicles will be traveling in the opposing arterial direction. This means that unlike a typical diamond interchange, immediate re-entry to the freeway (i.e., a U-turn maneuver spanning both signalized intersections) would not be possible. As such, proper signage is recommended near the off-ramp to notify drivers of this restriction. Figure 93 illustrates an example of "no re-entry" warning signage under a typical freeway exit sign. The no re-entry sign could also contain a flashing yellow light to be illuminated when dynamic lane reversals are in effect.

©2016 Google®

Figure 93. Map. Example of no re-entry signage.[(81)]

CLT POCKETS

During the peak hours of traffic congestion, some signalized intersections suffer from excessive queue spillover of left-turning vehicles into adjacent TH lanes. The innovative CLT pocket treatment aims to mitigate the problem by dynamically allocating lanes in the opposing direction to create an additional LT pocket lane, as shown by the green area in figure 94.[(82)] By adding additional capacity to the LT movement, delays for the entire intersection can be reduced. Increased LT movement capacity can help to mitigate or eliminate the spillover. In some cases, the additional LT capacity could allow LT green times to be reduced such that green time could be reallocated towards other turning movements at the signal. Although this congestion mitigation strategy requires presignals to control entrances to the contraflow turn pockets, it would not require advanced vehicles or infrastructure. As a result, it has the potential of being a cost-effective bottleneck mitigation strategy. Moreover, there appear to be at least 50 successful real-world implementations of CLT pocket intersections in China, which are described later in this section.

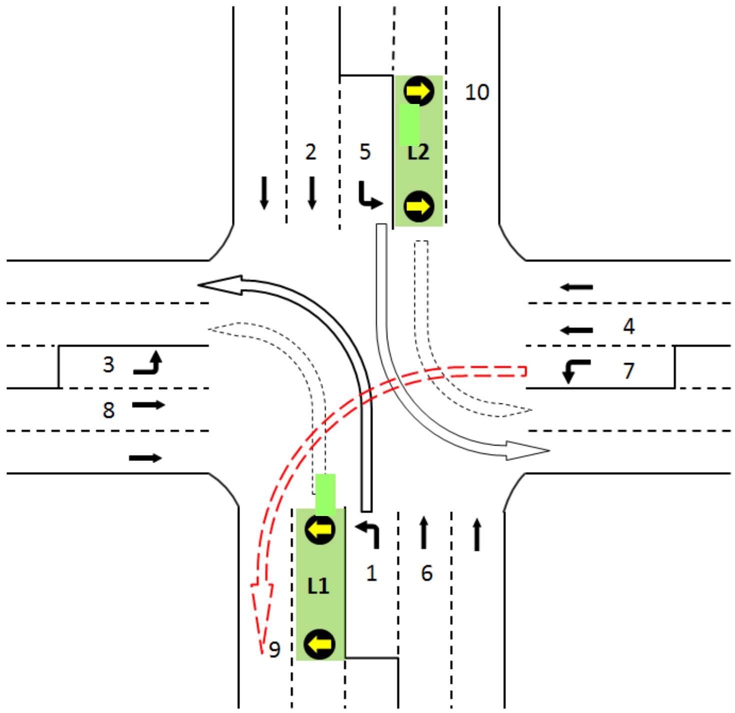


Figure 94. Illustration. CLT intersection geometry.

Maryland Case Study

The intersection between Tuckerman Lane and Rockville Pike in Bethesda, MD, was selected because the intersection has high LT volume demand, has enough lanes in the opposing direction, and has enough receiving lanes. NB and SB TH movements are oversaturated due to excessive volume demands. Thus, in the contraflow treatments, excess LT green times can be reassigned to the TH movement while leaving the cycle length unchanged. Figure 95 illustrates the lane configuration and turn movement demand volumes at this intersection.

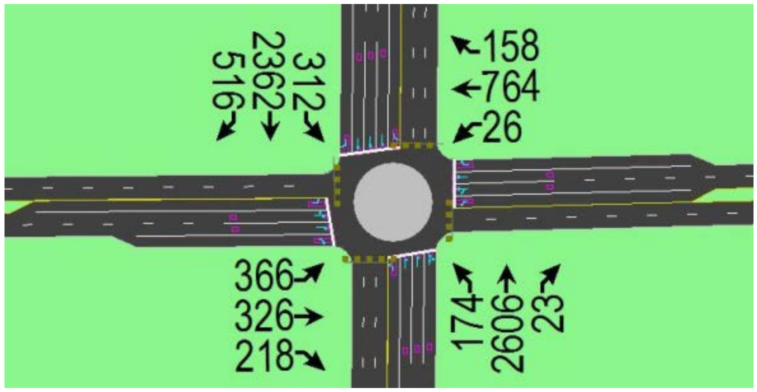


Figure 95. Illustration. Turn movement volumes at Tuckerman Lane and Rockville Pike in Maryland.

VISSIM™ simulation results showed that the contraflow treatment produced the most delay savings under high travel demands. Another finding was that the highest percentage delay reductions occurred when NB and SB turning movement demands were balanced (i.e., almost equal). Table 30 reveals delay reductions between 7 and 22 percent under various scenarios. Therefore, the case study simulations indicated that CLT pockets are capable of significant overall delay reduction.

Table 30. Case study simulation results for CLT pockets.

Volume Demand Scenario	1.0 v/c		1.05 v/c		1.1 v/c	
	Base	Contraflow	Base	Contraflow	Base	Contraflow
Scenario 1						
Cycle length (s)	100	100	100	100	100	100
Throughput (vehicles)	5,963	5,983	6,203	6,266	6,276	6,416
Delay (s/vehicle)	51	42	67	52	83	70
Scenario 2						
Cycle length (s)	100	100	100	100	100	100
Throughput (vehicles)	5,452	5,600	5,633	5,810	5,797	5,961
Delay (s/vehicle)	44	41	47	42	50	44
Scenario 3						
Cycle length (s)	120	120	120	120	120	120
Throughput (vehicles)	5,963	5,983	6,203	6,266	6,276	6,416
Delay (s/vehicle)	52	46	75	60	91	74

Sensitivity Analysis

An additional set of simulations was conducted to assess CLT benefits under various LT demand levels. To achieve this objective, researchers created an ideal VISSIM™ simulation model instead of modeling a real-world case. The experimental design and intersection conditions are shown in table 31. While traffic conditions were held constant on the NB and SB approaches, traffic demands increased in 100-vehicle/h increments on the EB and WB approaches. Figure 96 and figure 97 demonstrate that the delay reductions and throughput increases, respectively, became more significant as LT demands increased.

Table 31. Sensitivity analysis experimental design for CLT.

Intersection Characteristics	Approach			
	EB	**WB**	**NB**	**SB**
Lane configuration	1 CFLT pocket, 1 LT pocket, 2 TH, and 1 RT pocket	1 CFLT pocket, 1 LT pocket, 2 TH, and 1 RT pocket	1 LT pocket, 2 TH, and 1 RT pocket	1 LT pocket, 2 TH, and 1 RT pocket
Volume (vehicles/h)	• LT: 200–700 • TH: 1,500 • RT: 200	• LT: 200–700 • TH: 1,500 • RT: 200	• LT: 150 • TH: 750 • RT: 100	• LT: 150 • TH: 750 • RT: 150

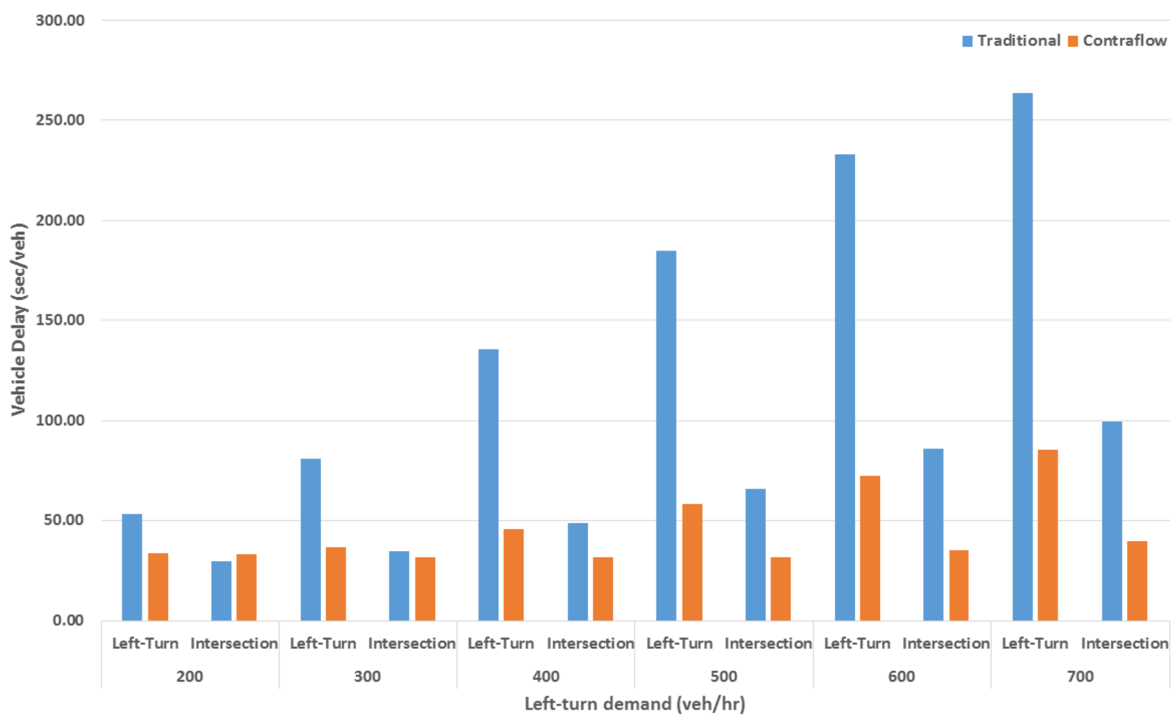


Figure 96. Graph. Delay reductions due to CLT at various LT demand levels.

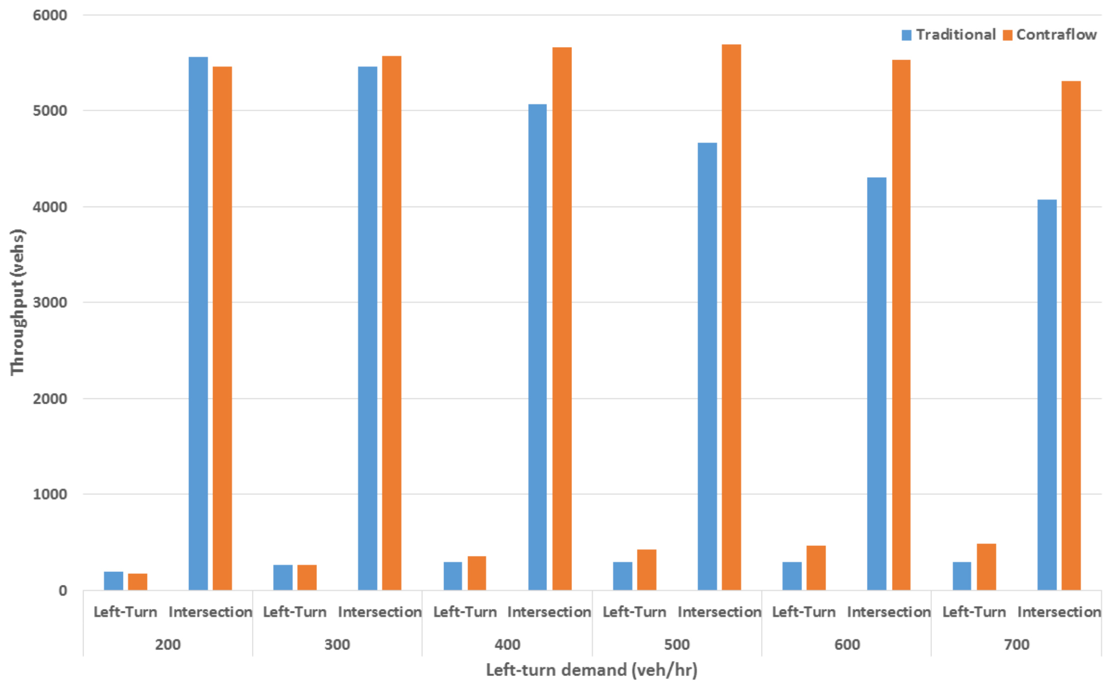


Figure 97. Graph. Throughput increases due to CLT at various LT demand levels.

Design Guidance for Safe and Efficient Implementation of CLT Pockets

The following guidance is suggested for safe and efficient implementation of CLT pockets. It is mostly consistent with real-world implementation in China, which are illustrated and described later in this chapter.

Geometric Design

The CLT treatment is only recommended for intersections having heavy LT demands and specific geometric designs. Under the right conditions, CLT application could greatly reduce travel time and increase intersection throughput. For safety reasons, left-turning vehicles cannot have access to the CLT pocket at all times. Instead, vehicles must enter the CLT pocket through an opening, as shown by the shaded area in figure 98. In the figure, L1 is the length of the CLT pocket, while L2 is the length of the standard LT pocket. Multiple factors can influence the most appropriate length of L1 in terms of both traffic flow efficiency and safety. Generally, L1 should be shorter than L2.

The bolded black lines in figure 98 through figure 101 indicate curbs that prohibit passing. The saturation flowrate of the CLT pocket is expected to be similar to the normal LT lane. Depending on the demand volume level, there are three possible lane utilizations (see figure 99 through figure 101). These vehicle assignments are based on assumption that the drivers prefer using the normal LT lane but will also use the CLT lane if the normal LT lane is filled. At low volume, as shown in figure 99, the normal LT pocket is long enough to store all the LT vehicles, and the expected lane utilization ratio is 1:0. At medium volume, as shown in figure 100, some vehicles will use the CLT lane, and the expected lane utilization ranges between 1:0 and 1:1. For the high-volume case, as shown in figure 101, when both the two lanes are oversaturated, the expected lane utilization is L2/L1.

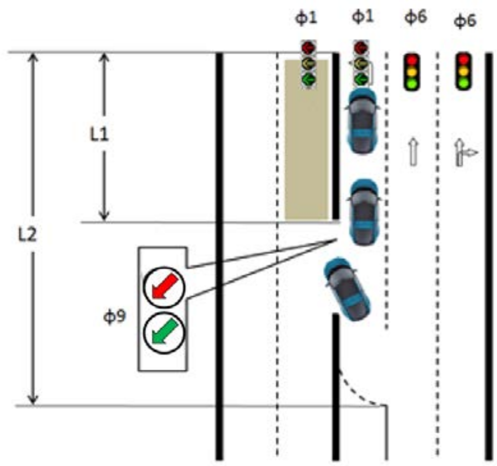


Figure 98. Illustration. CLT pocket configuration.

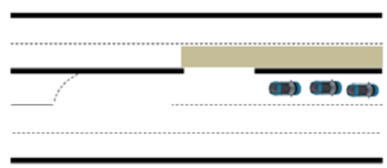


Figure 99. Illustration. Expected CLT lane utilization under low LT demand.

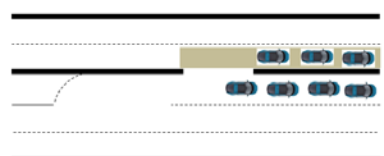


Figure 100. Illustration. Expected CLT lane utilization under medium LT demand.

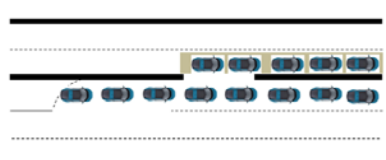


Figure 101. Illustration. Expected CLT lane utilization under high LT demand.

Since the CLT treatment is only considered for intersections with high LT demand, the ideal length would achieve a 1:1 lane utilization ratio. Using 95th percentile of the LT queue length in feet, the optimal CLT pocket length is one-half of that value (assuming only one existing LT lane). Figure 102 and figure 103 can be used to calculate the 95th percentile queue length based on the 2000 HCM.[(73)] In this case, the CLT pocket helps reduce congestion at the high travel demand levels.

$$Q_{95\%} = Q \times f_{B95\%}$$

Figure 102. Equation. 95th percentile queue length.

$$f_{B95\%} = 1.6 + e^{\frac{-Q}{5}}$$

Figure 103. Equation. 95th percentile back-of-queue factor.

Where:

$Q_{95\%}$ = 95th percentile queue length (number of vehicles).
Q = Average queue length (number of vehicles).
$f_{B95\%}$ = 95th percentile back-of-queue factor.
e = Exponential function.

Although the optimal CLT pocket length can be estimated by the previously mentioned methods, other constraints may apply. If the CLT pocket length is too long, traffic operations may be inefficient, as drivers would have a strong tendency to only use the standard LT pocket. Moreover, long CLT pockets would increase the risk of queues not clearing during the green phase, and blocking oncoming vehicles later in the signal cycle. If the CLT pocket length is too short, operational benefits might not be significant enough to justify deployment of the innovative treatment. Based on simulation experiments and engineering judgment, the researchers recommend CLT turn pocket lengths between 150 and 250 ft. The ability to use an opposing lane for CLT depends on the total number of opposing lanes and the number of receiving lanes. There should be at least two lanes in the opposing direction. While the innermost opposing TH lane is being used for CLT, the outermost opposing TH lane can be used by incoming vehicles. Next, the total number of LT lanes must not exceed the number of receiving lanes. Finally, incoming LT vehicles (from a perpendicular approach) would have a larger turning radius than before (see figure 94) because they must avoid the CLT pocket. Therefore, the intersection must have enough width to avoid LT conflicts.

Some consideration should be given to the risk of side-swipe accidents between CLT vehicles and right-turning vehicles on the adjacent approach. For example, figure 104 illustrates NB left-turning vehicles in lane 2 at risk of being side-swiped by EB vehicles turning SB into lane 1. This risk could be mitigated by channelized RT islands on the EB approach or by flex posts installed between lanes 1 and 2.

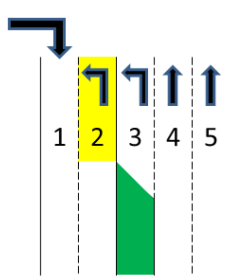


Figure 104. Illustration. Side-swipe risk between CLTs and adjacent RTs.

Signalization

CLT pocket entrances are controlled by presignals. Figure 105 illustrates a presignal, which should be presented at the CLT pocket entrance. This signal, which may have a similar construction and appearance as a ramp metering signal, is used to indicate when it is safe to enter the CLT pocket. The red-green light should be arrow-shaped instead of ball-shaped, so TH-moving drivers will not feel obligated to stop. Conflicting flows must be prevented by the use of proper TCLs. Vehicles in the CLT pocket must be cleared before vehicles move in the opposing direction. This can be achieved by terminating presignal green durations a few seconds in advance of conflicting phase start times. Figure 106 shows the proposed overhead signal at a CLT intersection when the CLT lane is available. Figure 107 shows the proposed overhead signal at a CLT intersection when the CLT lane is not available. An advance warning sign with a flashing yellow light should be installed to notify drivers of the CLT pocket.

Figure 105. Illustration. Presignal at CLT pocket entrance.

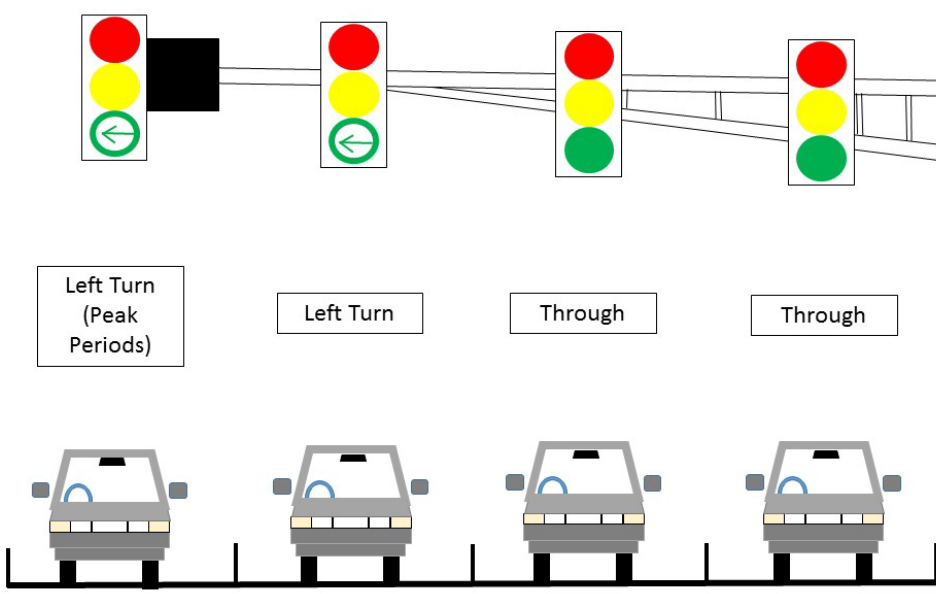


Figure 106. Illustration. Proposed CLT signalization during peak period.

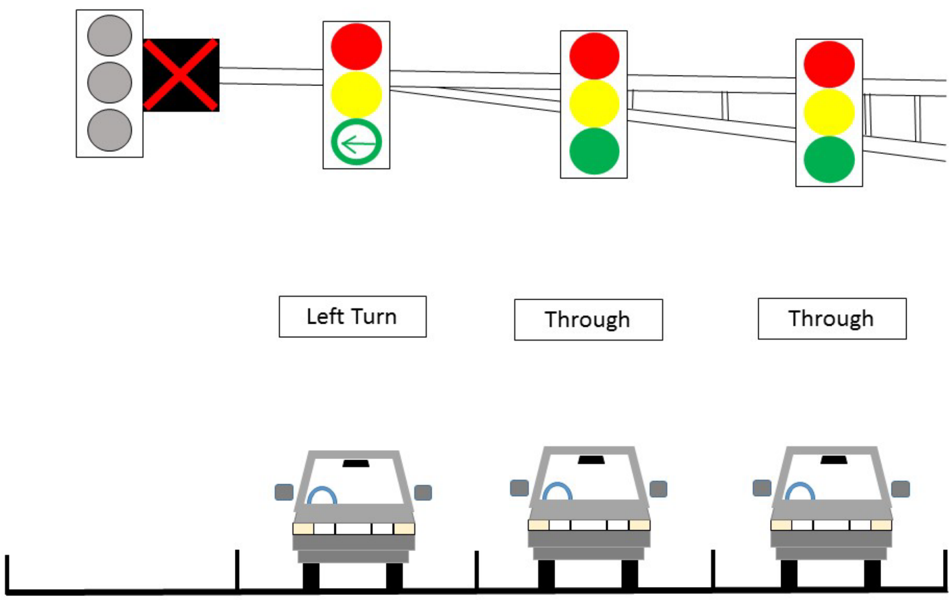


Figure 107. Illustration. Proposed CLT signalization during off-peak period.

A proposed signal phasing plan, having protected LT phases on all approaches, is illustrated in figure 108. NB and SB approaches have CLT treatments. The pocket entrances are controlled by Φ9 (NB and Φ10 (SB). Three incoming flows in conflict with the NB CLT lane are illustrated by red arrows in figure 109. The conflict with SB TH movement must be prevented by proper TCLs of Φ9 so that all vehicles on the CLT lane can be cleared before SB TH vehicles start moving. This can be achieved by terminating the Φ9 green phase a few seconds before Φ1 green ends, as shown by *TCL*1 in figure 108. *TCL*1 is the time offset between Φ9 and 1. The appropriate *TCL*1 and *TCL*2 durations depend on the CLT pocket length since longer time is needed to clear the vehicles if the CLT lane is longer.

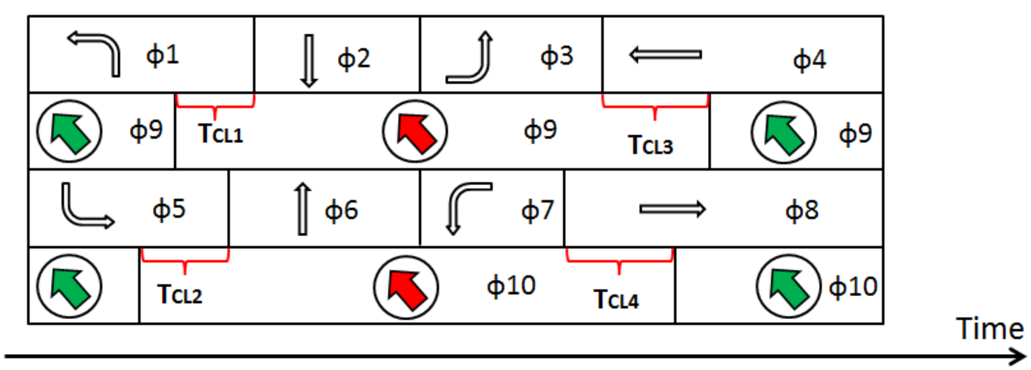


Figure 108. Illustration. CLT pocket signalization.

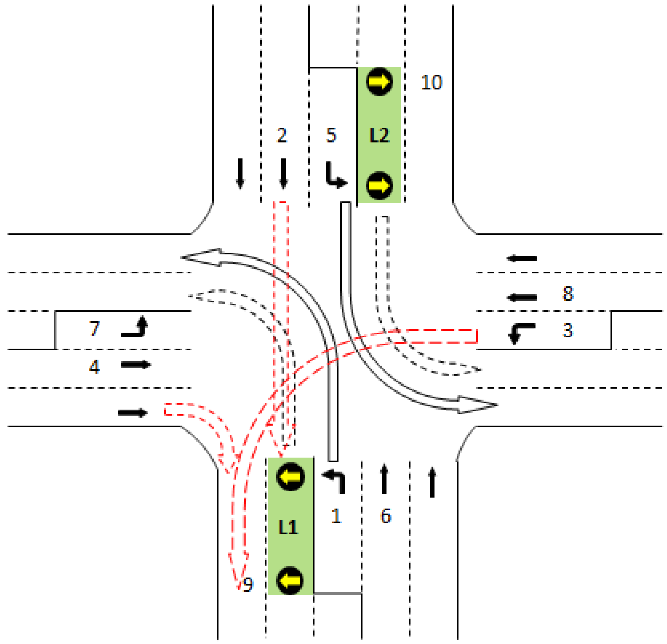


Figure 109. Illustration. CLT intersection geometry.

The equation in figure 110 is proposed to ensure that the offset time is long enough to clear the full length of the turn pocket. For example, if the CLT pocket is 150 ft long and the average left-turning speed is 20 mi/h, then *TCL*1 should be at least 5 s long.

$$TCL1 \text{ (or } TCL2) > \frac{L1 \text{ (or } L2)}{\text{Average vehicle speed}}$$

Figure 110. Equation. Calculation of CLT signal TCL for clearing the turn pocket.

To add a layer of safety from a temporal standpoint, it is best to have conflicting LT vehicles (e.g., WB left and NB left) moving at completely different times during the cycle. Thus, Φ9 should turn green *TCL*3 seconds after Φ3 to avoid time-based overlaps with WB LT vehicle movements. Note that routing WB LT vehicles into the right-most lane helps to prevent the same conflicts, albeit from a spatial standpoint. The distance of the WB LT red arrow in figure 109 is called the "red track." *TCL*3 should be longer than the time needed for WB LT vehicles to traverse the red track. The equation in figure 111 is proposed for ensuring that NB LT vehicles can safely enter the CLT pocket.

$$TCL3 > \frac{\text{Intersection spacing (red track)}}{\text{Average vehicle speed}}$$

Figure 111. Equation. Calculation of CLT signal TCL for ensuring safe pocket entry.

The amount of green time that could be truncated from the LT green depends on two factors: (1) how much green time can be saved by the contraflow lane and (2) whether the remaining green time should be enough to clear the queue since phase failures will lead to head-on collision conflicts. For the first factor, assuming 2-s vehicle clearing headway, the time savings of the contraflow lane is $2 \times x$, where x is the queue length by the number of vehicles. Thus the truncated green time should be equal to or less than $2 \times x$, as shown in figure 112 and figure 113.

Considering the minimum green time constraint, using 2-s start up lost time, the minimum green time should be $2 \times x + 2$. The critical shortest value in the equations in figure 112 and figure 113 should be used as truncated green time.

$$\text{Truncated green time} \leq 2 \times x$$

Figure 112. Equation. Truncated green time based on vehicle clearance.

$$\text{Original green time} - \text{Truncated green time} \geq 2 + 2 \times x$$

Figure 113. Equation. Truncated green time based on original green time.

A safety concern is that distracted drivers might not discharge from the CLT pocket during the current green phase. This creates head-on collision conflicts. A potential signalization-based counter-measure is to provide a green extension whenever un-cleared vehicles are detected in the contraflow area.

After an intersection approach has been identified as a CLT candidate, the following signalization and lane length design steps could be followed:

1. Optimize the cycle length and other signal timings based on existing non-CLT conditions using peak-hour traffic volumes.

2. Calculate the 95th percentile LT queue length (ft) using equations from the 2000 HCM and use $1/(N + 1)$ of it as the desired CLT lane length where N is the number of existing LT lanes.(73)

3. Check the minimum green time constraint and determine how much LT green time can be truncated.

4. Determine if the cycle length can be truncated. Truncated green times should be assigned to oversaturated TH movements but without shortening the cycle length. If the TH movement is undersaturated, the cycle length can be shortened depending on the truncated green time duration.

Signing and Pavement Marking

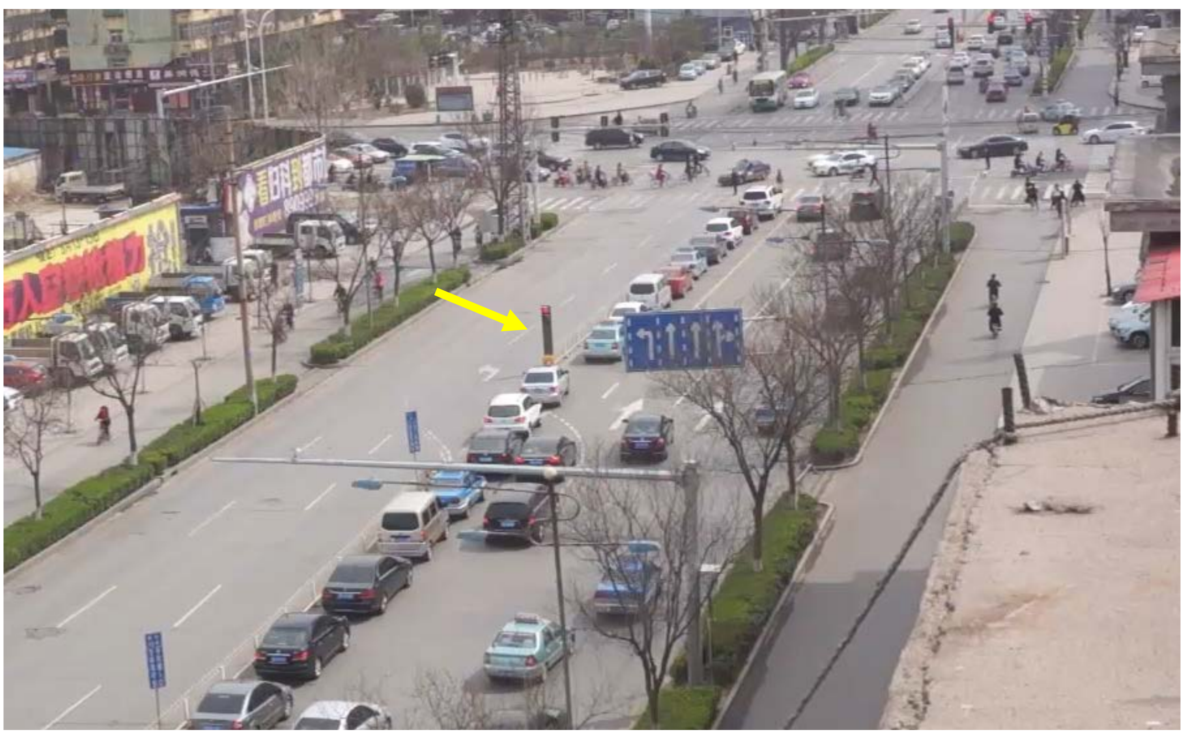
In addition to the presignal at the entrance, CLT pockets should be controlled by lane open/close signs. Conflicts between adjacent LT movements can be avoided by using pavement markings. These markings would route left-turning traffic to the right-most lane instead of the CLT lane (see figure 108). Regarding the safety issue mentioned previously (distracted drivers not discharging during the green phase), another possible countermeasure would be the use of special signage to remind drivers to pay attention.

Implementations in China

All information in this subsection was derived from unpublished interviews that the research team conducted with Chinese transportation officials. As illustrated in figure 114, there are believed to be at least 50 real-world implementations of CLT intersections in at least 9 Chinese cities.

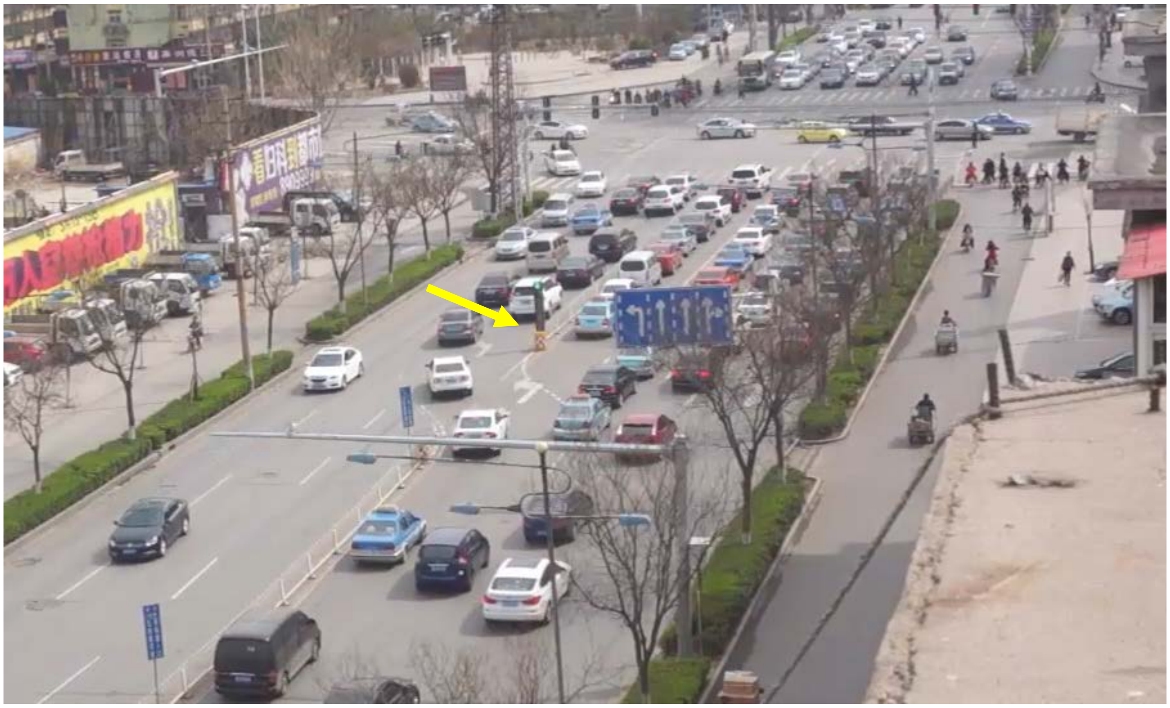
Figure 114. Map. CLT implementations in China.

Figure 115 through figure 117 illustrate real-world implementations of CLTs in China. They show left-turning vehicles before, during, and after their entry into the contraflow lanes.



Source: Professor Pan Liu of Southeast University School of Transportation with arrow overlay added by Leidos, Inc. to indicate the presignal location.

Figure 115. Photo. CLT intersection in China before the presignal has turned green.



Source: Professor Pan Liu of Southeast University School of Transportation with arrow overlay added by Leidos, Inc. to indicate the presignal location.

Figure 116. Photo. CLT intersection in China after the presignal has turned green but before the main signal has turned green.

Source: Professor Pan Liu of Southeast University School of Transportation with arrow overlay added by Leidos, Inc. to indicate the main signal location.

Figure 117. Photo. CLT intersection in China after the main signal has turned green.

The first real-world implementation is believed to be in Handan, China.[83] In 2014 and 2015, a total of 15 intersections and 45 intersection approaches in Handan were reconfigured for CLT operation. These CLT pockets were approximately 210 ft long. Throughput increases were measured at 720 vehicles/h.

Additional real-world implementations were constructed in Shenzhen and Hohhot, China.[84–86] Since August 2014, 10 intersections in Shenzhen have been reconfigured for CLT operation. These CLT pockets were approximately 330 ft long. Capacity increases were measured at 30 to 40 percent for LT movements and 6 percent for CLT intersections as a whole. Since September 2015, 18 intersections in Hohhot have been reconfigured for CLT operation. These CLT pockets were approximately 330 ft long. Capacity increases were estimated at 80 percent for LT movements. The Hohhot implementations were unique in regards to their automatic retractable gates. The retractable gates automatically open at the outset of morning/afternoon peak periods and automatically close at the outset of off-peak periods.

In December 2015, one intersection in Nanning, China, was reconfigured for CLT operation.[84] The CLT pocket was approximately 330 ft long. This implementation was reportedly not well-received by drivers. Some were concerned that driver avoidance of the CLT pockets would instead increase congestion. The Nanning implementation was unique in regards to the intersection being T-shaped with only three approaches. In November 2015, one intersection in Jiaozuo, China, was reconfigured for CLT operation.[87] The CLT pocket was approximately 165 ft long. No before-and-after operational studies were available for this implementation. The Jiaozuo implementation was unique in regards to its integrated LED signal and DMS.

CHAPTER 6. CONCLUSIONS

The *TTI's 2012 Urban Mobility Report* highlights a number of disturbing trends related to degradation of surface transportation.(1) Each of these trends could be highly detrimental to the quality of life for U.S. citizens. U.S. traffic congestion is worsening, and the resulting economic damages are increasing. Environmental impacts are accelerating the economic damages from traffic congestion. Reliability of surface transportation is decreasing, requiring drivers to depart earlier to ensure on-time arrival at their destinations.

Mitigation or elimination of traffic bottlenecks is believed to be a top priority. This report was intended to update the traditional approaches to congestion and provide new solutions. One of the modernized approaches was a comprehensive framework for congestion and bottleneck treatments developed for this report. The framework begins with a thorough definition of terminology and proceeds to a flowchart (playbook) of bottleneck classifications. Within the 7 main classifications are 70 possible solutions. The appendix of this report provides technical details on the 70 playbook solutions.

Despite the ease-of-use of the playbook, certain strategies have a strong tendency to outperform others. This report presents additional research on a much smaller set of bottleneck mitigation strategies, which are expected to be cost effective. For this small set of featured strategies, micro-simulations and B/C analyses were performed. The report further encourages States to consider alternative interchange/intersection designs (i.e., DDI, DLTI, RCUT, MUT, DLT, and roundabouts), provides a quick summary of their benefits, and provides references for additional information.

Beyond the bottleneck mitigation strategies, this report recognizes a need for improved methods of congestion identification to justify transportation investments. New methods of traffic measurement and modeling (e.g. INRIX®, 2016 HCM, etc.) provide rich sources of data for congestion analysis, accounting for influential factors (e.g., weather, incidents, demand variability, etc.) in a more robust and explicit manner.(45) STM quantifies congestion visually. Performance measures derived from the STM can further quantify congestion statistically. The STMs and related performance measures, shown in this report as case studies, were generated by software tools. These tools will be available for interested States and transportation departments and will hopefully lead to more scientific assessments of congestion in some areas.

APPENDIX. PLAYBOOK OF BOTTLENECK SOLUTIONS

To solve the U.S. traffic congestion problem, mitigation or elimination of bottlenecks is believed to be a top priority. This report was intended to update the traditional approaches to congestion and provide new solutions. One of the modernized approaches is a comprehensive framework for congestion and bottleneck treatments developed for this report. The framework begins with a thorough definition of terminology and proceeds to a flowchart (playbook) of bottleneck classifications. The flowchart is shown in figure 29 in chapter 3. Within the 7 main classifications are 70 possible solutions, which are described further in this appendix.

(1) GEOMETRIC—ROADWAY SPECIFIC

1. Design Speed

a. **Description/Definition of the Element**: Design speed is a selected speed used to determine the various geometric design features of the roadway.(2) Research confirms that lower speeds are safer and lowering speed limits can decrease both crash frequency and severity. There are two types of design speed as follows:
   i. **Designated design speed**: The speed established as part of the geometric design process for a specific segment of roadway.
   ii. **Inferred design speed**: The maximum speed for which all critical design speed-related criteria are met at a particular location.

b. **Theoretical/Empirical Effects:**
   i. Speed-flow-density relationships are well established in the literature; speed is inversely related to density, which is used in the 2010 HCM as the measure of quality of traffic service for freeways and multilane highways.(6) Speed and flow are related by a quadratic relationship, such that flow is maximized at an optimal speed above or below which flow decreases.(63,88)
   ii. Speed is used directly as a measure of mobility and facilities that emphasize mobility (highways/freeways) accommodate high speed.(63) As a result, lower speed causes lower flow, resulting in diminished capacity.

c. **Existing Solutions**: In general, speeds can be reduced/increased by changing the posted speed limit, geometric, and cross-sectional elements, or a combination of the following:
   i. Variable message signs.
   ii. General VSLs (which may include volume, weather, or incident responsive).
   iii. Increased lane and shoulder widths (which will impact operating speeds).
   iv. Improved access management to decrease interchange or intersection density.
   v. Reduced excessive grades.
   vi. Lengthening of vertical curve lengths.
   vii. Increased horizontal curvature radius or increased superelevation where reasonable and safe.

d. **New Solutions**: New solutions include the following:
    i. Speed harmonization.
    ii. More frequent VSL sign spacings.
    iii. Speed limit specification by lane.

2. Number of Lanes

a. **Description/Definition of the Element**: Number of lanes is the available lanes in a particular direction for drivers to use.
b. **Theoretical/Empirical Effects**: The number of lanes is directly related to flow and determines the capacity of the roadway; however, if the lane is an auxiliary lane, the utilization may be low, thereby producing partial benefits.
c. **Existing Solutions**: Existing solutions include the following:
    i. Plus lane (left/right).
    ii. Restripe narrower lanes to create new lane(s).
    iii. Build new lanes.
    iv. Managed lanes, including the following:
        1. Contraflow lanes.
        2. Reversible lanes.
        3. Bus-only or transit only lanes.
        4. High-occupancy toll (HOT)/HOV lanes.
    v. HSR.
    vi. Dynamic lane control or moveable barrier separated lane control.
    vii. Shoulder conversion (i.e., permanent conversion of left and/or right shoulders to a travel lane).
d. **New Solutions**: New solutions include the following:
    i. Electronic pavement marking on roadways.
    ii. Dynamic lane assignment for auxiliary lanes (i.e., convert option lanes between dedicated exit versus dedicated TH lanes based on DMS displays).

3. Lane Width

a. **Description/Definition of the Element**: This is defined as the width of driving lanes. The standard lane width on most roadways is 12 ft.
b. **Theoretical/Empirical Effects**: Narrow lanes reduce driver comfort, which cause drivers to slow down, reducing flow and capacity, particularly in locations with high volumes of oversize vehicles and large trucks. Other effects are as follows:
    i. Narrow lane widths increase steering workload and reduce speeds.[89]
    ii. Effective lane width reduction can reduce speeds by up to 7 percent.[90]
    iii. Drivers implement direct control of lateral error for extreme conditions of narrow lane width and high speeds.[91]
    iv. Extra pavement contributes to the total amount of pavement width, which decreases drivers' uncertainty and leads to higher speeds.[92]
    v. Lane width may increase the passenger car unit (capacity) of a roadway because of increased freedom of movement on wider roads.[93]

c. **Existing Solutions**: Existing solutions include the following:
    i. Widen lanes.
    ii. Auxiliary lanes.
    iii. Paved shoulder (right/left).
    iv. Heavy vehicle restriction.
d. **New Solutions**: New solutions include the following:
    i. **Static pavement marking changes**: Change the lane width based on lane utilization on the road. For example, the rightmost lane dedicated to trucks could be wider than the middle lane(s) for passenger cars on the same road.
    ii. **Electronic pavement marking on roadways**: Create pavement markings on roadways.

4. **Shoulder Width and Type**

    a. **Description/Definition of the Element**: It is the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles for emergency use and for lateral support of the base and surface courses.(94)
    b. **Theoretical/Empirical Effects**: Narrow shoulder widths might reduce driver comfort, causing drivers to reduce speeds, which results in reduced flow and capacity. Additional effects are as follows:
        i. As with pavement widths, the findings on shoulder width and crash rates are mixed. Similarly, some studies found no relationship between the right shoulder width and crash rates, while others have concluded that crash rates decreased as shoulder width increased.(95)
        ii. Extra pavement contributes to the total amount of pavement width which decreased drivers' uncertainty and led to higher speeds.(92)
    c. **Existing Solutions**: Existing solutions include the following:
        i. Provide and/or upgrade shoulders.
        ii. Widen shoulders.
        iii. Paved shoulder (right/left).
        iv. Auxiliary lanes using shoulder widths.
        v. HSR.
    d. **New Solutions**: A new solution includes the following:
        i. Electronic pavement marking on roadways.

5. **Lane Drops**

    a. **Description/Definition of the Element**: A dropped lane is a TH lane that becomes a mandatory turn lane on a conventional roadway or a TH lane that becomes a mandatory exit lane on a freeway or expressway. The end of an acceleration lane and reductions in the number of TH lanes that do not involve a mandatory turn or exit are not considered dropped lanes.(79)
    b. **Theoretical/Empirical Effects**: Inversely related to flow, a decrease in the number of TH lanes decreases the TH movement capacity on the roadway.

i. Using data from a freeway lane drop in Minneapolis, it was shown that the discharge flow was about 10 percent lower than the prevailing flow observed prior to queue formation.[96]

c. **Existing Solutions**: Existing solutions include the following:
   i. Advance lane drop warning.
   ii. Late merge techniques.
   iii. Dynamic merge techniques.
   iv. Change/optimize the location of lane drops.
   v. Avoid lane drops at low-volume exit ramps.
   vi. Run-by lanes.

d. **New Solutions**: A new solution includes the following:
   i. Dynamic lane use assignment using electronic pavement markings.

6. Lane Reduction Transition

a. **Description/Definition of the Element**: Lane reduction transitions are significantly different from lane drop situations because a lane reduction transition occurs between interchanges or between intersections. As a result, all vehicles in the ending lane must merge into the adjacent lane, while at a lane drop, vehicles that are unable to leave the lane have the ability to stay in the lane and proceed to the exit or turn.[79]

b. **Theoretical/Empirical Effects**: Inversely related to flow, a decrease in the number of lanes decreases the capacity of the roadway.

c. **Existing Solutions**: Existing solutions include the following:
   i. Advance lane drop warning.
   ii. Late merge techniques.
   iii. Dynamic merge techniques.
   iv. Change/optimize the location of lane drops.

d. **New Solutions**: None reported.

7. Horizontal Clearance

a. **Description/Definition of the Element**: Horizontal clearance is the lateral offset distance from the edge of the traveled way, shoulder, or other designated point to a vertical roadside element.[97]

b. **Theoretical/Empirical Effects**: Reduced horizontal clearance may reduce driver comfort and cause drivers to reduce speed, which decreases flow and capacity.[97]

c. **Existing Solutions**: Existing solutions include the following:
   i. Increase horizontal clearance.
   ii. Remove any side blockage.
   iii. Control roadway slopes to allow better vision.
   iv. Create higher vertical clearances.
   v. Cut back on vegetation.

d. **New Solutions**: A new solution includes the following:
   i. Electronic pavement marking on roadways.

8. Vertical Clearance

a. **Description/Definition of the Element**: Vertical clearance is the maximum vertical height for a particular roadway and overpass clearance.
b. **Theoretical/Empirical Effects**: Even with sufficiently tall clearances, drivers may reduce speeds when light or moisture creates an optical illusion of a shorter clearance. The effect of vertical clearance is more pronounced for heavy vehicles.
c. **Existing Solutions**: Existing solutions include the following:
    i. Ensure placement of vertical clearance signs on structures.
    ii. Mill pavement when necessary to avoid excessive layers of pavement over time when resurfacing.
    iii. Ensure adequate design for overpasses.
d. **New Solutions**: A new solution includes the following:
    i. **Vehicle height confirmation**: Provide visual or audible confirmation through in-vehicle communication or roadside equipment that vehicle height does (or does not) exceed clearance.

9. Sun Glare

a. **Description/Definition of the Element:** Sun glare is the alignment of the road to environmental conditions including cardinal direction with respect to the possibility of sun glare.
b. **Theoretical/Empirical Effects:** Sun glare is shown to have a statistically significant impact on mean traffic speeds.[(98)]
    i. Sunrise/sunset is listed as a distraction category in the Crashworthiness Data System and may contribute to crashes.[(99)]
c. **Existing Solutions**: Existing solutions include the following:
    i. Roadway realignment.
    ii. Backplates for traffic signals.
    iii. Additional redundant TCDs.
d. **New Solutions**: New solutions include the following:
    i. Provide (visual or audible) confirmation of roadside conditions through in-vehicle communications or roadside equipment.
    ii. Connected in-vehicle information on position of leading vehicles is not easily visible.

10. Horizontal Alignment

a. **Description/Definition of the Element**: In the design of highway alignment, it is necessary to establish the proper relation between design speed and curvature. The two basic elements of horizontal curves are curve radius and superelevation.[(100)]
b. **Theoretical/Empirical Effects:** Curvature on the roadway causes reduced speeds and may limit stopping sight distance, which further reduces travel speeds.
    i. Deflection angle is the horizontal curve variable that best contributes to explaining the variation in speeds on a horizontal curve. High deflection angle values are associated with lower speeds.[(101)]

  ii. Smaller curve radii result in poorer steering competence, which is compensated for by choosing a lower speed.[102]

c. **Existing Solutions**: Existing solutions include the following:
  i. Increasing the horizontal curvature if it is applicable.
  ii. Auxiliary/shoulder/plus lanes.
  iii. Truck/heavy vehicle restrictions.
  iv. Paved shoulder (right/left).
  v. Basic traffic signs and markings, enhanced/additional traffic control devices, rumble strips, minor roadway improvements, and innovative and experimental treatments.[103]
  vi. Static dual-advisory speed signing and pavement markings on freeway-to-freeway connectors.
  vii. Sequential dynamic curve warning systems.[104]

d. **New Solutions**: New solutions include the following:
  i. Dynamically triggered overhead or roadside mounted signs encouraging drivers to maintain posted or advisory speeds on curved sections when free flow traffic conditions prevail.
  ii. Connected in-vehicle information on position of leading vehicles not easily visible.

11. Vertical Alignment/Grades

a. **Description/Definition of the Element:** Vertical curves are used to transition from one grade (i.e., slope in the direction of travel) to another.[88] It also refers to the grading angles on the roadway.

b. **Theoretical/Empirical Effects:** Increased gravity causes speed reductions on uphill grades and speed increases on downhill grades. On downhill sections, drivers may brake to compensate. In general, heavy vehicles travel slower on uphill grades than on downhill grades due to the physical characteristics of those vehicles (e.g., power of engine, weight, braking ability, etc.).
  i. Vertical alignment affects both driver comfort and sight distance. Crest vertical curves can limit sight distance by restricting a driver's line of sight, but where vertical curves are long enough to provide adequate sight distance, they will also provide adequate driver comfort.[88]

c. **Existing Solutions**: Existing solutions include the following:
  i. Reducing steep grades.
  ii. Warning signs and flashing lights.
  iii. Careful placement of TCDs around steep vertical grades.

d. **New Solutions**: New solutions include the following:
  i. Dynamically triggered overhead or roadside-mounted signs encouraging drivers to maintain posted or advisory speeds on grades when free-flow traffic conditions prevail.
  ii. Connected in-vehicle information on position of leading vehicles not easily visible.

12. Stopping Sight Distance

a. **Description/Definition of the Element**: Stopping sight distance is defined as the distance needed for drivers to see an object on the roadway ahead and bring their vehicles to a safe stop before colliding with the object.(105)
b. **Theoretical/Empirical Effects**: Insufficient stopping sight may cause sudden vehicles brake, shock waves, and/or collision. Physical barriers, roadway alignments, curves, time of day, and other factors affect the stopping sight distance.(105)
c. **Existing Solutions**: Existing solutions include the following:
   i. Improve stopping sight distance.
   ii. Reduce speed limits.
   iii. ITS—queue warning.
   iv. Reduce steep grades.
   v. Create higher vertical clearances.
   vi. Cut back on vegetation.
d. **New Solutions**: A new solution includes the following:
   i. ATM solutions.

13. Pavement Friction

a. **Description/Definition of the Element**: Pavement friction is determined by the roughness level of the roadway surface pavement.
b. **Theoretical/Empirical Effects**: Lower pavement friction (e.g., when pavement is wet) may cause driver discomfort that results in speed reduction.
   i. Higher pavement friction results in fewer crashes stemming from wet pavement.(106)
   ii. Drivers on a curve experience unbalanced lateral acceleration (i.e., side friction), which is a direct indication of driver discomfort. Observed speeds on horizontal curves routinely exceed designated design speeds. The maximum friction factors used for the design speeds are intended to be adequate even under poor conditions; therefore, in average conditions, higher speeds can be attained. Steeper curves require a higher friction factor, which lowers the design speed.(88)
c. **Existing Solutions**: Existing solutions include the following:
   i. Warning signs.
   ii. Reduce speed limit.
   iii. Reducing excessive grades.
   iv. Weather responsive speed limits.
   v. Pavement techniques that increase friction include grooved, textured, or open-graded pavement. Grooving longitudinally reduces the noise while driving.(105)
d. **New Solutions**: A new solution includes the following:
   i. Installing dynamically triggered overhead or roadside-mounted signs encourages drivers to maintain posted or advisory speeds based on pavement condition.

14. Cross Slope

a. **Description/Definition of the Element**: The cross section slope (perpendicular plane to the flow of vehicles) mainly assists in rainfall drainage.
b. **Theoretical/Empirical Effects**: Improper cross slope may cause speed reductions during inclement weather conditions because of insufficient drainage.
c. **Existing Solutions**: Existing solutions include the following:
    i. Maintain a cross slope along the roadway.
    ii. Add more pipes for water storm drainage.
    iii. Adjustment of the high-side shoulder cross slope to avoid melting snow and ice draining into roadway.(105)
d. **New Solutions**: A new solution includes the following:
    i. Installing dynamically triggered overhead or roadside-mounted signs encourages drivers to maintain posted or advisory speeds based on pavement condition.

15. Superelevation

a. **Description/Definition of the Element** Superelevation is the rotation of the pavement on the approach to and through a horizontal curve. Superelevation is intended to assist the driver by counteracting the lateral acceleration produced by tracking the curve. It is expressed as a decimal, representing the ratio of the pavement slope to width and ranging from 0 to 0.12 ft/ft. The adopted criteria allow for the use of maximum superelevation rates from 0.04 to 0.12. Maximum superelevation rates for design are established by policy by each State.(105)
b. **Theoretical/Empirical Effects:** Insufficient superelevation requires vehicles to slow down, and excessive superelevation may reduce driver comfort. In addition, some curves are not provided with superelevation, which leads to severe reduction of speed to prevent crashes or getting off the road.
    i. Superelevation allows a driver to negotiate a curve at a higher speed than would otherwise be comfortable.(107)
c. **Existing Solutions**: Existing solutions include the following:
    i. Reduce speed limit for poorly designed superelevation.
    ii. Design superelevation specifically for the site.(107)
    iii. Appropriate signage for drivers.
d. **New Solutions**: A new solution includes the following:
    i. Installing dynamically triggered overhead or roadside-mounted signs encourages drivers to maintain posted or advisory speeds based on pavement condition.

16. Access Density

a. **Description/Definition of the Element:** Access density is the number of access points and driveways per unit distance.(108)
    i. **Access point**: An access point is an entrance, driveway, street, private drive, turnout, or other means of providing for the movement of vehicles to or from the public roadway network.(109)

    ii. **Driveway**: A driveway serves a single use on one property or multiple uses on more than one property.[109]

b. **Theoretical/Empirical Effects:** Traffic friction from entering/exiting vehicles creates turbulence in the traffic flow.
    i. Speeds decrease as the number of access points increases.[101,108]

c. **Existing Solutions**: Existing solutions include the following:
    i. Better access management, fewer access points, access point consolidation, better access point placement, better roadway signing, etc.
    ii. Access roads, feeder roads, and limited access roads.
    iii. Signals and other traffic control devices.
    iv. Alternative intersection designs should be considered at key access points.

d. **New Solutions**: A new solution includes the following:
    i. Dynamic ramp metering at access points.

17. Midblock Pedestrian Crossings

a. **Description/Definition of the Element**: A midblock pedestrian crossing is a pedestrian crossing at designated pedestrian crosswalks.

b. **Theoretical/Empirical Effects**: Lower speeds occur as pedestrian activity increases.

c. **Existing Solutions**: Existing solutions include the following:
    i. Marked crosswalks for better driver yielding compliance, including use of rectangular rapid flashing beacons.
    ii. Walk symbols that give enough time to cross.
    iii. Enforcement.
    iv. More meaningful positioning (e.g., crosswalks where people want to cross naturally) of midblock crosswalks in high-traffic areas.
    v. Bulbs, chokers, and neck downs, which all extend the sidewalk into the street to slow traffic and guide pedestrians.[110]

d. **New Solutions**: None reported.

18. Medians

a. **Description/Definition of the Element**: Medians constitute a portion of a highway separating opposing directions of the traveled way.[63,111]

b. **Theoretical/Empirical Effects**:
    i. The presence of a median contributes to higher speeds than when no median is present.[101]
    ii. When present, a raised, depressed, or two-way LT lane median results in slightly lower speeds.[108]
    iii. Medians can be designed to increase speeds, lower speeds using traffic calming islands or medians, or separate high-speed roadways.

c. **Existing Solutions**: Existing solutions include the following:
    i. Use for separation.
    ii. Use for traffic calming.
    iii. Use for pedestrian refuges.
    iv. The presence of trees slows drivers as well.[65]

d. **New Solutions**: None reported.

19. Lighting

a. **Description/Definition of the Element**: It is necessary to have sufficient roadway lighting for driving during dark conditions.
b. **Theoretical/Empirical Effects**: Insufficient lighting may cause drivers to reduce speed and use extra caution, reducing flow and capacity. Drivers may also misread or fail to observe a TCD indication causing incidents. Lighting generally improves sight distance and results in better safety performance.
c. **Existing Solutions**: Existing solutions include the following:
    i. Improve lighting and reflectivity of roadway signs and pavement markings.
    ii. Provide (visual/audible) confirmation through in-vehicle equipment to inform driver of roadside conditions.
d. **New Solutions**: A new solution includes the following:
    i. Installing dynamically triggered overhead or roadside mounted signs encourages drivers to maintain posted or advisory speeds based on lighting conditions.

20. Presence of a Centerline or Edgeline Markings

a. **Description/Definition of the Element**: Centerline and edgeline road markings indicate traffic lanes, oncoming traffic, passing rules, and other traffic laws.
b. **Theoretical/Empirical Effects**:
    i. Absence of centerline or edgeline markings is associated with lower speeds.[108] Likewise, the presence of edgelines and centerlines is associated with higher travel speeds.[112]
    ii. Road lines may not be visible in rain and snowy conditions.
    iii. Centerline and edgelines guide drivers and reduce congestion and raise roadway capacity.[112]
    iv. Lane markings reduce crashes.[112]
c. **Existing Solutions**: Existing solutions include the following:
    i. Uniform colors and meanings for better driver compliance.
    ii. Durability with high-quality paint.
    iii. Adding edgelines to rural roads.
    iv. Better reflectivity and frequent reflectivity assessment (because pavement markings are most effective when there is poor visibility).[112]
d. **New Solutions**: A new solution includes the following:
    i. Electronic pavement marking on roadways.

21. Bicycle Lanes

a. **Description/Definition of the Element:** A restricted right-of-way that utilizes city streets, secondary roads, and other existing facilities and is appropriately designated by signs, lane markings, and/or physical barriers, such as guardrails, special fencing, curbed, or parked vehicles.[113]

b. **Theoretical/Empirical Effects:** Drivers may reduce speed around bicycles when there is no bike lane but, if provided with adequate width, may not need to adjust speeds.
    i. Bicycle lanes, when placed on streets without parking and of adequate width, appear to have little detrimental effect on traffic flow and may serve the beneficial purpose of providing better separation between vehicles and bicycles.(113)
    ii. The presence of bicycles negatively affects auto flow (in terms of speed reduction and lateral movement) by limiting the usable roadway width. These negative impacts seem to be reduced or removed if a bike lane or a wide enough curve lane is provided.(114)

c. **Existing Solutions**: Existing solutions include the following:
    i. Widen or restripe lanes, create median refuges at street crossings, add bike-friendly signals, provide new bike trails in new communities where possible, and/or install bike-sensitive detectors.(115)
    ii. Shared lanes, wide outside lanes, bike lanes, shoulder, and separate bike path.(116)

d. **New Solutions**: A new solution includes the following:
    i. Electronic Pavement Marking on Roadways to dynamically include bike lanes and dynamically change the width and/or separation of bike lanes to make them safer.

22. Separation Type of Managed Lanes

a. **Description/Definition of the Element**: Separation lines include a set of lanes with access restrictions (e.g., vehicle occupancy or tolled) located within a freeway but separated from the general purpose lanes.

b. **Theoretical/Empirical Effects**: By separating traffic between the managed and general purpose lanes, users in the managed lanes will experience an increased LOS over the general purpose lanes while at the same time, the decrease in demand for the general purpose lanes will increase the LOS for the general purpose lanes.

c. **Existing Solutions**: Existing solutions include the following:
    i. Managed access points.
    ii. Movable separation barriers.
    iii. ITS monitoring.
    iv. Electronic toll collection (if appropriate).

d. **New Solutions**: None reported.

(2) GEOMETRIC—FACILITY SPECIFIC

1. Bridges

a. **Description/Definition of the Element:** A structure carrying a roadway over waterways or other surface feature including other roads, railroads, or other obstacles.

b. **Theoretical/Empirical Effects**: Drivers may reduce speed to use extra caution or use overload bypass routes. The effect may be more pronounced for heavy vehicles.
    i. Performance changes near temporary bridges demonstrate the effect of bridges on traffic flows. Lower speeds and higher average headways limit volume and throughput at the bridge.(117)
    ii. A narrowed cross section (of a bridge) can make some drivers uncomfortable and cause them to dramatically reduce speed, degrading operations on high-speed, high-volume facilities, inadequate space for storage of disabled vehicles, enforcement activities, emergency response, or maintenance work.(105)
c. **Existing Solutions**: Existing solutions include the following:
    i. Restriping to decrease lane width and add lanes.(118)
    ii. Widening lanes.
    iii. Truck restrictions.
    iv. Traffic diversion.
    v. Build shoulders or medians on bridges for stopped and disabled vehicles and accidents.(119)
    vi. Use ITS to alert drivers to bridge openings and/or sudden stops.(119)
    vii. Reduce grades at bridge entrances to increase sightlines.(119)
    viii. Discourage lane changing using pavement markings.
    ix. Contraflow lanes—use of moveable barrier systems to change cross section by time of day.
    x. Reversible lanes.(120)
    xi. Congestion pricing to manage demand.
d. **New Solutions**: New solutions include the following:
    i. Installing dynamically triggered overhead or roadside mounted signs encourages drivers to maintain posted or advisory speeds based on lighting conditions.
    ii. DRLT lanes.(82)

2. Tunnels and Underpasses

a. **Description/Definition of the Element**: A tunnel is a roadway that traverses waterways or other bodies by going underneath. An underpass is the roadway that goes under a bridge or other structure.
b. **Theoretical/Empirical Effects**: Drivers may reduce speed or use overload bypass routes. Even sufficiently tall clearances may reduce speeds when light or moisture creates an optical illusion of a shorter clearance and the effect of vertical clearance is more pronounced for heavy vehicles.
    i. Congested flow at most sections of a tunnel has relatively little to do with the tunnel environment but rather is a consequence of bottlenecking or queuing processes because vehicle passing, merging, and diverging are less common in tunnels. Tunnel flow is one of the least complex of all flow situations.(121)
    ii. When lane widths decrease on surface roads, traffic slows. Traffic also slows when ceiling heights decrease.(122)

iii. Obstacles that are close to travel lanes (bridge abutments, piers, sign structures) cause driver slowing due the shy distance.

c. **Existing Solutions**: Existing solutions include the following:
   i. Adequate lighting.
   ii. Adequate lane widths.
   iii. Signage indicating actual height above each lane to allow drivers of tall vehicles to select the appropriate lane. Include advance signage to warn drivers to move to high-clearance lane. Knowledge of actual clearance gives driver confidence and avoids panic breaking.
   iv. Restrict lane change and passing.
   v. Add lane using vertical bridge abutments (e.g., add lane between bridge pier and abutment replacing slope paving).
   vi. Higher ceilings.
   vii. Congestion pricing to manage demand.
   viii. Move tolls booths away from entrances or exits.

d. **New Solutions**: A new solution includes the following:
   i. Mainline metering similar to freeway-to-freeway metering (i.e., ramp metering technology applied to the main area of roadways).(123)

3. Collector-Distributor Network

a. **Description/Definition of the Element**: A collector-distributor network is a network of roads that efficiently distributes cars from smaller roads that have access to residential neighborhoods to larger roads with limited access points. This network consists of low to moderate capacity roads that serve to move traffic from local streets to arterial roads. In some States, such as Texas, collector-distributor roadways are a supplemental facility between freeway main lanes and the frontage roads. Their primary purpose is to move the weaving and lane changing away from the high-speed traffic on the freeway main lanes.(124)

b. **Theoretical/Empirical Effects**: Entering/exiting vehicles to/from the collector-distributor create friction, which causes speed reduction. Collector-distributor facilities can allow a single freeway exit ramp to distribute vehicles to two or more crossing arterials or collect vehicles from several crossing arterials so that they can enter the freeway at a single entrance ramp.

c. **Existing Solutions**: Existing solutions include the following:
   i. Limits to local traffic only.
   ii. Limiting access.

d. **New Solutions**: None reported.

(3) GEOMETRIC—SPECIFIC TO INTERCHANGES: FREEWAY TO FREEWAY

1. Diverge Sections and Merge Sections

a. **Description/Definition of the Element**: Diverge and merge sections consist of areas for exiting highways (e.g., off-ramps and areas for entering highways, on-ramps, and where highways join or split).(6)

b. **Theoretical/Empirical Effects**: Merging and diverging segments create confusion and chaotic movements often resulting from swift lane changing.[6]
c. **Existing Solutions**: Existing solutions include the following:
    i. Ramp metering.
    ii. Swift lane changing.
    iii. Appropriate signage.
    iv. Appropriate Interchange spacing.
    v. Ramp modification—reversal (i.e., switching entrance and exit ramp pattern).
    vi. Ramp modification—closure (i.e., permanent versus time of day).
    vii. Option lanes.
    viii. Proper lane balance and basic lanes (may be able to reduce number of basic lanes on TH roadways to allow more lanes for merging traffic from high-volume direct connect ramps).
d. **New Solutions**: New solutions include the following:
    i. Mainline metering.[123]
    ii. DMC (or junction control) to regulate or close specific lanes upstream of an interchange.[125]

2. Auxiliary Lanes

a. **Description/Definition of the Element**:
    i. An auxiliary lane is defined by AASHTO as the portion of the roadway adjoining the traveled way for speed change, turning, weaving, truck climbing, maneuvering of entering and leaving traffic, and other purposes supplementary to TH traffic movement.[62] Auxiliary lanes are used to balance the traffic load and maintain a more uniform LOS on the highway. They facilitate the positioning of drivers at exits and the merging of drivers at entrances.[24]
    ii. The portion of the roadway adjoining the traveled way for speed change, turning, weaving, truck climbing, maneuvering of entering and leaving traffic, and other purposes supplementary to TH traffic movement.[126]
b. **Theoretical/Empirical Effects**:
    i. Operational efficiency may be improved by using a continuous auxiliary lane between the entrance and exit terminals where interchanges are closely spaced.[24]
    ii. These can be effectively used to manage traffic but not to necessarily increase capacity. Auxiliary lanes may be used to allow drivers entering roadways additional time to merge into traffic.
    iii. Auxiliary lane use by TH vehicles decreases with an increase in right-turning vehicles (RT volume greater than 15 percent of the approach volume renders the auxiliary lane useless to TH vehicles) unless there is a separate RT lane.
    iv. Auxiliary lane length, intersection delay, and the proportion of right-turning vehicles work together in determining the utility of an auxiliary TH lane.

v. For intersections, the HCM default value for the lane utilization factor is 0.91for a three-lane group.[127]
vi. The proper use of auxiliary lanes increases the average speed on freeways, reduces delay on ramps, and reduces the number of conflicts between slow speed and higher speed vehicles.[128]

c. Existing Solutions: Existing solutions include the following:
i. Build auxiliary lanes.
ii. Increase merging area.
iii. Ensure appropriate lengths for acceleration and deceleration segments.
iv. Provide managed lanes.

d. New Solutions: None reported.

3. Weaving Areas

a. Description/Definition of the Element: A weaving area is roadway segment where a merge and diverge in close proximity require either merging or diverging vehicles to execute one or more lane changes.[129]
i. Weaving is generally defined as the crossing of two flows in the same general direction without the aid of traffic control devices.[130]

b. Theoretical/Empirical Effects: Increased lane changing maneuvers creates an increased number of potential conflicts and drivers may reduce speed to accommodate.
i. The number of lanes is the most critical factor in the determination of the capacity of weaving sections.[131]
ii. Shorter weaving sections will begin to break down at relatively lower volumes than longer weaving sections, and weaving speeds fall as weaving volumes increase.[132]
iii. Total number of lane shifts required by drivers in weaving sections affects both weaving and nonweaving speeds.[133]
iv. There is no relationship between speed and weaving flow rate in practice.[130]

c. Existing Solutions: Existing solutions include the following:
i. Minimize weaving impact by lengthening auxiliary lanes and intersection/interchange spacing.[130]
ii. Ramp metering.[130]
iii. Lengthen exit ramps instead of only auxiliary lanes.[130]
iv. Make changes to weaving section type.
v. Prohibit weaving movement.
vi. Eliminate left-side ramps.

d. New Solutions: None reported.

4. On-Ramp/Off-Ramp

a. Description/Definition of the Element: An on- or off-ramp is a ramp allowing access to or from a freeway.

b. **Theoretical/Empirical Effects**: Adding new traffic volume to existing traffic disturbs the flow of vehicles, increases demand, and consequently reduces travel speeds.
   i. Active bottlenecks arise whenever flows from an upstream on-ramp reach a certain peak. Average queue discharge rates were observed to decrease up to five percent than the flows before bottleneck activation.(134)
   ii. Discharge flow diminishes when a merge becomes an active bottleneck.(135)
   iii. An interchange's high off-ramp volume, in conjunction with peak-hour arterial flow rates, will cause some intersections to become oversaturated, consequently paralyzing traffic movements along the entire arterial.(136)
   iv. An oversaturated arterial adjacent to the interchange can cause off-ramp queue lengths to extend beyond their auxiliary lanes and block the freeway's mainlines.

c. **Existing Solutions**: Existing solutions include the following:
   i. On-ramp metering.(137)
   ii. Integrate with arterial signal controls.
   iii. Maintain proper ramp spacing.

d. **New Solutions**: None reported.

5. Acceleration/Deceleration Lanes

a. **Description/Definition of the Element**: These lanes provide drivers with an opportunity to speed up or slow down in a space not used by high-speed TH traffic.(128)

b. **Theoretical/Empirical Effect**s: Acceleration/deceleration lanes reduce the interference of exiting and entering traffic on main lines and consequently decrease the impact of lane changing maneuvers on freeway speeds.(24)
   i. Without sufficient length (> 164.04 ft), acceleration lanes decrease stability of main traffic flow due to shock waves (perturbations) in traffic. Longer acceleration lane lengths correlate directly with main traffic stability.(138)

c. **Existing Solutions**: An existing solution includes the following:
   i. Lengthen/shorten lanes.

d. **New Solutions**: None reported.

(4) GEOMETRIC—INTERSECTIONS/TCD/ITS

1. Intersection Sight Distance

a. **Description/Definition of the Element**: Sight distance is the distance a driver can see at an intersection and can vary according to the driver, obstacles, weather, light, and other conditions.(100)

b. **Theoretical/Empirical Effects**: Intersection sight distance has a strong correlation with vehicle speed.(139)
   i. Decreased sight distance at an intersection leads to less safe conditions for all drivers.

c. **Existing Solutions**: Existing solutions include the following:
    i. Clear sight triangles.[140]
    ii. Realign intersection approaches to reduce or eliminate intersection skew.[140]
    iii. Flat grading.[100]
    iv. Clearly painted traffic lines.[100]
    v. Easily understandable signage.[100]

d. **New Solutions**: A new solution includes the following:
    i. Upgrading to additional, higher intensity, or larger signal heads; backing plates; or modern controllers.

2. LT and RT Lane Overflow

a. **Description/Definition of the Element**: LT and RT lane overflow consists of intersections with turn lanes that do not hold all the cars attempting to turn and queue in traffic lanes that are not turning.

b. **Theoretical/Empirical Effects**:
    i. If the length of a RT lane is inadequate, vehicles waiting to turn may be doing so from the TH traffic lane, thus increasing the potential for rear-end crashes. If long enough, RT lanes provide sheltered locations for drivers decelerating or waiting to make a RT maneuver. Speed is used directly as a measure of mobility, and facilities that emphasize mobility (highways/freeways) accommodate high speed.[140]
    ii. If a RT lane is excessively long, TH drivers may enter the lane by mistake without realizing it is a RT lane. Effective signing and marking of the upstream end of the RT lane may fix this problem.[140]

c. **Existing Solutions**: Existing solutions include the following:[141]
    i. LT lanes are designed with adequate storage length, considering both the red-phase queue and leftover queue.
    ii. Multiple LT lanes should be provided at an intersection where the LT volume exceeds its capacity and an extreme long LT queue exists.
    iii. The length of the LT lane should be extended, or the single LT lane updated to multiple LT lanes, for intersections with LT lane over flow problems to reduce rear-end crash risk.
    iv. Longer bay taper lengths should be provided for intersections in the nonurban areas.
    v. Appropriate signal phasing sequence should be adopted to reduce the delay caused by LT.
    vi. Single lane TH intersection.[142]

d. **New Solutions**: A new solution includes the following:
    i. Utilizing electronic pavement marking on roadways to dynamically include bike lanes and dynamically change the width of the lanes.

3. Parking

a. **Description/Definition of the Element**: Parking for vehicles may be on the street, in lots, garages, private driveways, and other locations. Time spent looking for parking, as well as the price of parking and its resulting availability, are factors in urban congestion.

i. Cars looking for parking cause congestion.
ii. Cars parked near intersections.
iii. Double parked cars.
iv. Truck parking/delivery truck parking.

b. **Theoretical/Empirical Effects**: Improperly priced (or not) parking leads to an inefficient allocation of parking.

c. **Existing Solutions**: Existing solutions include the following:

i. Variable pricing/peak pricing/true cost of parking.(143)
ii. Separating parking costs from rent prices and employee benefits.(143)
iii. Parking taxes, targeted or not.(143)
iv. Time limits.(143)
v. Alternatives—park and rides.(143)
vi. Electronic monitoring in garages and elsewhere (e.g., how many spots are available, also called dynamic parking guidance systems).(143)
vii. Parking cash-out.(143)
viii. Shared parking/park once.(143)
ix. Park-and-ride lots.(144)

d. **New Solutions**: New solutions include the following:

i. Automated parking technology.
ii. Multilevel parking for each parking spot.

4. TCDs (Signal, Stop Sign, etc.)

a. **Description/Definition of the Element**: The purpose of TCDs is to promote highway safety and efficiency by managing the movement of all road users at intersections. Traffic control devices notify road users of regulations and provide warning and guidance needed for the uniform and efficient operation of all elements of the traffic stream in a manner intended to minimize the occurrences of crashes.(79)

b. **Theoretical/Empirical Effects**: Poor signal design and inadequate/inefficient flow interruption cause delay and vehicles queuing.

c. **Existing Solutions**: Existing solutions include the following:

i. Use of real-time or archived data to enhance TCDs.
ii. Properly timed signals.
iii. Effective enforcement.
iv. Actuated signal timing.
v. Coordinated signal timing along the corridor.

d. **New Solutions**: New solutions include the following:

i. Vehicle connectivity.
ii. In-vehicle signal timing display.
iii. Infrastructure-based electronic warning signs.

(5) OPERATIONAL CHALLENGES—AGENCY RELATED

1. Managing Demand

a. **Description/Definition of the Element**: Managing demand is using information to promote more efficient use of road capacity. Transportation demand management (also called mobility management) is a general term for strategies that result in more efficient use of transportation resources.[145]

b. **Theoretical/Empirical Effects**:
   i. Demand management can relieve congestion, even if total volume changes are modest. When a road network is at capacity, adding or subtracting even a single vehicle has disproportionate effects for the network.[146]

c. **Existing Solutions**: Existing solutions include the following:
   i. Use of technology:
      1. Electronic toll passes to charge user-fees, including HOT lanes.
      2. Use of real-time and/or archived operations data to adjust toll rates dynamically.
      3. Ramp metering.
   ii. Use of incentives/disincentives:
      1. Peak-pricing to encourage shoulder period use.
      2. Daily restrictions (i.e., odd licenses one day).
      3. User-fees (vehicle-miles traveled or other).
      4. Encouragement of transit benefits through taxes or other means.
      5. Raising the gas tax or other indirect fee-for-driving.
   iii. Use of alternatives:
      1. Car-sharing.
      2. Park-and-ride lots.
      3. “Guaranteed Ride Home” programs for commuters.
      4. Increased transit options.
      5. Promotion of telework programs.
      6. Express bus service.[147]
      7. Special event traffic management.[148]
   iv. Use of physical barriers:
      1. Traffic-calming devices.
      2. Local traffic only restrictions.
      3. Zone-based pricing bus-only lanes.[149]
      4. Express lanes.

d. **New Solutions**: New solutions include the following:
   i. Smartphone application to orient/manage vehicles routing decision.
   ii. Smartphone application to match drivers with riders for carpools.

2. Intersection Spacing

a. **Description/Definition of the Element**: Intersection spacing is the minimum distance between intersections on the same side of the street. It is measured by the nearest curb returns on the TH street.[150]

b. **Theoretical/Empirical Effects**:
   i. Having more intersections in closer proximity increases the likelihood of accidents.[151]
   ii. Inadequate TCD and small intersection spacing cause interfering of vehicles queues between consecutive intersections.
c. **Existing Solutions**: Existing solutions include the following:
   i. Uniform spacing between intersections.
   ii. Access roads and other intersection avoidance techniques.
d. **New Solutions**: None reported.

3. **Interchange Spacing**

   a. **Description/Definition of the Element**:
      i. An interchange on an interstate refers to the connections between the interstate and other roadway, which can be another interstate or other lesser roadway. It may include bridges, ramps, and other design features.
      ii. Interchange spacing will include the distances between each exit that allows a connection on a limited access road and the spacing within the interchange itself.
   b. **Theoretical/Empirical Effects**: Limiting access speeds traffic and increasing access with more interchanges slows traffic.
      i. Poorly designed interchange spacing can also add confusion to driver behavior.
   c. **Existing Solutions**: Existing solutions include the following:
      i. Uniform interchange spacing as appropriate to specific locations and conditions such as rural areas or urban areas.[151]
      ii. Appropriate signage and facilities to access interchanges.
      iii. Longer auxiliary lanes between on- and off-ramps in interchanges.[127]
   d. **New Solutions**: None reported.

4. **Policy on Entry/Exit Ramp Placement—Left Side Versus Right Side**

   a. **Description/Definition of the Element**: Exit and entry ramps on interstates are means of accessing and exiting the limited access roads and are typically located on the right side.
   b. **Theoretical/Empirical Effects**:
      i. In design, left exits are not ideal because they do not meet driver expectations, create signing issues, and can create confusion for drivers attempting to exit the highway.[152]
      ii. Having exits on the left and right sides on a single road is inconsistent and not a good idea.
      iii. Configurations to achieve right-hand entrances and exits could influence ramp-freeway junction locations and, therefore, the ramp spacing between interchanges.[152]
   c. **Existing Solutions**: Existing solutions include the following:
      i. Consistency in use of right-hand exits.
      ii. Appropriate signage when using left-hand exits.

d. **New Solutions**: None reported.

5. **Posted Speed Limit (Static/Dynamic and by Lane Where Applicable)**

   a. **Description/Definition of the Element**: Posted speed is the maximum lawful vehicle speed for a particular location as displayed on a regulatory sign.[88]
      i. When no speed is posted, a statutory speed is the lawful limit.[88]
   b. **Theoretical/Empirical Effects**:
      i. The posted speed is established based on an isolated restrictive feature (e.g., sharp curve or limited sight distance) within a segment.[88] The congestion occurs when the posted speed does not reflect the horizontal/vertical alignment, which leads to form vehicle queues.
      ii. Reducing incidence of secondary rear-end crashes.
   c. **Existing Solutions**: Existing solutions include the following:
      i. VSLs.[153]
      ii. Traffic calming.
      iii. Speed enforcement.
      iv. Speed limits based on road conditions that differ based on specific segments rather than the whole road.
      v. Always having a posted limits rather than rely on statutory limits.
   d. **New Solutions**: A new solution includes the following:
      i. Advance queue and lane closure warning (i.e., dynamically drop speed limit incrementally miles upstream of slow or stopped traffic in lane).

6. **Signal Timing Administration**

   a. **Description/Definition of the Element**: Signal timing administration includes the coordination of signals for the most efficient use at intersections across a network. It must optimize the intersection for all uses including motor vehicles, pedestrians, and bicyclists as well as transit and freight trains with the goal of increasing flow while decreasing environmental impacts from idle vehicles and dangers.[154]
   b. **Theoretical/Empirical Effects**: Poorly timed signals create undue congestion as resources are not being efficiently utilized.
   c. **Existing Solutions**: Existing solutions include the following:
      i. Update signal timing administration annually to evaluate if it is serving existing traffic patterns.[154]
      ii. Utilization of new technology and control centers.
      iii. Signal operation and management.[155]
   d. **New Solutions**: A new solution includes the following:
      i. Transit signal priority provides an additional travel time advantage, thereby encouraging transit use.

7. **Challenges due to Traffic Composition**

   a. **Description/Definition of the Element**: Traffic composition describes the characteristics of vehicles using a roadway at a given point in time.

    i. Traffic composition usually refers to the percentage of trucks on a roadway but also includes the travel type such as leisure or work-related characteristics.

b. **Theoretical/Empirical Effects**: Larger and heavier vehicles tend to require more road space and are slower to accelerate. As a result, they cause more traffic congestion than smaller lighter vehicles. The relative congestion impact of different vehicles is measured in terms of passenger car equivalents (PCEs). Large trucks and buses tend to have 1.5–4 PCEs depending on roadway conditions and even more for TH intersections or under stop-and-go driving conditions. A large sport utility vehicle imposes 1.4 PCEs, while a van has 1.3 PCEs when traveling through an intersection.(146)

c. **Existing Solutions**: Existing solutions include the following:
    i. Vehicle restrictions.(156)
    ii. User fees.
    iii. Additional weigh stations to enforce restrictions.
    iv. Fewer weigh stations to lessen stopping times.
    v. Peak period restrictions.
    vi. Trucks allowed only in right lanes.(157)
    vii. Weather restrictions.
    viii. Strict enforcement and penalties.

d. **New Solutions**: New solutions include the following:
    i. Use a dynamic tolling for heavy vehicles based on the weight or trip purpose to manage the access of those types of vehicles.
    ii. Freight shuttle.(158)

8. Work Zone Administration

a. **Description/Definition of the Element**: Construction on a roadway causes traffic disturbance, lane closure, speed reduction, and bottlenecks.

b. **Theoretical/Empirical Effects**: Poor management for a work zone leads to unnecessary speed reduction and reduce the level of comfort for drivers.

c. **Existing Solutions**: Existing solutions include the following:
    i. Extra lighting.
    ii. Warning and advance signage.
    iii. Barriers between workers and driving lanes.
    iv. Strict speed and driver behavior enforcement including more severe penalties.
    v. Driver education through signage reminding them of workers present.
    vi. Use of flaggers.(79)
    vii. Use of channelization devices to guide drivers through unfamiliar roadways.(79)
    viii. Lane closure policies (e.g., promotion of nighttime/off-peak work to avoid closures during peak traffic volume periods).
    ix. Queue warning.(159)

d. **New Solutions**: None reported.

9. Complete Roadway Closure Administration

a. **Description/Definition of the Element**: It may be necessary to completely close a roadway for any reason that may include a special event, evacuation, construction, or other event.
    i. Detours direct traffic elsewhere.
    ii. Inferred design speed is the maximum speed for which all critical design speed-related criteria are met at a particular location.
b. **Theoretical/Empirical Effects**: Roadway blockage/closure causes immense delay and reflects on the entire capacity of the network due to the rerouting of vehicles.
c. **Existing Solutions**: Existing solutions include the following:
    i. Restrictions on vehicle types.
    ii. Detour plans in place.(160)
    iii. Appropriate use of contraflow.
    iv. ITS to alert drivers of other routes or changes in roadways.
d. **New Solutions**: None reported.

10. Incident Management and Clearance

a. **Description/Definition of the Element**: In general, many of the traffic congestion sections are caused by some sort of traffic incident, such as a disabled vehicle, a crash, or dangerous driving. Many urban regions have coordinated programs that prevent, identify, and respond to such events quickly and efficiently. These may include centralized traffic management centers, video traffic surveillance, emergency response teams, and even helicopters to move disabled vehicles.(146)
b. **Theoretical/Empirical Effects**: Traffic incidents (disabled vehicles and accidents) account for an estimated 60 percent of delay hours.(146) Although random events, they tend to cause the greatest delays where traffic volumes approach road capacity and so are considered congestion costs. In uncongested conditions, an incident causes little or no traffic delay, but a stalled car on the shoulder of a congested road can produce 100–200 vehicle-h of delay on adjacent lanes.(146)
c. **Existing Solutions**: Existing solutions include the following:
    i. Increased police presence on highways.
    ii. Use of ITS to detour and alert drivers.
    iii. Incident screens to prevent rubbernecking.(161)
    iv. Solid, integrated freeway traffic management system which can be utilized as a prototype and/or applied in real-time traffic operations.(160)
    v. Prepositioned incident response vehicles to clear disabled vehicles in a timely manner.
    vi. Aggressive incident clearance programs and policies.(162)
d. **New Solutions**: None reported.

11. On-Ramp Management (including ramp metering, etc.)

a. **Description/Definition of the Element**: Managing traffic on freeway entrance and exit ramps, or ramp management, is the application of control devices, such as traffic signals, signing, and gates, to regulate the number of vehicles entering or leaving the freeway in order to achieve operational objectives. Most ramp management strategies are employed to balance freeway demand and capacity, maintain optimum freeway operation by reducing incidents that produce traffic delays, improve safety on adjacent freeways or arterial streets, or give to a specific section.(163)
    i. *Ramp metering* is defined as the process of facilitating traffic flow on freeways by regulating the amount of traffic entering the freeway through the use of control devices (such as traffic signals) on entrance ramps. Traffic signals are commonly linked to metering devices, allowing cars to enter the freeway at a predetermined rate.(164)

b. **Theoretical/Empirical Effects**:
    i. Ramp meters control the number of vehicles that can enter a highway ramp. This tends to maintain smoother traffic flow on highways.(146)
    ii. This strategy may not eliminate traffic congestion but can delay its onset and shorten its duration.(165)

c. **Existing Solutions**: Existing solutions include the following:
    i. Consistent use in roadway design.
    ii. Properly timed signals.
    iii. Sufficient merging areas including auxiliary lanes.
    iv. Effective signage, including dynamic signage.
    v. Bus/HOV queue jump.

d. **New Solutions**: A new solution includes the following:
    i. Priced queue jump.

12. Policy for Heavy Vehicles Exclusion on Certain Routes/Lanes

a. **Description/Definition of the Element**: Larger and heavier vehicles tend to require more road space and are slower to accelerate and, as a result, cause more traffic congestion than smaller, lighter vehicles.(146)

b. **Theoretical/Empirical Effects**: Vehicle restrictions that affect customers with time window constraints do not have an impact on customer costs. However, vehicle restrictions are found to be costly when vehicle capacity is limited.(166)
    i. Congestion increases vehicle emissions from heavy vehicles.
    ii. Heavy vehicles damage roads.

c. **Existing Solutions**: Existing solutions include the following:
    i. Restrictions on time and location (peak periods, or on narrow streets).
    ii. Strict penalties and enforcement for restrictions.
    iii. User fees (heavy vehicles pay for greater damage caused to roads due to weight).
    iv. Detector systems for enforcement if weigh stations not in use.
    v. Weigh stations to determination of policy for heavy vehicles.

vi. Detours indicated with signage for heavy vehicles as well as Global Positioning System (GPS) maps with this data included.

d. New Solutions: A new solution includes the following:

i. Dynamic truck restrictions.[167]

13. Managing Lanes

a. Description/Definition of the Element: Managing lanes includes highway facilities or a set of lanes where operational strategies are proactively implemented and managed in response to changing conditions.[168]

b. Theoretical/Empirical Effects:[24]

i. Reduce the frequency of collisions caused when motorists encounter congested conditions, work zones, or incidents.
ii. Improve throughput and/or reduce emissions by achieving more uniform and stable traffic flow as demand approaches capacity. This uses the freeway more efficiently and delays or prevents the onset of congestion.
iii. Improve reliability of travel times for certain classes of travelers.
iv. Distribute total delay in a more equitable manner, preserving some capacity for downstream segments.
v. Increase the efficiency of operation under reduced capacity conditions caused by incidents or maintenance operations.
vi. Divert some freeway traffic to alternative routes or encourage alternative departure times to better use corridor capacity. This will reduce peak-period traffic demand on the freeway.
vii. Provide a travel time incentive to HOVs.
viii. Extend pavement life (by restricting trucks from particular lanes).[24]

c. Existing Solutions: Existing solutions include the following:

i. Maintained managed lanes (e.g., working signals, pricing systems, etc.).
ii. Appropriately priced lanes so that lanes are not overused.
iii. Pay-for-use to efficiently allocate use.
iv. Use of data to evaluate successful strategies.
v. Driver outreach to explain advantages and ways to use managed lanes.
vi. Use of ITS to deliver traveler information, allowing drivers to make informed route, mode, and time of day choices.
vii. HOT/HOV lanes.
viii. Electronic toll lanes.
ix. Congestion/peak pricing.
x. Reversible lanes.
xi. Transit-only lanes.

d. New Solutions: New solutions include the following:

i. Mainline metering.[123]
ii. Green-wave flow techniques.[40]
iii. Priced dynamic shoulder lanes.
iv. In-vehicle technology to allow police to read transponder of vehicles traveling at full speed in adjacent lane. The device reports whether a transponder has had a legal transaction in the past couple of minutes on nonbarrier separated HOT lanes.

14. Responses to Weather

- **a. Description/Definition of the Element**: Environmental conditions can lead to changes in driver behavior that affect traffic flow. Due to reduced visibility, drivers will usually lower their speeds and increase their headways when precipitation, bright sunlight on the horizon, fog, or smoke are present. Wet, snowy, or icy roadway surface conditions will also lead to the same effect even after precipitation has ended.(169)
- **b. Theoretical/Empirical Effects**: Extreme weather can slow traffic and cause accidents with lower sight distances and wet, snowy, or icy conditions. High winds can bring debris into the roadway causing abrupt slowdowns or accidents. Splash and spray from water on the pavement can disorient drivers.(106)
- **c. Existing Solutions**: Existing solutions include the following:
  - i. Education (e.g., campaigns from emergency management or State to alert drivers to changes due to weather).
  - ii. Use of ITS especially if putting restrictions into place.
  - iii. Reflective lighting.
  - iv. Roadside lighting.
  - v. Roadside shelters for pulling over.
  - vi. Laws with corresponding enforcement and signage (i.e., lights on when raining).
  - vii. Contraflow for evacuations and law enforcement management.
  - viii. Salting, putting sand, or other use of material to coat roadway to increase traction.
- **d. New Solutions**: A new solution includes the following:
  - i. Restrictions during weather events (e.g., during warnings).

15. Overheight Management Policy

- **a. Description/Definition of the Element**: These are policies creating guidelines for vehicles over specified heights.
- **b. Theoretical/Empirical Effects**: Vehicles with overheight can cause congestion if they are on a roadway in which clearances are too low. Restrictions can cause overheight vehicles to clog other roadways as well.
- **c. Existing Solutions**: Existing solutions include the following:
  - i. Vehicle restrictions.
  - ii. Strict enforcement and penalties.
  - iii. Vehicle height detectors.
  - iv. Signage and warnings.
  - v. Time restrictions (i.e., with regard to bridge raisings or other dynamic road conditions).
  - vi. Plans for alternate routes that aims to distribute large vehicles evenly and safely and not only on small roads not designed for large vehicle traffic.
- **d. New Solutions**: None reported.

16. Congestion Pricing

a. **Description/Definition of the Element**: Congestion pricing (i.e., value pricing or zone-based pricing) is a way of harnessing the power of the market to reduce the waste associated with traffic congestion. Congestion pricing works by shifting purely discretionary and lower value peak period highway travel to other transportation modes or to off-peak periods, taking advantage of the fact that the majority of rush hour drivers on a typical urban highway are not commuters. By removing a fraction (even as small as 5 percent) of the vehicles from a congested lane or roadway, pricing enables the system to flow much more efficiently, allowing more cars to move through the same physical space.
   i. Variably priced lanes, involving variable tolls on separated lanes within a highway, such as express toll lanes or HOT lanes.
   ii. Variable tolls on entire roadways, both on toll roads and bridges, as well as on existing toll-free facilities during rush hour.
   iii. Cordon charges, either variable or fixed charges, to drive within or into a congested area within a city.(170)
b. **Theoretical/Empirical Effects**: Congestion pricing allows the efficient allocation of capacity by charging those who most demand the space to use it and using the principles of supply and demand. It allows planners to price the benefit of less congestion and the true cost of that trip In some cases, may unfairly burden low-income drivers and may not take into account urban to suburban commuting patterns and other nontraditional but common commuting patterns.
c. **Existing Solutions**: Existing solutions include the following:
   i. Express toll lanes.
   ii. Variable pricing/peak pricing/dynamic pricing.
   iii. HOT/HOV lanes.
   iv. Bus lanes.
   v. GPS systems that allow for distance-based fees.(171)
   vi. “Pay as you drive” insurance programs.(171)
   vii. Use of fees to fund alternatives to driving, such as transit.(171)
d. **New Solutions**: New solutions include the following:
   i. Taxes on driving, parking, or types of vehicles.
   ii. Tax breaks or incentives for transit use.

17. Toll Booth Operations

a. **Description/Definition of the Element:** Toll booths are the facilities in which drivers must pass through when tolls are collected for use of a roadway. They may or may not be staffed or be automated and may have electronic access lanes that accept passes. For facilities where cash tolling is necessary, optimal numbers indicate having twice as many toll booth lanes as traffic lanes.(172)
   i. **Toll booth**: A shelter where a toll attendant is stationed to collect tolls or issue toll tickets. It is located adjacent to a toll lane and is typically set on a toll island.
   ii. **Toll island**: A raised island on which a toll booth or other toll collection and related equipment are located.

iii. **Toll lane**: An individual lane located within a toll plaza in which a toll payment is collected or, for toll ticket systems, a toll ticket is issued.
iv. **Toll plaza**: The location at which tolls are collected. It consists of a grouping of toll booths, toll islands, toll lanes, and, typically, a canopy. Toll plazas might be located on highway mainlines or on interchange ramps. A mainline toll plaza is sometimes referred to as a "barrier toll plaza" because it interrupts the traffic flow.
v. **Toll ticket system**: A system in which the user of a toll road receives a ticket from a machine or toll booth attendant upon entering a toll system. The ticket denotes the user's point of entry. Upon exiting the toll system, the user surrenders the ticket and is charged a toll based on the distance traveled between the points of entry and exit.(79)

b. **Theoretical/Empirical Effects**:
   i. A car without an electronic pass is delayed 8–12 s at an automatic booth and 13–17 s at a manual booth. Speed is used directly as a measure of mobility. Facilities that emphasize mobility (i.e., highways/freeways) accommodate high speeds.(172)
   ii. Cash tolls, by definition, slow traffic, as they must pass through narrow barriers and stop and pay.
c. **Existing Solutions**: Existing solutions include the following:(172)
   i. Increased use of electronic toll passes or automated tolls booths.
   ii. Open road tolling, where all vehicles must either utilize a toll transponder or pay a higher toll receiving a bill in the mail through optical character recognition of the license plate.
   iii. Toll booth reconfigurations where vehicles with transponders travel at up to full posted speed in the through lanes and cash toll payers exit to toll booths on the side.
   iv. Elimination of physical road barriers.
   v. All electronic tolling.(173)
d. **New Solutions**: None reported.

18. Service Patrols Placement

a. **Description/Definition of the Element**: Service patrols vary by State and locality but generally offer free services to motorists such as providing a gallon of gas, water for an overheated radiator, a cell phone to make a call, as well changing a flat tire or jump starting a car battery. These services are provided using different funding sources and with different personnel, some of which are volunteers, while others are uniformed officers.
b. **Theoretical/Empirical Effects**: These may get cars off the road sooner but may cause gaper delays.
c. **Existing Solutions**: Existing solutions include the following:
   i. Larger shoulders.
   ii. Larger deployment of service vehicles.
d. **New Solutions**: A new solution includes the following:
   i. Use of different marked vehicles than law enforcement so as not to create delays associated with seeing law enforcement vehicles.

19. **Law Enforcement Policy/Location**

   a. **Description/Definition of the Element**: Highways and roadways often have police units assigned to them to monitor conditions. Law enforcement may take place with parked police vehicles or electronic speed detection, red light cameras, or other method.
   b. **Theoretical/Empirical Effects**: The presence of law enforcement causes slowdowns to avoid moving violations. Areas known for speed traps may be avoided by drivers.
   c. **Existing Solutions**: Existing solutions include the following:
      i. Electronic enforcement of speed and other violations that is indicated by signage.
      ii. Elimination of speed traps for revenue-generation only.
   d. **New Solutions**: None reported.

20. **Forecasting Traffic Demand**

   a. **Description/Definition of the Element**: Forecasting traffic demand allows transportation practitioners to understand future traffic volumes on their roadways. By analyzing this demand, practitioners can develop and implement improvements (e.g., capacity building or operational strategies) to ensure that the transportation system provides the appropriate LOS to the traveling public.
   b. **Theoretical/Empirical Effects**: Improperly calibrated forecasting tools or poor assumptions may lead to miscalculations on traffic demands and roadway volumes causing practitioners to implement solutions that do not meet the actual need.
   c. **Existing Solutions**: Existing solutions include the following:
      i. Use of ITS to understand regional traffic demand.
      ii. Using archived data to calibrate or validate tools or models.
      iii. Use of a feedback loop to continually update tools or models.
   d. **New Solutions**: None reported.

(6) OPERATIONAL CHALLENGES—DRIVER RELATED

1. **Bunching of Vehicles as a Result of Reduced Speed and Resulting Shockwaves on Abrupt Speed Changes**

   a. **Description/Definition of the Element**: In a study carried out by the Organization for Economic Cooperation and Development, a vehicle is regarded as being bunched when it is travelling less than 5 s behind the preceding vehicle and the difference in speed is less than 6.21 mi/h.[174]
   b. **Theoretical/Empirical Effects**: The bunching of vehicles is directly causing shock waves and flow disturbance. It might be caused by the changing lanes of heavy vehicles and/or sudden brake by a lead vehicle.
   c. **Existing Solutions**: Existing solutions include the following:
      i. Express buses and buses allowed to make express routes.[175]
      ii. Use of real-time data to direct detours and inform drivers.

   iii. Managed lanes.
   iv. Ramp metering.
   v. Dynamic signage and speed limits.
   vi. Flat grades and fewer sharp curves.[169]
d. **New Solutions**: None reported.

2. **Roadside Distractions/Rubbernecking**

a. **Description/Definition of the Element**: Roadside distractions include unusual or atypical events that cause drivers to become distracted from driving.[169]
   i. Rubbernecking is the act of drivers looking at incidents on the opposite side and causing a delay on the opposite side of the roadway. This can cause accidents as a result.
b. **Theoretical/Empirical Effects**: Distractions from unusual incidents and rubbernecking often creates slowdowns but also accidents from distracted drivers.
c. **Existing Solutions**: Existing solutions include the following:
   i. Barriers between traffic flows with higher Jersey barriers or other wall structures.
   ii. Screens to prevent drivers from seeing accidents.[161]
   iii. Barriers on roadsides.
d. **New Solutions**: None reported.

3. **Nonroadside Distractions (i.e., Cellphone Usage, GPS, etc.)**

a. **Description/Definition of the Element**: Distracted driving is anything that takes the driver's attention from the road including but not limited performing activities not related to driving, using technology, GPS, or interacting with passengers.
b. **Theoretical/Empirical Effects**: Distracted driving continues to cause more accidents each year.
c. **Existing Solutions**: Existing solutions include the following:[176]
   i. Strict penalties and enforcement.
   ii. Education campaigns such as Distraction.gov and through partnerships with mobile phone service providers.
   iii. Text stops (like rest stops).
   iv. Laws allowing law enforcement to pull over vehicles for distracted driving as a first offense.
   v. Signage.
   vi. Use of headsets and restrictions on handheld use and texting bans.
d. **New Solutions**: A new solution includes the following:
   i. Automatic disabling technology while driving.

4. **Unsafe Vehicle Conditions for Weather Conditions Present**

a. **Description/Definition of the Element**: Weather conditions may present additional hazards for vehicles in poor condition.

b. **Theoretical/Empirical Effects**: Unprepared/unsafe vehicle conditions can cause sudden stopping actions which will lead to flow disturbance, shockwaves, and a high probability of crashing, especially at poor visibility levels.
c. **Existing Solutions**: An existing solution includes the following:
    i. Vehicle inspections and requirements.
d. **New Solutions**: A new solution includes the following:
    i. Electronic enforcement.

5. **Aggressive Driving**

    a. **Description/Definition of the Element**: Aggressive driving includes improper driver behavior, including the following behaviors performed poorly:
        i. Lane-changing.
        ii. Weaving.
        iii. Merging.
        iv. Tailgating.
    b. **Theoretical/Empirical Effects**: Aggressive driving can lead to shockwaves, speed reduction, and possible crashes.
    c. **Existing Solutions**: Existing solutions include the following:
        i. Strict penalties and enforcement.
        ii. Driver education campaigns, including outreach in communities and through media.(177)
    d. **New Solutions**: None reported.

6. **Driving on Unauthorized Roadway Sections (i.e., Unauthorized Shoulder Usage, Median Opening Usage, etc.)**

    a. **Description/Definition of the Element**: This includes vehicles and persons driving in areas they are not authorized to drive in.
    b. **Theoretical/Empirical Effects**: Driving on unauthorized roadway section definitely causes flow disturbance and law enforcement.
    c. **Existing Solutions**: Existing solutions include the following:
        i. Signage.
        ii. Education.
        iii. Penalties and enforcement.
        iv. Clear alternatives.
        v. Clearly marked detours, restricted areas, etc.
    d. **New Solutions**: None reported.

7. **Suboptimal Driver Performance in Work Zones**

    a. **Description/Definition of the Element**: Motorists alter their driving behavior in work zones compared to when they are traveling in normal freeway conditions.(178)
        i. A recent FHWA study showed that drivers who are typically comfortable driving on the freeway are less comfortable when passing through work zones. The study involved a survey in which most drivers reported that

they "remain in the same lane while in a construction zone and maintain the speed of the vehicle in front of them."[178]

b. **Theoretical/Empirical Effects**: Poor driving in work zones can result in damage to cars, people, construction crews, and their equipment. Drivers in heavy construction zones may slow down. Lane decreases and other alterations may cause confusion or congestion.
c. **Existing Solutions**: Existing solutions include the following:
    i. Traveler information to allow motorists to make more informed decisions (e.g., detour around work zone).[179]
    ii. Variable message signs.[180]
    iii. Proper signage, lighting, warning signs, and education.
    iv. Clear directives to altered roadways.
    v. Strict enforcement and penalties.
d. **New Solutions**: None reported.

8. Suboptimal Driver Performance When Involved in an Incident

a. **Description/Definition of the Element**: Drivers involved in a minor incident may fail to pull over even when they are able to do so (i.e., there is minimal damage to the vehicle and it is safe to do so), leaving a lane blocked.
b. **Theoretical/Empirical Effects**: Blocked lane(s) reduce capacity and definitely reduce traffic flow and level of driving comfort.
c. **Existing Solutions**: Existing solutions include the following:
    i. Signage explaining need to move vehicles from roadway.
    ii. Large shoulder on which to move vehicles.
    iii. Use of ITS for other drivers.
d. **New Solutions**: None reported.

9. Suboptimal Driver Performance on a Roadway with an Incident

a. **Description/Definition of the Element**: One study found that the reaction time of drivers increases in response to a traffic incident, thus resulting in capacity reductions on highway lanes.[181]
b. **Theoretical/Empirical Effects**: The same study noted previously found that capacity per available lane was 30 to 50 percent lower during an incident than under normal conditions due to driver distraction. Similar reductions in flow were found in the opposite direction of the roadway on which an incident occurred.[181] Drivers cannot be distracted by events or objects they cannot see. To mitigate rubbernecking in the opposite direction, this statement calls for barriers that block vision to opposite direction traffic conditions. Certain segments along the freeway system could have guardrails and a grass median dividing the freeway traffic.[182]
c. **Existing Solutions**: Existing solutions include the following:
    i. Faster incident clearance.
    ii. Higher barriers between roadways.
d. **New Solutions**: A new solution includes the following:
    i. GPS data linked to ITS data to provide drivers with choices to take alternate routes.

10. Suboptimal Driver Performance with Regard to Emergency Vehicles

a. **Description/Definition of the Element**:
   i. Drivers may slow down and/or get distracted when they hear emergency vehicle sirens but cannot locate the direction the vehicle is coming from.
   ii. Drivers may slow down below the design speed limit when they are traveling near a law enforcement vehicle, even if the vehicle is not responding to an emergency.

b. **Theoretical/Empirical Effects**: Drivers who do not move over or are slow to move over can slow emergency services.

c. **Existing Solutions**: Existing solutions include the following:
   i. Emergency vehicle preemption uses special control features in traffic signals to provide clear guidance on whether vehicles should stop (providing a red display) or go (providing a green display) at signalized intersections during the approach of emergency vehicles. In these systems, ITS systems attempt to reduce the surprise factor, which may cause drivers to make bad decisions or perform poorly. The benefit of the ITS is the change in the performance of the traffic flow as a result of improved driver behavior.(183)
   ii. Larger shoulders for vehicles to pull over.
   iii. Properly timed signals that coordinate with emergency vehicles.
   iv. Sirens that can be heard consistently rather than when the vehicle is right behind a vehicle.
   v. Enforcement and stricter penalties for noncompliance.

d. **New Solutions**: A new solution includes the following:
   i. Interaction with ITS and personal GPS devices as well as automated vehicle guidance systems to alert drivers ahead of time.

11. Suboptimal Driver Performance with Respect to TCD and ITS Device Operations

a. **Description/Definition of the Element**: This includes driver misuse of information devices or reactions to the devices.

b. **Theoretical/Empirical Effects**: Ignoring, misreading, or misinterpreting TCD and ITS devices can lead to poor driving decisions that may negatively impact other drivers or the roadway.

c. **Existing Solutions**: Existing solutions include the following:
   i. Clear signage.
   ii. Maintenance and correct information displayed.
   iii. Real-time information.
   iv. Strict enforcement and severe penalties.
   v. Red light cameras to enforce traffic signal use.

d. **New Solutions**: A new solution includes the following:
   i. Integration of GPS mapping with ITS information and TCDs (i.e., GPS directions would alert drivers to upcoming signals, delays, and other roadway information).

12. Suboptimal Driver Performance with Respect to Usage of Conventional and Alternative Intersections and Interchanges

a. **Description/Definition of the Element**:
   i. At alternative intersections/interchanges, drivers may slow down because they are unfamiliar with the design of the intersection and uncertain how to proceed through it.
   ii. An FHWA study looked at driver behaviors in several different intersection scenarios: red-light running, LTs at busy intersections, turning onto a major road with moderate traffic, and rear-end crashes.(184)
b. **Theoretical/Empirical Effects**:
   i. Reduced speed.
   ii. In an FHWA study, for LTs at busy intersection scenario, some younger drivers indicated that they would force their way into the oncoming lane, thereby causing other drivers to slow down or stop.(185)
   iii. Improper driver behavior at intersections and interchanges can negatively affect other driver's behavior, cause crashes, confusion, etc.
c. **Existing Solutions**: Existing solutions include the following:
   i. Advance direction signs (e.g., diagrammatic, stack, and modified stack).(186)
   ii. Consistent use of signage and roadway design (i.e., right-hand exits).
   iii. Driver education with adequate or extra signage to indicate proper behavior.
d. **New Solutions**: None reported.

13. Suboptimal Driver Performance with Respect to Pedestrians and Bicyclists

a. **Description/Definition of the Element**: Drivers may violate traffic rules with regard to other nonvehicle users of the road.
b. **Theoretical/Empirical Effects**: Either pedestrians or bicyclists can block the roadway, causing congestion or accidents.
c. **Existing Solutions**: Existing solutions include the following:(79)
   i. Driver education, including campaigns to increase awareness and laws regarding bicyclists and pedestrians (e.g., giving bikers 3 ft).
   ii. Proper signage and roadway marking for yielding, crosswalks, bike lanes, parking, and signals for bikers and walk symbols for pedestrians.
   iii. Effective road design with the pedestrian rather than only the car in mind.
   iv. Consistent enforcement of violations.
   v. Pedestrian refuges and barriers from car traffic for bicyclists, as well.
d. **New Solutions**: A new solution includes the following:
   i. Strict penalties and enforcement for those violating and negatively impacting pedestrians and bicyclists, including criminal proceedings as would take place in injuries relating to vehicle accident. Criminal charges are rarely filed for drivers hitting pedestrians on sidewalks.

14. Suboptimal Driver Performance with Respect to Animal Crossings

a. **Description/Definition of the Element**: Animals in the roadway can cause accidents and slowing of traffic. Additionally, it is often harmful to species, as they can be severely injured or killed.
b. **Theoretical/Empirical Effects**: Animals in the roadway can cause gaper delays, slow-downs, and accidents, as well as loss of life for animals attempting to cross the road.
c. **Existing Solutions**: Existing solutions include the following:
    i. Signage indicating need for extra vigilance.
    ii. Fences to keep wildlife at bay.
    iii. Designated wildlife crossings and bridges that, while expensive, recognize the importance of wildlife in their habitat that has been interrupted by a roadway. These can include bridges, overpasses, culverts, and other physical structures meant to guide animals to safety.(187)
d. **New Solutions**: None reported.

15. Suboptimal Driver Performance with Respect to Commercial and Heavy Vehicle Operation

a. **Description/Definition of the Element**: This constitutes violation of traffic rules and norms by commercial and heavy vehicles operation either by drivers of commercial and heavy vehicles or by drivers interacting with them.
b. **Theoretical/Empirical Effects**: Accidents and congestion can result from improper behavior by drivers of these vehicles or by drivers around them.
c. **Existing Solutions**: Existing solutions include the following:
    i. Effective enforcement.
    ii. Restrictions on roadways and timing (i.e., delivery only at night).
d. **New Solutions**: None reported.

(7) OPERATIONAL CHALLENGES—NONMOTORIST RELATED

1. Suboptimal Pedestrian and Bicyclist Performance

a. **Description/Definition of the Element**: Pedestrians and bicyclists may violate traffic rules.
b. **Theoretical/Empirical Effects**: They can either block the roadway and cause congestion or result in accidents possibly with injuries to the pedestrians and bicyclists.
    i. Sharing the road with cars slows cars down.
c. **Existing Solutions**: Existing solutions include the following:(79)
    i. Working walk symbols with properly timed countdowns in marked crosswalks and signage indicating pedestrians present.
    ii. Designated bike lanes, especially separate from the roadway, but clearly marked, and signage indicating bicyclists sharing the roadway or are present.

iii. Enforcement to penalize and discourage violation of traffic rules and equipment requirements such as helmets, reflectors, and lamps.
iv. Pedestrian refuges in large road crossings.
v. Effective roadway design.
vi. Complete streets where roadways are designed or redesigned with all users in mind rather than strictly vehicles.
vii. Requiring bicyclists to be registered so if they are ticketed they will have a record.[188]
viii. Media campaigns about sharing the road and pedestrian and bicyclist laws and behavior.
ix. Retiming an arterial street to achieve pedestrian access.

d. New Solutions: A new solution includes the following:

i. Separate bike-only lanes between parking and sidewalk or part of sidewalks, as in many European countries.

REFERENCES

1. Schrank, D., Eisele, B., and Lomax, T. (2012). *TTI's 2012 Urban Mobility Report*, Texas A&M Transportation Institute, The Texas A&M University System, College Station, TX.

2. Margiotta, R.A. and Spiller, N.C. (2012). *Recurring Traffic Bottlenecks: A Primer Focus on Low-Cost Operational Improvements*, Report No. FHWA-HOP-12-012, Federal Highway Administration, Washington, DC.

3. Dbindsa, A. and Spiller, N.C. (2010). *Traffic Analysis Toolbox Volume X: Localized Bottleneck Congestion Analysis: Focusing on What Analysis Tools Are Available, Necessary, and Productive for Localized Congestion Remediation*, Report No. FHWA-HOP-09-042, Federal Highway Administration, Washington, DC.

4. Park, B., Jones, T.K., and Griffin, S.O. (2010). *Traffic Analysis Toolbox Volume XI: Weather and Traffic Analysis, Modeling and Simulation*, Report No. FHWA-JPO-11-019, Federal Highway Administration, Washington, DC.

5. Cambridge Systematics, Inc. and Texas Transportation Institute. (2004). *Traffic Congestion and Reliability: Linking Solutions to Problems*, Federal Highway Administration, Washington, DC. Obtained from: http://www.ops.fhwa.dot.gov/congestion_report_04/ congestion_report.pdf. Site last accessed August 4, 2016.

6. Transportation Research Board. (2010). *Highway Capacity Manual*, Fifth Edition, Transportation Research Board, Washington, DC. Obtained from: hcm.trb.org. Site last accessed August 4, 2016.

7. Federal Highway Administration. *Traffic Bottlenecks*, 21st Century Operations Using 21st Century Technologies, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/bn/lbr.htm. Site last accessed August 4, 2016.

8. Margiotta, R. (2011). *An Agency Guide on How to Establish Localized Congestion Mitigation Programs*, Report No. FHWA-HOP-11-009, Federal Highway Administration, Washington, DC.

9. University of Maryland CATT Laboratory. *The Vehicle Probe Project Suite*, University of Maryland, College Park, MD. Obtained from: https://vpp.ritis.org/suite/. Site last accessed August 4, 2016.

10. Bertini, R.L. (2006). *You Are the Traffic Jam: An Examination of Congestion Measures*, Transportation Research Board, Washington, DC.

11. Skabardonis, A., Varaiya, P.P., and Petty, K.F. (2003). "Measuring Recurrent and Non-Recurrent Traffic Congestion," *Transportation Research Record 1856*, Transportation Research Board, Washington, DC.

12. Luo, Y. (2013). *Asymmetric Driver Behaviour-Based Algorithms for Estimating Real-Time Freeway Operational Capacity*, Master's Thesis, Edmonton, Alberta, Canada.

13. Dunn, W.M., and Latoski, S.P. (2003). *Safe and Quick Clearance of Traffic Incidents*, NCHRP Synthesis 318, Transportation Research Board, National Research Council, Washington, DC.

14. Farradyne, P.B. (2003). *Quick Clearance and 'Move-It' Best Practices Final Report*, I-95 Corridor Coalition, College Park, MD. Obtained from: http://i95coalition.org/wp-content/uploads/2015/03/I-95CC_QC-MI_PB_Final_Report.pdf?dd650d. Site last accessed September 21, 2016.

15. Federal Highway Administration. *FHWA 2015 Traffic Incident Management Self-Assessment*, U.S. Department of Transportation, Washington, DC. Obtained from: http://atri.checkboxonline.com/TIM-SA-2015.aspx. Site last accessed September 21, 2016.

16. Federal Highway Administration. *National Traffic and Road Closure Information*, U.S. Department of Transportation, Washington, DC. Obtained from: https://www.fhwa.dot.gov/trafficinfo/. Site last accessed June 22, 2016.

17. Federal Highway Administration. *QuickZone*, 21st Century Operations Using 21st Century Technologies, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.ops.fhwa.dot.gov/wz/traffic_analysis/quickzone/index.htm. Site last accessed September 22, 2016.

18. Federal Highway Administration. *Work Zone Self-Assessment*, 21st Century Operations Using 21st Century Technologies, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/wz/decision_support/self-assess.htm. Site last accessed September 22, 2016.

19. National Center for Atmospheric Research. *Maintenance Decision Support System*, Research Applications Laboratory, Boulder, CO. Obtained from: http://www.rap.ucar.edu/projects/rdwx_mdss/. Site last accessed September 22, 2016.

20. Murphy, R., Swick, R., and Guevara, G. (2012). *Best Practices for Road Weather Management, Version 3.0*, Report No. FHWA-HOP-12-046, Federal Highway Administration, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/publications/fhwahop12046/. Site last accessed September 22, 2016.

21. Federal Highway Administration. *Road Weather Management Overview*, 21st Century Operations Using 21st Century Technology, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.ops.fhwa.dot.gov/weather/overview.htm. Site last accessed October 19, 2016.

22. Latoski, S.P., Dunn Jr., W.M., Wagenblast, B., Randall, J., and Walker, M.D. (2003). *Managing Travel for Planned Special Events*, Report No. FHWA-OP-04-010, Federal Highway Administration, Washington, DC.

23. Smith, B. (2003). *Configuration Management for Transportation Management Systems*, Report No. FHWA-OP-04-013, Federal Highway Administration, Washington, DC.

24. Neudorff, L.G., Randall, J.E., Reiss, R., and Gordon, R. (2003). *Freeway Management and Operations Handbook*, Report No. FHWA-OP-04-003, Federal Highway Administration, Washington, DC.

25. SRF Consulting Group, Inc. (2000). *Adaptive Urban Signal Control and Integration (AUSCI) Evaluation Final Report*, SRF No. 0942089.8, Minnesota Department of Transportation, Roseville, MN. Obtained from: http://vejdirektoratet.dk/DA/viden_og_data/temaer/its/Documents/Evalueringer/Adaptiv%20signalstyring/13343.pdf. Site last accessed September 22, 2016.

26. Federal Highway Administration. *Intermodal Research Integrated Corridor Management*, Intelligent Transportation Systems Joint Program Office, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.its.dot.gov/research_archives/icms/index.htm. Site last accessed September 23, 2016.

27. Federal Highway Administration. *ITS Professional Capacity Building Program*, Intelligent Transportation Systems Joint Program Office, U.S. Department of Transportation, Washington, DC. Obtained from: https://www.pcb.its.dot.gov/. Site last accessed September 23, 2016.

28. Federal Highway Administration. (2005). *Managing Demand Through Traveler Information Services*, Publication No. FHWA-HOP-05-005, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.ops.fhwa.dot.gov/publications/manag_demand_tis/travelinfo.htm. Site last accessed September 23, 2016.

29. Elhenawy, M., Chen, H., and Rakha, H.A. (2015). *Traffic Congestion Identification Considering Weather and Visibility Conditions Using Mixture Linear Regression*, Proceedings of the 94th Transportation Research Board Annual Meeting, Washington, DC.

30. Transportation Research Board. (1994). *Highway Capacity Manual*, Third Edition, TRB Special Report 209, Transportation Research Board, Washington, DC.

31. Research and Innovative Technology Administration. "2012 Urban Mobility Report Released with New Congestion Measures," *UTC Spotlight*, University Transportation Centers Program, U.S. Department of Transportation, Washington, DC. Obtained from: https://www.rita.dot.gov/utc/sites/rita.dot.gov.utc/files/utc_spotlights/pdf/spotlight_0313.pdf. Site last accessed August 4, 2016.

32. Iteris. (2014). *2013 Most Congested Freeways Report and Methodology*, Metropolitan Transportation Commission, San Francisco, CA.

33. Mallat, S. (1989). "A Theory for Multiresolution Signal Decomposition: The Wavelet Representation," *Institute of Electrical and Electronics Engineers Pattern Analysis and Machine Intelligence*, *11*(7), 674–693.

34. List, G.F., Williams, B., Rouphail, N., Hranac, R., Barkley, T., Mai, E., Ciccarelli, A., Rodegerdts, L., Pincus, Kl, Nevers, B., Karr, A.F., Zhou, X., Wojtowicz, J., Schofer, J., and Khattak, A. (2014). *Establishing Monitoring Programs for Travel Time Reliability*, Report No. S2-L02-RR-1, Second Strategic Highway Research Program, Washington, DC.

35. Federal Highway Administration. *Current News*, 21st Century Operations Using 21st Century Technologies, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/. Site last accessed August 4, 2016.

36. Cambridge Systematics, Inc. (2005). *An Initial Assessment of Freight Bottlenecks on Highways*, White Paper, Bethesda, MD.

37. White, K. and Grenzeback, L.R. (2007). "Understanding Freight Bottlenecks," *Public Roads*, *70*(5).

38. Cooner, S.A., Ranft, S.E., and Zimmerman, C. (2011). *An Agency Guide on Overcoming Unique Challenges to Localized Congestion Reduction Projects*, Report No. FHWA-HOP-11-034, Federal Highway Administration, Washington, DC.

39. Skolnik, J., Chami, R., and Walker, M. (2008). *Planned Special Events—Economic Role and Congestion Effect*, Report No. FHWA-HOP-08-022, Federal Highway Administration, Washington, DC.

40. Kerner, B.S. (2013). "The Physics of Green-Wave Breakdown in a City," *Europhysics Letters*, *102*(2), 28010-1–28010-6.

41. "Bottleneck." (2016). *Merriam-Webster*. Obtained from: http://www.merriam-webster.com/dictionary/bottleneck. Site last accessed August 4, 2016.

42. Daganzo, C.F. (1999). "Remarks on Traffic Flow Modeling and its Applications," *Traffic and Mobility*, 105–115, Springer Berlin Heidelberg, Berlin, Germany.

43. Geroliminis, N., Srivastava, A., and Michalopoulos, P. (2011). *Development of the Next Generation Stratified Ramp Metering Algorithm Based on Freeway Density*, ITS Institute, University of Minnesota, Minneapolis, MN.

44. Kittelson Associates. (2012). *Low-Cost Improvements for Recurring Freeway Bottlenecks*, NCHRP Report 3-83, National Cooperative Highway Research Program, Washington, DC.

45. Transportation Research Board. (2016). *Highway Capacity Manual: A Guide for Multimodal Mobility Analysis*, Sixth Edition, Transportation Research Board, Washington, DC.

46. Hughes, W. et al. (2010.) *Alternative Intersections/Interchanges: Informational Report (AIIR)*, Report No. FHWA-HRT-09-060, Federal Highway Administration, Washington, DC.

47. Federal Highway Administration. *Alternative Intersection Design*, U.S. Department of Transportation, Washington, DC. Obtained from: http://safety.fhwa.dot.gov/intersection/alter_design/. Site last accessed August 4, 2016.

48. Federal Highway Administration. *Capacity Analysis for Planning of Junctions (CAP-X) Tool*, U.S. Department of Transportation, Washington, DC. Obtained from: https://www.fhwa.dot.gov/software/research/operations/cap-x/register/Capacity%20Analysis%20for%20Planning%20of%20Junctions%20Description%20capx%20web%20page%20license%20agreement.pdf. Site last accessed August 4, 2016.

49. Zhang, L. and Wu, G. (2012). "Dynamic Lane Grouping at Isolated Intersections: Problem Formulation and Performance Analysis," *Transportation Research Record 2311*, 152–166, Transportation Research Board, Washington, DC.

50. Zhao, J., Ma, W., Zhang, H.M., and Yang, X. (2013). "Two-Step Optimization Model for Dynamic Lane Assignment at Isolated Signalized Intersections," *Transportation Research Record 2355*, 39–48, Transportation Research Board, Washington, DC.

51. Potts, I.B., Harwood, D.W., Hutton, J.M., Fees, C.A., Bauer, K.M., Lucas, L.M., Kinzel, C.S., and Frazier, R.J. (2014). *Identification and Evaluation of the Cost-Effectiveness of Highway Design Features to Reduce Nonrecurrent Congestion*, Report No. S2-L07-RR-1, Second Strategic Highway Research Program, Washington, DC.

52. Intelligent Transportation Systems Joint Program Office. *Knowledge Resources*, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.itscosts.its.dot.gov/its/itsbcllwebpage.nsf/krhomepage. Site last accessed August 4, 2016.

53. Sallman, D., et al. (2012). *Operations Benefit/Cost Analysis Desk Reference*, Report No. FHWA-HOP-12-028, Federal Highway Administration, Washington, DC.

54. Mirshahi, M., Obenberger, J., Fuhs, C., Howard, C., Krammes, R., Kuhn, B., Mayhew, R., Moore, M., Sahebjam, K., Stone, C., and Yung, J. (2007). *Active Traffic Management: The Next Step in Congestion Management*, Report No. FHWA-PL-07-012, Federal Highway Administration, Washington, DC.

55. Schneider Electric and RK&K. (2015). *Bryan Park Interchange Dynamic Lane Merge Assessment*, Contract Number 40903, Task Order 17, Virginia Department of Transportation, Richmond, VA.

56. Helleman, B. (2010). *Managed Motorways in the Netherlands*, Centre for Transport and Navigation, ICTCT, Vienna, Austria.

57. Telvent USA, LLC. (2014). *Greater Richmond Safety and Mobility Study*, Final Report Volume 2: Detailed Report, Virginia Department of Transportation, Richmond, VA.

58. Parsons Brincherhoff and Texas A&M Transportation Institute. (2009). *Screening Criteria for Managed Use Lane Projects*, New York State Department of Transportation, New York, NY.

59. Texas A&M Transportation Institute. (2011). *General Guidelines for Active Traffic Management Deployment*, UTCM Project No. 10-01-54, Texas A&M Transportation Institute, College Station, TX.

60. Alexiadis, V., Colyar, J., Halkias, J., Hranac, R., and McHale, G. (2004). "The Next Generation Simulation Program," *ITE Journal*, 74(8).

61. CMTLabs. *Berkeley Highway Lab*, California Traffic Management Labs. Obtained from: https://www.ctmlabs.its.uci.edu/facilities/bhl. Site last accessed August 4, 2016.

62. AASHTO. (2011). *A Policy on Geometric Design of Highways and Streets*, Sixth Edition, Second Printing, American Association of State Highway and Transportation Officials, Washington, DC.

63. AASHTO. (2004). *A Policy on Geometric Design of Highways and Streets*, Fifth Edition, American Association of State Highway and Transportation Officials, Washington, DC.

64. Texas A&M Transportation Institute. *Strategies*, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies.php. Site last accessed August 4, 2016.

65. Alliance for Community Trees. *What We Do: Safe Communities*, Alliance for Community Trees, Lincoln, NE. Obtained from: http://actrees.org/what-we-do/public-policy/safe-communities/. Site last accessed August 4, 2016.

66. MyArlingtonTX. (2014). *New Acceleration Lane in IH-30 Improves Traffic Flow*, Office of Communication, Arlington, TX. Obtained from: http://www.arlington-tx.gov/news/2014/03/12/new-acceleration-lane-on-ih-30-improves-traffic-flow/. Site last accessed August 4, 2016.

67. ITS International. (2013). "Traffic Monitoring and Hard Shoulder Running," *ITS International*, Kent, United Kingdom. Obtained from: http://www.itsinternational.com/categories/detection-monitoring-machine-vision/features/traffic-monitoring-and-hard-shoulder-running/. Site last accessed August 4, 2016.

68. Texas A&M Transportation Institute. (2001). *Houston's Travel Rate Improvement Program: "Toolbox" of Improvement Strategies Add Capacity*, Texas A&M University, College Station, TX.

69. Kakesako, G.K. (2012). "Re-striping Project Will Add Fourth Lane on Busy Corridor of H-1," *Honolulu Star Advertiser*, Honolulu, HI. Obtained from: http://www.staradvertiser.com/breaking-news/re-striping-project-will-add-fourth-lane-on-busy-corridor-of-h-1/. Site last accessed August 4, 2016.

70. Pool, B. (1985). "Ventura Freeway Panel Keeps Moving Without Tooting Horn," *Los Angeles Times*, Los Angeles, CA.

71. Ma, J., Hu, J., Hale, D., and Bared, J. (in publication). "Dynamic Hard Shoulder Running for Traffic Incident Management," *Transportation Research Record*, Transportation Research Board, Washington, DC.

72. Hadi, M., Sinha, P., and Wang, A. (2007). "Modeling Reductions in Freeway Capacity due to Incidents in Microscopic Simulation Models." *Transportation Research Record 1999*, 62–68, Transportation Research Board, Washington, DC.

73. Transportation Research Board. (2000). *Highway Capacity Manual*, Fourth Edition, Transportation Research Board, Washington, DC.

74. Miller-Hooks, E., TariVerdi, M., and Zhang, X. (2012). *Standardizing and Simplifying Safety Service Patrol Benefit-Cost Ratio Estimation*, I-95 Corridor Coalition, College Park, MD.

75. Federal Highway Administration. *Travel Time Displays Prior to Freeway Entrances*, U.S. Department of Transportation, Washington, DC. Obtained from: http://tmcpfs.ops.fhwa.dot.gov/projects/ttdfrwyentrance.htm. Site last accessed August 4, 2016.

76. Krause, C., Kronpraset, N., Bared, J., and Zhang, W. (2014). "Operational Advantages of Dynamic Reversible Left-Lane Control of Existing Signalized Diamond Interchanges," *Journal of Transportation Engineering*, *141*(5), 1–8.

77. Zhao, J., Liu, Y., Ma, W., and Yang, X. (2013). *Operation of Signalized Diamond Interchange with Frontage Roads Using Dynamic Reversible Lane Control*, 2014 Transportation Research Board Annual Meeting, Washington, DC.

78. Google Maps®. (2016). *Selected signalized diamond interchange case study*. Generated via Google Maps® online by Janell Niederriter. Obtained from: https://www.google.com/maps/place/Coca+Cola+Dr,+Hanover,+MD+21076/@39.177627,-76.7395852,19z/data=!4m5!3m4!1s0x89b7e180b85a305b:0xa74f9c7875ef1443!8m2!3d39.183775!4d-76.7345538!5m1!1e1. Generated August 5, 2016.

79. Federal Highway Administration. (2009). *Manual on Uniform Traffic Control Devices*, U.S. Department of Transportation, Washington, DC. Obtained from http://mutcd.fhwa.dot.gov/index.htm. Site last accessed August 4, 2016.

80. Institute of Transportation Engineers. (2003). *Making Intersections Safer: A Toolbox of Engineering Countermeasures to Reduce Red-Light Running*, Federal Highway Administration, Washington, DC. Obtained from: http://safety.fhwa.dot.gov/intersection/conventional/signalized/rlr/rlr_toolbox/rlrbook.pdf. Site last accessed September 9, 2016.

81. Google Maps®. (2016). Example of no re-entry signage. Generated via Google Maps® online by Chris Melson. Obtained from: https://www.google.com/maps/@39.7999617,-75.4842429,3a,75y,63.07h,88.71t/data=!3m6!1e1!3m4!1s2p7ahIbaB4X5Qg5Rr0JgdQ!2e0!7i13312!8i6656?hl=en. Generated August 8, 2016.

82. Su, P., Krause, C., Hale, D., and Bared, J. (2015). *Operational Advantages of Contraflow Left-Turn Lane of Signalized Intersection*, 95th Transportation Research Board Annual Meeting, Washington, DC.

83. Yanzhao Metropolis. *Handan Police and "Route Left" Intersections*, Handan, China. Obtained from: http://news.sina.com.cn/c/p/2014-05-16/165130151057.shtml. Site last accessed August 24, 2016.

84. Wenxiu, X. *Sun Gang and Huanggan Crossing Route*, Shenzhen, China. Obtained from: http://city.shenchuang.com/city/20140814/100397.shtml. Site last accessed August 24, 2016.

85. Shenzhen Public Security Bureau. *Turn Left Route Through the Period of Use and Control Consultation*, Shenzhen, China. Obtained from: http://www.stc.gov.cn/GZCY/WSDC/201410/t20141021_31998.htm. Site last accessed August 24, 2016.

86. "Hohhot's First "Route Left" Intersection Trial Today," *Hohhot Evening News*, Hohhot, China. Obtained from: http://www.nmg.xinhuanet.com/xwzx/2015-09/01/c_1116436640.htm. Site last accessed August 24, 2016.

87. Tingeing, D. *The Contraflow Left-Turn Lane, Citizens Are Happy with It*, Henan Province, China. Obtained from http://epaper.jzrb.com/jzwb/page/82/2015-12/05/03/2015120503_pdf.pdf. Site last accessed August 24, 2016.

88. Donnell, E.T., Hines, S.C., Mahoney, K.M., Porter, R.J., and McGee, H. (2009). *Speed Concepts: Informational Guide*, Report No. FHWA-SA-10-001, Federal Highway Administration, Washington, DC.

89. Godley, S.T., Triggs, T.J., and Fildes, B.N. (2004). "Perceptual Lane Width, Wide Perceptual Road Centre Markings and Driving Speeds," *Ergonomics*, *47*(3), 237–256.

90. Richards, S.H., Wunderlich, R.C., and Dudek, C.L. (1985). "Field Evaluation of Work Zone Speed Control Techniques," T*ransportation Research Record 1035*, 66–78, Transportation Research Board, Washington, DC.

91. McLean, J.R. and Hoffman, E.R. (1972). *The Effects of Lane Width on Driver Steering Control and Performance*, Sixth Australian Road Research Board Conference, Canberra, Australia.

92. Martens, M., et al. (1997). *The Effects of Road Design on Speed Behaviour: A Literature Review*, Transportation Research Board, Washington, DC.

93. Chandra, S. and Kumar, U. (2003). "Effect of Lane Width on Capacity Under Mixed Traffic Conditions in India," *Journal of Transportation Engineering*, *129*(2), 155–160.

94. Federal Highway Administration. (1990). *Technical Advisory T 5040.29: Paved Shoulders*, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.fhwa.dot.gov/pavement/t504029.cfm. Site last accessed August 4, 2016.

95. Garber, N.J. and Ehrhart, A.A. (2007). “Effect of Speed, Flow, and Geometric Characteristics on Crash Frequency for Two-Lane Highways,” *Transportation Research Record 1717*, 76–83, Transportation Research Board, Washington, DC.

96. Bertini, R.L. and Leal, M.T. (2005). “Empirical Study of Traffic Features at a Freeway Lane Drop,” *Journal of Transportation Engineering*, *131*(6), 397–407.

97. Federal Highway Administration. *Clear Zone and Horizontal Clearance*, Office of Program Administration, U.S. Department of Transportation, Washington, DC. Obtained from: http://www.fhwa.dot.gov/programadmin/clearzone.cfm. Site last accessed August 4, 2016.

98. Churchill, A., Tripodis, Y., and Lovell, D. (2012). ”Sun Glare Impacts on Freeway Congestion: Geometric Model and Empirical Analysis,” *Journal of Transportation Engineering*, *138*(10), 1,196–1,204.

99. Stutts, J.C., et al. (2001). *The Role of Driver Distraction in Traffic Crashes*, AAA Foundation for Traffic Safety, Washington, DC. Obtained from: https://www.forces-nl.org/download/distraction.pdf. Site last accessed August 4, 2016.

100. Marek, M.A. (2010). *Roadway Design Manual*, Texas Department of Transportation, Austin, TX. Obtained from: http://library.ctr.utexas.edu/hostedpdfs/txdot/design/rdw_2010-11.pdf. Site last accessed August 4, 2016.

101. Fitzpatrick, K., et al. (2000). *Design Factors that Affect Driver Speed on Suburban Arterials*, Research Report 1769-3, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/1769-S.pdf. Site last accessed August 4, 2016.

102. Winsum, W.V. and H. Godthelp. (1996). “Speed Choice and Steering Behavior in Curve Driving,” *Human Factors: The Journal of the Human Factors and Ergonomics Society*, *38*(3), 434–441.

103. McGee, H.W. and Hanscom, F.R. (2006). *Low-Cost Treatments for Horizontal Curve Safety*, Report No. FHWA-SA-07-002, Federal Highway Administration, Washington, DC.

104. Smadi, O., Hawkins, N., Hallmark, S., and Knickerbocker, S. (2013). *Evaluation of the TAPCO Sequential Dynamic Curve Warning System*, Report No. FHWA-HIF-13-040, Federal Highway Administration, Washington, DC.

105. Stein, W.J. and Neuman, T.R. (2007). *Mitigation Strategies for Design Exceptions*, Report No. FHWA-SA-07-011, Federal Highway Administration, Washington, DC.

106. Hall, J.W., Smith, K.L., Titus-Glover, L., Wambold, J.C., Yager, T.J., and Rado, Z. (2009). *NCHRP Web-Only Document 108: Guide for Pavement Friction*, National Cooperative Highway Research Program, Washington, DC.

107. Indiana Department of Transportation. (2013). *Indiana Department of Transportation—2013 Design Manual*. Obtained from http://www.in.gov/indot/design_manual/design_manual_2013.htm. Site last accessed August 4, 2016.

108. Fitzpatrick, K., et al. (2003). *Design Speed, Operating Speed, and Posted Speed Practices*, NCHRP Report 504, National Cooperative Highway Research Program, Washington, DC. Obtained from: http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_504.pdf. Site last accessed August 4, 2016.

109. Greene County. (1978). *Article IV (Special Provisions), Section 36: Access Management*, Greene County, MO. Obtained from: http://www.greenecountymo.org/file/resource_management/Access_Management_Regulation.pdf. Site last accessed August 4, 2016.

110. Project for Public Spaces. *Traffic Calming 101: Project for Public Spaces*, New York, NY. Obtained from: http://www.pps.org/reference/livememtraffic/. Site last accessed August 4, 2016.

111. Gattis, J.L., Duncan, L.K., and Tooley, M.S. (2010). *Roadway Median Treatments—Phase 1 and Phase 2*, Report No. MBTC DOT 2055, U.S. Department of Transportation, Washington, DC. Obtained from: http://ntl.bts.gov/lib/32000/32500/32534/MBTC_DOT_2055-2067.pdf. Site last accessed August 4, 2016.

112. Miller, T.R. (1992). "Benefit-Cost Analysis of Lane Marking," *Transportation Research Record 1334*, 38–45, Transportation Research Board, Washington, DC.

113. Jilla, R.J. (1974). *Effects of Bicycle Lanes on Traffic Flow*, Report No. FHWA/IN/JHRP-74/10, Joint Highway Research Project, Indiana Department of Transportation and Purdue University, West Lafayette, IN.

114. Taylor, D. and Davis, W.J. (1999). "Review of Basic Research in Bicycle Traffic Science, Traffic Operations, and Facility Design," *Transportation Research Record 1674*, 102–110, Transportation Research Board, Washington, DC. Obtained from: http://www.industrializedcyclist.com/Review_basic_research.pdf. Site last accessed August 4, 2016.

115. Federal Highway Administration. (1999). *Implementing Bicycle Improvements at the Local Level*, Report No. FHWA-98-105, U.S. Department of Transportation, Washington, DC. Obtained from: http://ntl.bts.gov/lib/21000/21300/21384/localbike.pdf. Site last accessed August 4, 2016.

116. Federal Highway Administration. (1994). *Bicycle Facilities Recommendations*, Report No. FHWA-RD-92-073, U.S. Department of Transportation, Washington, DC. Obtained from: http://safety.fhwa.dot.gov/ped_bike/docs/select.pdf. Site last accessed August 4, 2016.

117. Huber, M.J. (1957). "Effect of Temporary Bridge on Parkway Performance," *Highway Research Board Bulletin*, *167*, 63–74. Obtained from: http://trid.trb.org/view.aspx?id=116682. Site last accessed August 4, 2016.

118. Abbas, A.R. (2009). *The Ohio Department of Transportation Office of Research & Development Executive Summary Report: Long Term Striping Alternatives for Bridge Decks*, Report No. FHWA/OH-2008/#13, Ohio Department of Transportation, Garfield Heights, OH.

119. Columbia River Crossing Coalition. *Welcome to the Columbia River Crossing Coalition Website*. Obtained from: www.crossingcoalition.org. Site last accessed October 20, 2016.

120. Texas A&M Transportation Institute. *Reversible Traffic Lanes,* Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Reversible-Traffic-Lanes-4-Pg.pdf. Site last accessed August 4, 2016.

121. Edie, L.C. and Foote, R.S. (1958). "Traffic Flow in Tunnels," *Proceedings of the 37th Annual Meeting of the Highway Research Board*, *37*, 334–344, Washington, DC. Obtained from: http://tft.ceng.calpoly.edu/greenshields/docs/edie_foote_1958.pdf. Site last accessed August 4, 2016.

122. Hung, C.J., Monsees, J., Munfah, N., and Wisniewski, J. (2009). *Technical Manual for Design and Construction of Road Tunnels—Civil Elements*, Report No. FHWA-NHI-10-034, Federal Highway Administration, Washington, DC. Obtained from: http://www.fhwa.dot.gov/bridge/tunnel/pubs/nhi09010/tunnel_manual.pdf. Site last accessed August 4, 2016.

123. Nowlin, L. and Brehmer, C. (1997). *Freeway Management Handbook*, Report No. FHWA-SA-97-064, Federal Highway Administration, Washington, DC. Obtained from: http://www.library.unt.edu/gpo/OTA/pubs/fmh/mastoccd.pdf. Site last accessed August 4, 2016.

124. Texas Department of Transportation. *Collector-Distributor Roads Fact Sheet*, Texas Department of Transportation, Austin, TX. Obtained from: https://ftp.dot.state.tx.us/pub/txdot-info/aus/collector-distributor-fact-sheet.pdf. Site last accessed August 4, 2016.

125. Texas A&M Transportation Institute. *Dynamic Merge Control,* Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/active-traffic/technical-summary/Dynamic-Merge-Control-4-Pg.pdf. Site last accessed August 4, 2016.

126. AASHTO. (2001). *A Policy on Geometric Design of Highways and Streets*, 4th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

127. Ringert, J., Koonce, P., Bansen, J., Nguyen, T., McGill, J., Stewart, D., Suggett, J., Neuman, T., Antonucci, N., Hardy, K., and Courage, K. (2004). *Signalized Intersections: Informational Guide*, Report No. FHWA-HRT-04-091, Federal Highway Administration, Washington, DC.

128. Texas A&M Transportation Institute. *Acceleration /Deceleration Lanes*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/system-modification/technical-summary/Acceleration-Deceleration-Lanes-4-Pg.pdf. Site last accessed August 4, 2016.

129. Golob, T.F., Recker, W.W., and Alvarez, V.M. (2004). "Safety Aspects of Freeway Weaving Sections," *Transportation Research Part A*, *38*, 35–51, Transportation Research Board, Washington, DC. Obtained from: http://www.its.uci.edu/its/personnel/recker/A71.pdf. Site last accessed August 4, 2016.

130. Kwon, E. (1999). *Estimation of the Capacity in Freeway Weaving Areas for Traffic Management and Operations*, Report No. Mn/DOT 1999-40, University of Minnesota, Minneapolis, MN.

131. Steward, J., Baker, M., and Van Aerde, M. (1996). "Evaluating Weaving Section Designs Using INTEGRATION," *Transportation Research Record 1555*, 33–41, Transportation Research Board, Washington, DC.

132. Fitzpatrick, K. and Nowlin, L. (1996). "One-Sided Weaving Operations on One-Way Frontage Roads," *Transportation Research Record 1555*, 42–49, Transportation Research Board, Washington, DC.

133. Reilly, W., Kell, J., and Johnson, J. (1984). *Weaving Analysis Procedures for the New Highway Capacity Manual*, JHK & Associates, Tucson, AZ.

134. Bertini, R.L. and Malik, S. (2004). "Observed Dynamic Traffic Features on Freeway Section with Merges and Diverges," *Transportation Research Record 1867*, 25–35, Transportation Research Board, Washington, DC.

135. Cassidy, M.J. and Rudjanakanoknad, J. (2005). *Increasing Capacity of an Isolated Merge by Metering its On-Ramp*, Department of Civil and Environmental Engineering and the Institute of Transportation Studies, University of California–Berkeley, Berkeley, CA.

136. Chang, G.L. and Li, Z. (2010). *Integration of Off-Ramp and Arterial Signal Controls to Minimize the Recurrent Congestion on the I-495 Capital Beltway*, Report No. MD-10-SP808B4D and UMD-2008-03, Maryland State Highway Administration, Baltimore, MD.

137. Chen, C., et al. (1974). "Entrance Ramp Control for Travel Rate Maximization in Expressways," *Transportation Research*, *8*(6), 503–508.

138. Ngoduy, D. (2008). "Operational Effects of Acceleration Lane on Main Traffic Flow at Discontinuities," *Transportmetrica*, *4*(3), 195–207.

139. Liu, M., Guangquan, L., and Li, Y. (2010). "The Impact Analysis of Intersection Sight Distance on Vehicle Speed," *Proceedings of the Seventh International Conference on Traffic and Transportation Studies*, 295–304, Kunming, China.

140. Federal Highway Administration. (2008). *Intersection Safety Strategies*, Publication No. FHWA-SA-08-008, U.S. Department of Transportation, Washington, DC. Obtained from: http://ntl.bts.gov/lib/30000/30800/30899/intersection_guide12.pdf. Site last accessed August 4, 2016.

141. Yu, L. (2007). *Project Summary: 0-5290: Left-Turn Design and Operation*, Texas Department of Transportation, Austin, TX. Obtained from: http://library.ctr.utexas.edu/hostedpdfs/txdot/psr/5290.pdf. Site last accessed August 4, 2016.

142. Kikuchi, S., Kronprasert, N., and Kii, M. (2007). "Lengths of Turn Lanes on Intersection Approaches: Three-Branch Fork Lanes—Left-Turn, Through, and Right-Turn Lanes," *Transportation Research Record 2023/2008*, Transportation Research Board, Washington, DC.

143. Seattle Department of Transportation. (2008). *Briefing Book: Seattle Urban Mobility Plan*, Seattle Department of Transportation, Seattle, WA. Obtained from: http://www.seattle.gov/transportation/briefingbook.htm. Site last accessed August 4, 2016.

144. Texas A&M Transportation Institute. *Park-and-Ride Lots*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/system-modification/technical-summary/Park-And-Ride-Lots-4-Pg.pdf. Site last accessed August 4, 2016.

145. Victoria Transport Policy Institute. (2013). *Online TDM Encyclopedia*, Victoria Transport Policy Institute, Victoria, British Columbia, Canada. Obtained from: http://vtpi.org/tdm/. Site last accessed August 4, 2016.

146. Victoria Transport Policy Institute. (2013). "Congestion Reduction Strategies: Identifying and Evaluating Strategies to Reduce Traffic Congestion," *Online TDM Encyclopedia*, Victoria Transport Policy Institute, Victoria, British Columbia, Canada. Obtained from: http://www.vtpi.org/tdm/tdm96.htm. Site last accessed August 4, 2016.

147. Texas A&M Transportation Institute. *Express Bus Service*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/system-modification/technical-summary/Express-Bus-Service-4-Pg.pdf. Site last accessed August 4, 2016.

148. Texas A&M Transportation Institute. *Special Event Management*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Special-Event-Management-4-Pg.pdf. Site last accessed August 4, 2016.

149. Samdahl, D., Swisher, M., Symoun, J., and Lisska, W. (2013). *Congestion Pricing: A Primer on Institutional Issues*, Report No. FHWA-HOP-13-034, Federal Highway Administration, Washington, DC.

150. City of Sacramento. (2009). *Design and Procedures Manual Section 15: Street Design Standards*, Sacramento, CA. Obtained from: http://www.cityofsacramento.org/Public-Works/Resources/Publications. Site last accessed August 4, 2016.

151. Iowa State University. *Access Management Frequently Asked Questions: 4—Intersection Spacing and Traffic Signal Spacing*, Center for Transportation Research and Education,

Iowa State University, Ames, IA. Obtained from: http://www.ctre.iastate.edu/research/access/toolkit/4.pdf. Site last accessed August 4, 2016.

152. Ray, B.L., Schoen J., Jenior, P., Knudsen, J., Porter, R.J., Leisch, J.P., Mason, J., and Roess, R. (2011). *NCHRP Report 697: Guidelines for Ramp and Interchange Spacing*, National Cooperative Highway Research Program, Washington, DC.

153. Texas A&M Transportation Institute. *Variable Speed Limits*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/active-traffic/technical-summary/Variable-Speed-Limit-4-Pg.pdf. Site last accessed August 4, 2016.

154. Institute of Transportation Engineers. *Traffic Signal Timing*, Institute of Transportation Engineers, Washington, DC. Obtained from: http://www.ite.org/signal/index.asp. Site last accessed August 4, 2016.

155. Texas A&M Transportation Institute. *Signal Operation & Management*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Signal-Operations-and-Management-4-Pg.pdf. Site last accessed August 4, 2016.

156. Texas A&M Transportation Institute. *Truck Incentives & Use Restrictions*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/truck-incentives-and-use-restrictions-4-pg.pdf. Site last accessed August 4, 2016.

157. Texas A&M Transportation Institute. *Truck Lane Restrictions*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/executive-summary/truck-lane-restrictions-1-pg.pdf. Site last accessed August 4, 2016.

158. Texas A&M Transportation Institute. *Freight Shuttle System*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/added-capacity/technical-summary/freight-shuttle-system-4-pg.pdf. Site last accessed August 4, 2016.

159. Texas A&M Transportation Institute. *Queue Warning*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/active-traffic/technical-summary/Queue-Warning-4-Pg.pdf. Site last accessed August 4, 2016.

160. Chang, G. and Kim, W. (2012). *Enhancement of Freeway Incident Traffic Management and Resulting Benefits*, Report No. MD-11-SP009B4Q, Maryland State Highway Administration, Baltimore, MD.

161. Highways Agency. *Incident Screen Trial Update*, England, United Kingdom. Obtained from: http://webarchive.nationalarchives.gov.uk/20120810121037/http://www.highways.gov.uk/business/19085.aspx. Site last accessed August 4, 2016.

162. Texas A&M Transportation Institute. *Aggressive Incident Clearance*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Aggressive-Incident-Clearance-4-Pg.pdf. Site last accessed August 4, 2016.

163. Federal Highway Administration. (2013). *Ramp Management and Metering*, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/freewaymgmt/ramp_mgmnt.htm. Site last accessed August 4, 2016.

164. University of Minnesota. *Ramp Metering*, ITS Institute, University of Minnesota, Minneapolis, MN. Obtained from: http://www.its.umn.edu/Education/k12outreach/modules/RampMetering/. Site last accessed August 4, 2016.

165. Texas A&M Transportation Institute. *Ramp Flow Control*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/active-traffic/technical-summary/Ramp-Flow-Control-4-Pg.pdf. Site last accessed August 4, 2016.

166. Figliozzi, M.A. (2010). *The Impacts of Congestion on Time-definitive Urban Freight Distribution Networks CO2 Emission Levels: Results from a Case Study in Portland, Oregon*, Portland State University, Portland, OR.

167. Texas A&M Transportation Institute. *Dynamic Truck Restrictions*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/active-traffic/executive-summary/dynamic-truck-restrictions-1-pg.pdf. Site last accessed August 4, 2016.

168. Federal Highway Administration. (2013). *Managed Lanes*, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/freewaymgmt/managed_lanes.htm. Site last accessed August 4, 2016.

169. Cambridge Systematics, Inc. and Texas Transportation Institute. (2005). *Traffic Congestion and Reliability: Trends and Advanced Strategies for Congestion Management*, Federal Highway Administration, Washington, DC.

170. Federal Highway Administration. (2006). *Congestion Pricing: A Primer*, U.S. Department of Transportation, Washington, DC. Obtained from: http://ops.fhwa.dot.gov/publications/congestionpricing/sec2.htm. Site last accessed August 4, 2016.

171. Victoria Transport Policy Institute. (2013). "Road Pricing: Congestion Pricing, Value Pricing, Toll Roads and HOT Lanes," *Online TDM Encyclopedia*, Victoria Transport Policy Institute, Victoria, British Columbia, Canada. Obtained from: http://www.vtpi.org/tdm/tdm35.htm. Site last accessed August 4, 2016.

172. Spann, A., Kane, D., and Gulotta, D. (2005). "Lane Changes and Close Following: Troublesome Tollbooth Traffic," *The UMAP Journal*, *26*(3), 251–264.

173. Texas A&M Transportation Institute. *Electronic Toll Collection Systems*, Texas A&M Transportation Institute, College Station, TX. Obtained from: http://mobility.tamu.edu/mip/strategies-pdfs/traffic-management/technical-summary/Electronic-Toll-Systems-4-Pg.pdf. Site last accessed August 4, 2016.

174. Lenz, K.H. and Steinhoff, H. (1976). "An Introduction to the Definition of Vehicle Bunching," *Strassenverkehrstechnik*, *20*(1), 7–14.

175. Daganzo, C.F. and Pilachowski, J. (2009). *Reducing Bunching with Bus-to-Bus Cooperation*, UC Berkeley Center for Future Urban Transport, University of California–Berkeley, Berkeley, CA.

176. Distraction.gov. (2013). *Facts and Statistics*, National Highway Traffic Safety Administration, Washington, DC. Obtained from http://www.distraction.gov/stats-research-laws/facts-and-statistics.html. Site last accessed August 4, 2016.

177. National Highway Traffic Safety Administration. (2015). *Aggressive Driving Enforcement: Strategies for Implementing Best Practices*, U.S. Department of Transportation, Washington, DC.

178. Al-Deek, H., Lochrane, T., Paracha, J., and Scriba, T. (2013). "Understanding Driver Behavior in Work Zones," *Public Roads*, *76*(5).

179. The Second Strategic Highway Research Program. (2013). *Effectiveness of Different Approaches to Disseminating Traveler Information on Travel Time Reliability*, SHRP2 Reliability Project L14, Federal Highway Administration, Washington, DC.

180. Leu, M. and Bham, G. (2012). *Influencing Work Zone Traffic Flow Through Variable Messaging Technologies*, Transportation Research Board, Washington, DC.

181. Knoop, V.L. (2009). *Road Incidents and Network Dynamics: Effects on Driving Behaviour and Traffic Congestion*, Transportation Research Board, Washington, DC.

182. Masinick, J.P. and Teng, H. (2004). *An Analysis on the Impact of Rubbernecking on Urban Freeway Traffic*, National ITS Implementation Research Center, U.S. Department of Transportation, Washington, DC.

183. Collura, J., Louisell, C., Teodorovic, D., and Tignor, S. (2004). *Evaluating Driver Response to Unexpected Conditions: A Case Study in Emergency Vehicle Operations*, Transportation Research Board, Washington, DC.

184. Richard, C.M., Michaels, E.F., and Campbell, J.L. (2006). *Driver Attitudes and Behaviors at Intersections and Potential Effectiveness of Engineering Countermeasures*, Report No. FHWA-HRT-05-158, Federal Highway Administration, Washington, DC.

185. Richards, C.M., Michaels, E.F., and Campbell, J.L. (2005). *Driver Attitudes and Behaviors at Intersections and Potential Effectiveness of Engineering Countermeasures*, Report No. FHWA-HRT-05-078, Federal Highway Administration, Washington, DC.

186. Meerman, A., Hoffman, E.R., and MacDonald, W.A. (1982). *A Comparison of Advance Direction Signs for Use at Irregular Intersections in Terms of Drivers' Route Negotiation Behavior*, Transportation Research Board, Washington, DC.

187. Wildlife and Roads. *Wildlife and Roads: Crossings*. Obtained from: http://www.wildlifeandroads.org/crossings/. Site last accessed August 4, 2016.

188. DeMarco, P. (2011). "Biking Violations Unpunished," *Boston Globe*, Boston, MA. Obtained from: http://www.boston.com/news/local/massachusetts/articles/2011/08/08/loophole_in_law_a_hurdle_to_ticketing_bikers/?page=1. Site last accessed August 4, 2016.

Recycled
Recyclable