Managed Freeway Before Study Appendices

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RESEARCH & DEVELOPMENT

Appendix A - Detailed Literature Review of Managed Freeway Strategies

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1. Introduction

A managed freeway is a relatively new concept encompassing operational strategies aimed at actively managing transportation demand as well as the efficiency of the transportation network. Technically, a managed freeway is a sub-category of Active Traffic Management (ATM). The Federal Highway Administration (FHWA) defines ATM as "*the ability to dynamically manage recurrent and non-recurrent congestion based on prevailing and predicted traffic conditions*" (Sheehan et al., 2012). Focusing on trip reliability, a managed freeway uses a combination of ATM strategies and intelligent transportation system (ITS) interventions to actively manage traffic flows, improve roadway efficiency and safety, control demand, and provide real-time information to road users.

Managed freeways refer to roadways that incorporate technologies enabling transportation managers to observe, communicate, and control the infrastructure and traffic in and alongside the freeway. These technological and operational interventions are deployed to actively manage traffic flows, improve road capacity and safety, and provide travel information to roadway users. Some of the technological interventions incorporated in a managed freeway include coordinated adaptive ramp metering (CARM), speed and lane use management (dynamic lane assignment, variable speed limit), hard shoulder running, real-time traveler information using variable message signs (such as queue warnings), and network intelligence (such as automated vehicle and incident detection). This integration of technology allows transportation authorities to respond dynamically to fluctuations in demand, ensuring optimal utilization of road infrastructure and eventually promoting smoother traffic flow and reductions in travel times.

Managed freeways were first tried in the Netherlands and have been in operation in the United Kingdom since 2006 (van Vuren et al., 2012). Soon after the first implementations of the managed freeway technologies, several studies were carried out outlining the guidelines for designing, operating, and assessing those treatments. Public agencies and consultancy firms published several reports describing their efforts to implement this concept on their freeways, and realized numerous benefits with managed freeways (van Vuren et al., 2012; Dutta et al., 2019). General safety and operational benefits include:

- decrease in primary incidents by alerting drivers to congested conditions and promoting more uniform speeds;
- decrease in secondary incidents by alerting drivers to the presence of downstream queues or incidents and proactively managing traffic in and around incidents;
- increased throughput by reducing the delay associated with the number of primary and secondary incidents by reducing speed differentials in traffic flow, and reducing the shockwave effects of excessive breaking;
- increased overall capacity by allowing shoulder use during congested periods when it is needed most;
- overall improvement in speed uniformity during congested periods; and
- increased trip reliability by increasing capacity and throughput and reducing incident delay and improving vehicle throughput.

The North Carolina Department of Transportation (NCDOT) is spearheading the implementation of managed freeways in the state, starting with two pilot projects, one of which is the I-40 active traffic management (ATM) system in the Raleigh-Durham Metropolitan Area. This initiative is a pivotal move towards addressing the escalating congestion issues and enhancing the safety and efficiency of the roadway network. The phased implementation of managed freeway strategies such as ramp metering, queue warning, dynamic speed advisories, and dynamic lane use control on the I-40 corridor is expected to substantially improve travel times, reduce congestion, and increase overall roadway safety. These efforts signify a transformative approach to traffic management, focusing on the practical impact of deploying advanced technological solutions to create a more responsive and efficient transportation system. The I-40 ATM project, in particular, is expected to demonstrate the benefits of managed freeways, showcasing how strategic investments in technology can lead to better mobility and safer travel conditions.

2. State-of-the-Art and Practice

In the U.S., the Transportation Research Board Committee on Freeway Operations (ACP20) has been leading an effort in tracking information on ATM projects around the country. The National Operations Center of Excellence (NOCoE) developed a map outlining projects related to ATM strategies and sub-strategies. Figure 2-1 depicts the 28 states that have deployed at least one active traffic management strategy. The ATM strategies mainly include the following:

- Adaptive Ramp Metering
- Coordinated Adaptive Ramp Metering
- Dynamic Ramp Metering
- Adaptive Traffic Signal Control
- Dynamic Junction Control
- Connected Vehicles
- Dynamic Routing
- Dynamic Lane Use Control
- Dynamic Shoulder Lanes
- Dynamic Speed Limits
- Dynamic Speed Limits and Ramp Metering
- Dynamic Speed Limits/Advisories
- Dynamic Wayfinding
- Part Time Shoulder Use
- Signal Operations
- Transit Signal Priority

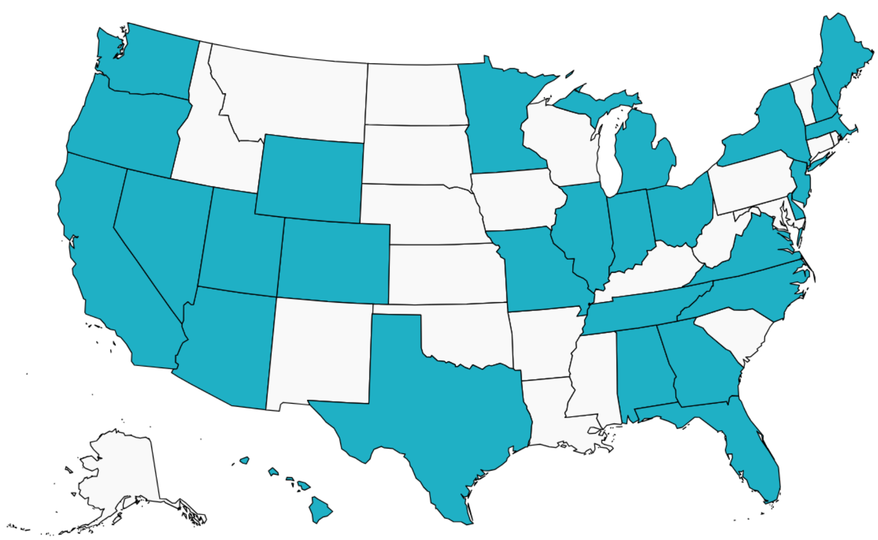

Figure 2-1. Active Traffic Management Deployment in the U.S.

Source: https://transportationops.org/atm

This literature review focuses on ATM strategies strictly deployed on freeways; the remainder of this section provides a comprehensive review of five commonly used freeway ATM strategies: Adaptive Ramp Metering, Queue Warning, Dynamic Speed Control, Dynamic Lane Use Control, and Hard Shoulder Running.

2.1 Adaptive Ramp Metering

Ramp meters use traffic signals at a freeway on-ramp roadway to break up the vehicle platoons that are released at the upstream signalized intersection and thus regulate the rate at which vehicles enter the freeway. This strategy is aimed at addressing freeway recurring congestion problems and improving safety issues at the freeway merge point. A number of studies have been performed to evaluate the benefits of ramp metering. According to a Minnesota study the average benefit/cost (B/C) ratio is approximately 20:1, with an range from 4:1 to 62:1, depending on study locations and metering strategies (MnDOT, 2001).

Like regular traffic signals at intersections, an individual ramp meter can be either pre-timed or traffic responsive; by coordinating a series of ramp meters along several freeway segments, ramp metering can also be operated in an adaptive mode. This literature review focuses primarily on coordinated adaptive ramp metering (CARM), which implements site-specific metering rates at individual entrance ramps based on traffic flow performance measures. Under the CARM approach, metering rates for each metered ramp are calculated in real-time and in response to traffic conditions on the metered ramp – the adjacent freeway and in coordination with observed conditions throughout the entire covered network – to provide the most equitable wait time for all who utilize the freeway. An overview of the states that have deployed adaptive ramp metering projects is illustrated in Figure 2-2 and detailed in Table 2-1.

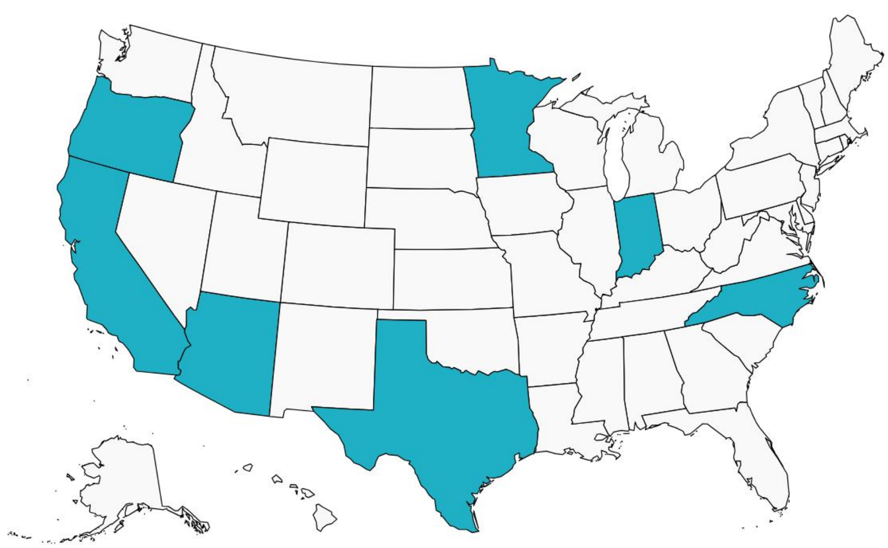

Figure 2-2. States that Deployed Adaptive Ramp Metering Projects

Table 2-1. Summary of Adaptive Ramp Metering Projects in the U.S.

State	City/Region	Adaptive Ramp Metering Project(s)
North Carolina	Raleigh	I-540 Ramp Meters*
	Charlotte	I-85 Managed Freeway Feasibility Study**
	Raleigh	I-40 managed freeway**
Indiana	Indianapolis	I-465 Southeast Transportation Management Operations (TSMO) project
Minnesota	Minneapolis /St. Paul	Real-Time Adaptive Ramp Metering Throughout the Minneapolis/St. Paul Region
Arizona	Phoenix	AZDOT Adaptive Ramp Metering - Multi-objective, Integrated, Large-Scale, Optimized System (MILOS)
Oregon	Portland	Adaptive Ramp Metering - Entire Portland metropolitan area (I-5, I-84, I-205, I-405, OR217, US26)
California	Alameda County	Real-Time Adaptive Ramp Metering on I-680
	Los Angeles	Connected Corridors I-210 Pilot / Adaptive Ramp Metering
Texas	Houston	Adaptive Ramp Metering - IH 610 (North Loop Freeway)
	Houston	Hardy Toll Road and Adaptive Ramp Metering
	Houston	Westpark Tollway and Adaptive Ramp Metering
	Houston	HOT lanes and Adaptive Ramp Metering - IH 45 South (Gulf Freeway)

* Time of day operation, with traffic responsive based on occupancy and volume threshold;
**Concept of Operation. Source: https://ops.fhwa.dot.gov/atdm/approaches/adm_table/index.htm

In addition to the presented state-of-the-practice deployment of adaptive ramp metering projects in the U.S., this section summarizes some of the ramp metering algorithms developed worldwide. An overview of adaptive or traffic-responsive ramp metering algorithms that have been developed and implemented is presented in Figure 2-3 and discussed in the following text.

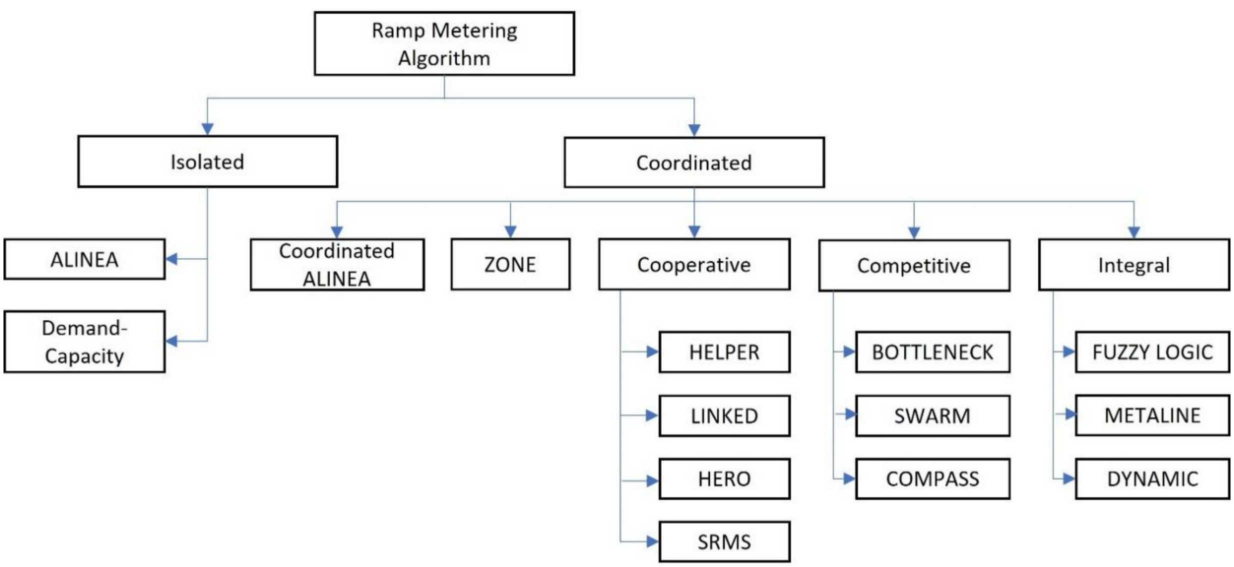

Figure 2-3. Overview of the classification of adaptive/traffic-responsive ramp metering algorithms

Asservissement Linéaire d'Entrée Autoroutière (ALINEA)

ALINEA, developed by Papageorgiou et al. (1991), is a local/isolated traffic responsive feedback control algorithm for ramp metering. The primary objective of ALINEA is to establish a metering rate that ensures the overall flow does not surpass the capacity of the freeway. The algorithm takes freeway occupancy as an input and calculates the metering rate as a control variable that varies in response to fluctuations in occupancy. ALINEA incorporates the consideration of the previous interval's metering rate when computing the rate for the subsequent interval to avoid significant variations in the metering rates for smooth operation.

Due to its simplicity and lower implementation cost, ALINEA has become the most widely employed local ramp metering strategy. It fine-tunes the metering rate to maintain the downstream occupancy of the on-ramp at a predetermined level, referred to as the occupancy set point (desired occupancy) $\hat{o}$, as outlined in the formula provided by Papageorgiou et al. (1997).

$$r(k) = r(k-1) + K_R \times [\hat{o} - o_{out}(k-1)]$$

where,

k = 1, 2, . . . discrete time index;

$r(k)$: metering rate at cycle k (veh/sec);

K_R: regulator parameter;

$\hat{o}$: optimal (or desired) value for the downstream occupancy; and

$o_{out}(k-1)$: last measured downstream freeway occupancy (%).

METALINE

METALINE, a coordinated ramp metering extension derived from ALINEA, was introduced by Papageorgiou et al. (1990). This algorithm calculates metering rates based on occupancy values collected from detectors installed on various ramps. Similar to ALINEA, METALINE exhibits responsiveness to changes in occupancy over two successive time intervals, enhancing its sensitivity to traffic variations.

Proportional Integral ALINEA (PI-ALINEA)

Wang et al. (2014) introduced PI-ALINEA as an improved version of ALINEA, specifically tailored to overcome a limitation in the original model. While ALINEA primarily targets the reduction of mainline congestion within a short distance, typically a few hundred meters downstream of the merge point, it encounters challenges when a bottleneck emerges further downstream from the ALINEA occupancy detectors. In such cases, the system struggles to detect and promptly respond to evolving congestion, allowing it to escalate over time and potentially culminate in bottleneck formation.

COMPASS

The Compass algorithm was first developed in Toronto, Canada (Lam et al., 1993). This metering algorithm provides the flexibility of both manual and automatic control over metering rates. The automated control feature calculates metering rates at regular 30-second intervals, taking into account local mainline occupancy, downstream mainline occupancy, and upstream mainline occupancy. Each recorded occupancy value is compared against threshold values stored in a look-up table to determine the appropriate metering rates. In cases where the rates from the table differ for each parameter, the most restrictive rate is applied. Additionally, the algorithm employs queue spillback detection to preemptively mitigate queue formation on ramps.

Zone Algorithm

Zone algorithms segment a freeway corridor into multiple zones, typically spanning 3 to 6 miles and may encompass both metered and non-metered ramps. These algorithms aim to uphold mainline density below a specified threshold by managing both inflow and outflow within each designated zone. The collective metering rate is derived from the calculated inflow and outflow values, subsequently distributed among all ramp meters using a predetermined ramp factor. The overarching objective is to regulate the overall volume within a defined freeway segment referred to as a "zone." Any rise in mainline density is counteracted by reducing the metering rates specifically within that particular zone (MnDOT, 1998; Levinson and Zhang, 2006).

Stratified Zone Metering Algorithm

Stratified zone metering (SZM) is a refined version of the zone algorithm, wherein ramp delays are minimized, and a stringent maximum delay limit is imposed on each ramp meter (Geroliminis et al., 2011). In the SZM method, the mainline is segmented into multiple zones based on the strategic placement of critical bottlenecks along the freeway. Ideally, each zone commences with a free-flow section and ends in a congested area. The algorithm aims to strike a balance between the density of each zone and the rates of the ramp meters. Metering rates are determined to effectively manage the incoming (inflow) and outgoing (outflow) traffic volumes for each zone in every iteration.

HELPER Algorithm

The HELPER algorithm (Lipp et al., 1991) was first deployed in the Denver, Colorado area in 1981, specifically on the I-10 freeway (Corcoran and Hickman, 1989). A comprehensive system-wide implementation was adopted, utilizing a central override control strategy that spanned the entire freeway corridor. On-ramps were categorized into six groups, each with six defined metering rates. Individual ramp meters could then select the appropriate rate based on local traffic conditions. When the queue on a ramp surpasses a predefined threshold for three consecutive time intervals, the central override feature comes into play, decreasing the metering rate by one level. This process persists until the issue is resolved. If the queue persists, the central override extends to all upstream ramps, subjecting them to more restrictive metering levels.

System-Wide Adaptive Ramp Metering (SWARM)

The System-Wide Adaptive Ramp Metering (SWARM) algorithm developed by Paesani et al. (1997) employs traffic forecasting algorithms to establish metering rates. In the case of coordinated control (i.e., SWARM1), traffic data are gathered and predictions about the future state of traffic density are made through linear regression. To account for non-linear relationships within the dataset, a Kalman Filter is applied. The coordinated metering rates are then computed based on both current and desired densities.

Fuzzy Logic Algorithm

A fuzzy logic algorithm developed by Taylor and Meldrum (1995; 1998; 2000), was implemented on the Seattle freeway. Fuzzy logic algorithms are particularly advantageous in scenarios where an accurate system model is unavailable. In this algorithm, fuzzy logic takes real-time measured values from detectors and transforms them into various textual values based on the input measurements, a process known as fuzzification. The controller logic, influenced by these fuzzified inputs, then decides on control actions, which are subsequently defuzzied to ascertain the actual values of metering rates.

Genetic Fuzzy Logic Control (GFLC)

Genetic Fuzzy Logic Control (GFLC) is an adaptive fuzzy logic control integrated with a genetic algorithm (Ghods et al., 2009; Yu et al., 2012). To calibrate the fuzzy controller, a genetic algorithm (GA) is used to tune the fuzzy sets parameters so that the total time spent in the network is minimized. Ghods's approach utilizes a fuzzy controller along with variable speed limits (VSLs) to effectively manage congestion and regulate traffic flow on freeways. Comparative analyses were conducted, pitting the fuzzy controller with and without VSLs against scenarios with no control and the standard ALINEA implementation, all operating under identical network conditions. The results demonstrated that the genetic fuzzy logic-based ramp metering leads to smoother traffic states. In Yu's work, the GA-based tuning occurred at 5-minute intervals to adapt to changes in flow density on the on-ramp. Simulation using AIMSUN showed improved total travel time compared to the fuzzy logic controller without GAs. Xu et al. (2013) utilized a fuzzy logic controller in conjunction with the particle swarm optimization method to minimize the weighted total-time-spent (WTTS) on the freeway and ramp. The controlled logic was specifically tailored for local ramp control, and simulation results from the PARAMICS microsimulation software showcased a favorable balance between freeway and on-ramp traffic.

Bottleneck Algorithm

The Bottleneck Algorithm, pioneered by Jacobson et al. (1989), demonstrates versatility in addressing both local and system-wide traffic dynamics. At the coordinated system-wide level, the control algorithm becomes operational when the occupancy surpasses a predefined threshold in a potential bottleneck region, coupled with a buildup of vehicles in the upstream area preceding the bottleneck. The coordinated control mechanism then calculates metering rates for all meters within the designated area or zone. This strategic approach aims to curtail the influx of vehicles entering the bottleneck area, aligning it with the volume present in the upstream queue on the mainline.

Advanced Real Time Metering System (ARMS)

Advanced Real Time Metering System (ARMS), developed by Liu et al. (1994) at the Texas Transportation Institute, is a proactive metering algorithm that is designed to mitigate the risk of congestion. The algorithm predicts traffic conditions and potential bottleneck locations and resolves them using real time mainline flow information. ARMS is much more complex relative to other ramp metering algorithms. However, simulation results show that the control algorithm balances the congestion resolution time with the service quality of the surface streets and improves both the freeway and surface street service.

Linear Programming

Yoshino et al. (1995) initially developed and applied a linear programming algorithm on the Hanshin Expressway in Japan. This algorithm incorporates detectors on both on and off ramps, eliminating the need for communication between ramp meters. The computation of metering rates for all ramps is achieved through intricate functions within the algorithm.

Dynamic Ramp Metering

Chen et al. (1997) introduced the dynamic control model of ramp metering, encompassing both local and coordinated control strategies for ramps. Under coordinated control, considered a strategic approach, the model employs a rolling horizon predictive algorithm to minimize system travel time. On the other hand, local control functions apply metering rates to uphold traffic conditions as directed by system-wide or coordinated control.

Linked Algorithm

The Linked algorithm, pioneered by Taylor et al. (1998) at Lancaster University in United Kingdom is a coordinated algorithm that seeks to optimize a linear-quadratic objective function that minimize total travel time and vehicle delay. It relies on the non-minimal state space (NMSS) approach to control system design. This algorithm is based on adaptive proportional integral-plu, linear quadratic (PIP-LQ) adaptive optimal controllers, which determine metering rates by minimizing a linear-quadratic function. The objective of this algorithm is to minimize a linear-quadratic cost function, and the control inputs encompass ramp flows and occupancy values derived from detectors across the entire control area.

Additive Increase Multiplicative Decrease (AIMD)

The Additive Increase Multiplicative Decrease (AIMD) algorithm, crafted by Wang et al. (2008), represents a coordinated ramp metering approach inspired by strategies employed in computer network congestion control. It operates by constraining on-ramp flow to a fraction of the ramp demand during congestion, gradually escalating metering rates to prevent queue spillover. The algorithm underwent evaluation through VISSIM simulations of Seattle's I-5 and I-90 freeways. In comparison with the fuzzy logic algorithm, the results revealed a noteworthy 28.2% reduction in travel time.

Heuristic Ramp-Metering Coordination (HERO)

Papamichail et al. (2010) introduced the Heuristic Ramp-Metering Coordination (HERO) algorithm, initially implemented on the Monash Freeway in Victoria, Australia. HERO represents an extended version of the ALINEA algorithm specifically designed for coordinated ramp control. In the HERO system, the critical occupancy is gauged at the merge area of ramps. While each on-ramp is autonomously managed using ALINEA, all ramps are interconnected through a central controller. In the event of a bottleneck emerging on the mainline near an on-ramp – signified by the queue length on the master on-ramp reaching a predefined threshold – the central controller designates the on-ramp within the bottleneck segment as the “master” ramp and designates some of its upstream on-ramps as “slaves”. Subsequently, the algorithms work collaboratively to enhance the capacity of the designated master on-ramp.

Zippered Control Strategy (ZCS)

Tan et al. (2013) proposed the Zippered Control Strategy (ZCS), employing a cellular automata-based capacity model simulated within the microsimulation software PTV VISSIM. The primary goal of this strategy is to enhance the capacity of the onramp junction. The model operates on the principle of ensuring an equitable distribution of road priority between mainline traffic and on-ramp traffic. However, it's worth noting that the model lacks any field evaluation in addition to lack of details provided about the simulation data.

The Sydney Coordinated Adaptive Traffic System RM System (SRMS)

The Sydney Coordinated Adaptive Traffic System (SCATS) serves as the foundational element of the SCATS Ramp Metering system (SRMS) approach (Aydos and O'Brien, 2014; Amini et al., 2016). To fine-tune ramp metering rates, this method calculates the accumulated occupancy error by comparing a pre-calibrated critical occupancy with the actual measured occupancy. SRMS comprises four key modules: data fusion that consolidates information from multiple traffic sources, identification of bottleneck locations, simultaneous coordination of responses across several ramps, and seamless integration with arterial traffic signals. The incorporation of the latter module renders SRMS less reliant on manual operator intervention, particularly when traffic spillbacks extend to adjacent arterials from the motorway.

A summary of the reviewed ramp metering algorithms is given in Table 2-2, which includes the tested freeway (when available), methods used for performance evaluation, and the source of the evaluation.

Table 2-2. Summary of Coordinated Adaptive Ramp Metering Algorithms

Algorithm	Tested Freeway(s)	Evaluation Method(s)	Reference(s)
ALINEA	World-wide	Field data Simulation (*PARAMICS; VISSIM*)	Papageorgiou et al., 1991 Papageorgiou et al., 1997 Chu et al., 2004 Abuamer et al., 2016
METALINE	Blvd Peripherique, Paris Milwaukee, WI (I-94; I-43; I-894, I-94, US-45) A-10, Amsterdam	Field data Simulation (*PARAMICS; SATURN and FLEXSYT*)	Papageorgiou et al., 1990 Taale et al., 1994 Horowitz et al., 2004
PI-ALINEA	No field test reported	Simulation (*Macroscopic Traffic Flow Model*)	Wang et al., 2014
COMPASS	Queue Elizabeth Way, Ontario; Highway 401, Toronto	N/A	Lam et al., 1993 Morala et al., 2008
Zone Algorithm	Minneapolis Ring Road	Field data	Stephanedes, 1994 Lau, 1997 MnDOT, 1998 Cambridge Systematics, 2001
Stratified Zone Metering	I-35, I-94, I-494	Field data Simulation *(AIMSUN)*	Lau et al., 2001 Xin et al., 2004 Michalopoulos et al., 2005 Levinson and Zhang, 2006 Liu et al., 2007
HELPER	I-25, I-225, and I-270, Denver, CO	Field data	Lipp et al.,1991
SWARM	I-5, Portland, OR; Southern California	Field data Simulation *(PARAMICS)*	Paesani et al., 1997 Zhang et al., 2001 Ahn et al., 2007
FUZZY Logic	I-5, I-90, I-405, Seattle, WA A-10, Amsterdam	Field data Simulation *(FRESIM)*	Taylor and Meldrum, 1995; 1998; 2000 Taale et al., 1996

GFLC	No field test reported	Macroscopic Traffic Flow Model Simulation *(AIMSUN; PARAMICS)*	Ghods et al., 2009 Yu et al., 2012 Xu et al., 2013
Bottleneck	I-5, Seattle	Field data	Jacobsen et al., 1989
ARMS	Not reported	N/A (Concept of Operation)	Liu et al., 1993
Linear Programming	Hanshin Expressway, Osaka, Japan	Field data	Yoshino et al., 1995
Dynamic Ramp Metering	No field test reported	Simulation *(MITSIM)*	Chen et al., 1997
Linked Algorithm	No field test reported	Simulation *(MATLAB/SIMULINK)*	Taylor et al., 1998
AIMD	No field test reported	Simulation *(VISSIM)*	Wang et al., 2008
HERO	Monash Motorway, Melbourne; M1 and M3 Motorways, Brisbane, Australia	Field data	Papamichhail et al., 2011 Faulkner et al., 2014
ZCS	No field test reported	Simulation *(AIMSUN)*	Tan et al., 2013
SRMS	84 ramps on 5 motorways in Auckland, New Zealand	Field data Simulation *(AIMSUN)*	Aydos and O'Brien, 2014 Amini et al., 2015

https://www.itskrs.its.dot.gov/its-map

2.2 Queue Warning

Queue warning systems' basic principle is to inform travelers of the presence of downstream stop-and-go traffic (based on real-time traffic detection) using warning signs and flashing lights. Drivers can anticipate an upcoming situation of emergency braking and slow down thus avoiding erratic behavior, and reducing queuing-related collisions. These systems typically employ vehicle sensors situated within and just upstream of freeway segments prone to congestion and queues caused by frequent unexpected events such as incidents, adverse weather, poor pavement conditions, or construction. Upon detecting slowed or stopped traffic, an automated warning message is displayed on a Dynamic Message Sign (DMS), often complemented by flashing lights to signify an active warning, enabling drivers to prepare to decelerate when they approach the slowed traffic. The system can be automated or controlled by a traffic management center operator. Work zones also benefit from queue warning with portable dynamic message sign units placed upstream of expected queueing points.

The Texas Department of Transportation (TxDOT) recommended several general criteria for deploying queue warning systems, including considerable peak hour congestion on a freeway facility; the presence of queues in predictable locations; sight distance restricted by vertical grades, horizontal curves, or inadequate illumination; right-of-way for overhead gantries and DMS; and a significant number of incidents related to queuing or merging. In summary, sites with the following characteristics may be prioritized for deploying queue warning systems:

- frequently congested freeways or roads;
- facilities with frequent queues in predictable locations; and
- facilities with sight distance restricted by vertical grades, horizontal curves, or poor illumination.

A number research efforts have been conducted to assess the operational and safety performance of queue warning systems. In general, it was found that queue warning systems can help reduce primary and secondary crashes by alerting drivers to congested conditions. Furthermore, incident severity is reduced because drivers are prepared for the impending downstream congestion:

An earlier study conducted by Khan (2007) developed a queue-end warning system based on a combination of traffic detection sensors and an artificial neural network (ANN) model-based algorithm for predicting queue-end location and issuing warning messages displayed on portable variable message sign (PVMS) boards at freeway work zones. A VISSIM microsimulation model was developed and calibrated for assessing the performance of the ANN-based queue-end prediction model. Another simulation-based effort conducted by Pesti et al. (2013) evaluated the performance and reliability of a queue warning system deployed at freeway work zones using microscopic traffic simulation. Various combinations of design parameters – such as speed thresholds, data aggregation intervals, update intervals, and detector spacing – were tested for identifying the most appropriate setting of these parameters. Simulation results indicated that a queue warning system with a detector spacing of half a mile could detect the back-of-queue more accurately than a system with a one-mile detector spacing. In addition, 35 mph was identified as the queue detection threshold which corresponds to a "*Stop*" traffic condition, and 55 mph as the speed threshold for a "*Slow*" traffic stream which is the same as the freeway applications recommendation.

Nowakowski et al. (2012) conducted a connected vehicle (CV)-based field experiment to assess a real-time freeway back-of-queue alerting system with the goal of improving driver situational awareness through the use of vehicle-to-infrastructure communications employed to provide drivers with an auditory alert when they approached slow traffic ahead on a freeway. The system provides soft safety alerts approximately 60 seconds before the driver is expected to reach slowed traffic. Experiment results from 24 participants showed an increase in driver situational awareness, which resulted in a smoother transition into the back of the queue and a reduction in the risk of rear-end crashes. Moreover, it was found that auditory alerts provided a small but significant reduction in mean peak deceleration rates during morning and off-peak travel as compared with baseline conditions. Khazraeian et al. (2017) investigated the accuracy and benefits of the queue warning systems (QWS) on the basis of CV data. Results indicate that a relatively low CV market penetration (about 3% to 6%) on a congested freeway was sufficient to generate an accurate and reliable estimation of the queue length in comparison with traffic detector-based measurements. In terms of safety effects of QWS, results show that QWS improved the safety conditions of the network by reducing the number of rear-end conflicts, particularly when the compliance rate with the queue warning messages exceeded 15%. Similarly, Liu et al. (2019) proposed a DSRC-based back-of-queue collision warning system and evaluated the factors that may affect the reliability of the system such as DSRC penetration rate, using real-world speed data. Results showed that the proposed queue collision warning system was effective under relatively low DSRC penetration rates.

Ullman et al. (2016) and Hsieh et al. (2017) evaluated the safety effects of a back-of-queue (BOQ) warning system combined with portable rumble strips (PRS) in nighttime work zones on Interstate 35 (I-35) in Texas. The findings of these two studies showed a 44% reduction in overall crashes, as well as a reduction in severe crashes and rear-end crashes, due to the deployment of the combined BOQ warning system and PRS. As part of the Minnesota DOT's I-94 and I-35 Active Traffic Management systems, Hourdos et al. (2017) developed and tested two separate systems that can identify lane-specific shockwave or queuing conditions on the freeway. They used existing intelligent lane control signals to warn motorists upstream of the queue as a means to prevent rear-end collisions. Based on a before-after comparison, it was found that the I-94 queue warning system resulted in a 22% decrease in crashes and a 54% decrease in near-crashes. For the I-35 system, results showed that queue warning messages helped reduce the speed variance near queue locations and the speed difference between upstream and downstream locations. In a follow-up study, Hourdos et al. (2023) further evaluated the system performance of the MnDOT queue warning algorithm. They concluded that the right-side lane model had a detection rate of 25% and a false alarm rate of 36%; the left-side lane model had a detection rate of 64% and a false alarm rate of 23%.

An overview of states and countries that have deployed queue warning systems is given in Table 2-3.

Table 2-3. Summary of Queue Warning Projects in the U.S. and Worldwide

State/Country	City/Region	Queue Warning Project(s) Locations
Minnesota	Minneapolis	I-94 Crash Prone Traffic Conditions (CPTCs) Queue Warning System I-35W SB Queue Warning System
Texas	Central Texas	Safety Effects of Portable End-of-Queue Warning System Deployments on Texas I-35 Work Zones
	Houston	Operational Review of Slow Speeds on DMS System on I-610 (Borchardt and Tydlacka, 2009)
Nebraska	State-wide	Intelligent Work Zone Using Automatic Queue Detection Systems
Florida	Tampa	Connected Vehicle Pilot Deployment Program Independent Evaluation: Stakeholder Acceptance and User Satisfaction Evaluation— Tampa (THEA)
Indiana	State-wide	Impacts to Traffic Behavior from Queue Warning Truck deployed on I-70, I-65, and I-74
England		M42
Denmark	Copenhagen	M3
	Vejle	E40/M60
Germany		A2, A8 Autobahn

Sources:
https://www.itskrs.its.dot.gov/its-map
https://active-traffic.tti.tamu.edu/active-traffic-management-0
https://active-traffic.tti.tamu.edu/resources/queuewarning

2.3 Dynamic Speed Control

Dynamic speed control systems (a.k.a. Variable Speed Limits, or VSL) aim at alerting drivers of potential slowdowns due to prevailing road, traffic, or weather conditions necessitating reduced speeds. In practice, speed reductions are implemented gradually, typically in increments of 10 or 15 mph between each sign location, allowing drivers to gradually adjust their travel speed as they approach the slowdown. These systems can present either a recommended (advisory) speed, or a regulatory (enforceable) speed limit, ahead of a traffic slowdown. Dynamic speed advisories have proven to be effective in reducing downstream speeds and decreasing the likelihood of crashes that could compromise freeway capacity.

In general, VSL control strategies can be divided into two broad categories: reactive and proactive approaches (Khondaker and Kattan, 2015). Reactive VSL control strategies were mainly formulated as simple “rule-based” logic, where real-time VSL decisions were altered based on preselected thresholds of traffic flow, occupancy, or mean speed. The objectives of these strategies were harmonization of speed differences and stabilization of traffic flow. VSLs were only triggered when detected flows approached capacity, and the reductions in speed variation were intended to contribute to reductions in crash frequency and severity (Zackor,1991; Smulders and Helleman, 1998; Abdel-Aty et al., 2006; Yang et al., 2020). Because rule-based strategies are reactive rather than proactive, they may experience a time lag from real-time conditions. In this case VSL actions may be deployed well after traffic may have already reached breakdown conditions, and thus VSL may not able to resolve the situation. Proactive VSL control strategies work in a preemptive manner, where future traffic conditions are predicted and remedial VSL strategies are issued to reduce the inflow to the anticipated congestion area. This approach can resolve shockwaves to prevent freeway traffic breakdowns before they actually happen (Yang et al., 2013; Hadiuzzaman et al., 2013; Luo et al., 2015). The main benefit of this method is that it eliminates the time lag between prevailing traffic conditions and the time needed to optimize the traffic conditions.

In general, past research concluded that using dynamic traffic management strategies to harmonize speed could efficiently alleviate freeway congestion (Ma and Shladover, 2016). For instance, Walker et al. (2009) assessed the impact of variable speed limits and peak-period shoulder use on freeway traffic operations and safety, and concluded that although these strategies did not increase the freeway throughput, they were found to be efficient in homogenizing traffic flow and creating safer driving conditions. Talebopour et al. (2013) employed microsimulation modeling to evaluate the impacts of speed harmonization on traffic flow characteristics and safety. Simulation results indicated that the implementation of a reactive speed limit control strategy significantly improved traffic flow characteristics under congested conditions. Ackaah et al. (2016) assessed the harmonization potential of VSL systems using field collected speed data. The speed coefficient of variation (Standard Deviation/Average) was used to quantify speed variation, and consistency was assessed by observing the consecutive dynamic changes in the displayed speed limits. Based on a real-word case study, it was found that a VSL system has the ability to reduce the speed differential while exhibiting consistency in the displayed speed limits.

An overview of the states that have deployed dynamic speed control projects is illustrated in Figure 2-4 and detailed in Table 2-4.

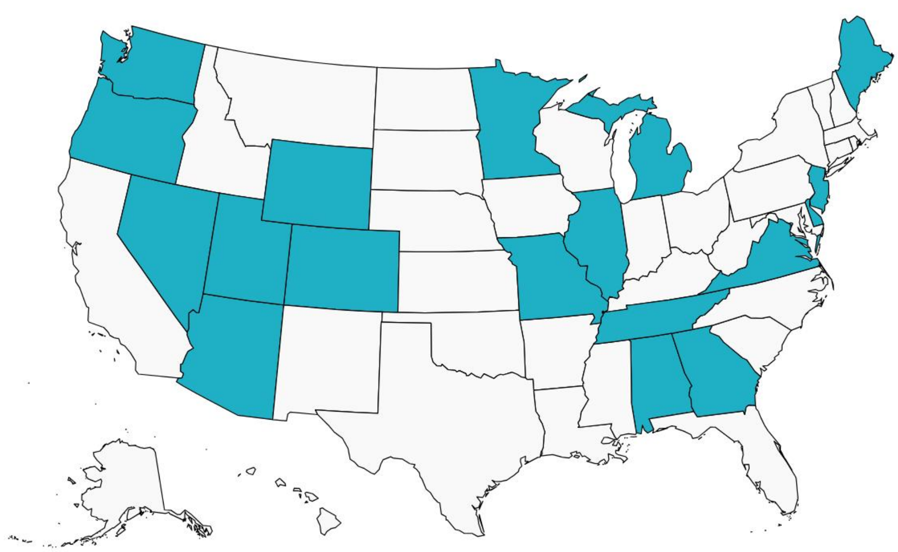

Figure 2-4. States that Deployed Dynamic Speed Control Projects

Table 2-4. Summary of Dynamic Speed Control Projects in the U.S. and Worldwide

State / Country	City/Region	Dynamic Speed Control Project(s)
New Hampshire	Pembroke	Variable Speed Limit on I-93
North Carolina	Multiple counties	Standard Practice for Variable Speed Limits in Work Zones
Nevada	Truckee River	Variable Speed Limit and RWIS on I-80
Arizona	Phoenix	I-17 ATM Pilot
	Flagstaff	Variable Speed Limit on I-40
Washington	Seattle	SR 520 / I-90 - Active Traffic Management
		I-5 - Active Traffic Management
		Adverse Weather Variable Speed Limit on I-90 and US-2
Oregon	Pendleton	Variable Speed Limit on I-84
	Portland	Variable advisory speed limits on I-5 and I-405
		ATM System on OR-217
		Variable speed limit on US26 and I-84
		I-5 ATM: Marine Dr - Fremont Bridge
		I-205 ATM
	Banks	Variable speed limit on US26 at interchange with OR47
Utah	Wanship	Variable Speed Limit on I-80
Wyoming	Cheyenne	Variable speed limit on I-80
Colorado	Denver	Dynamic Downhill Truck Speed Warning System on I-70
		Active Traffic Management System on I-70 (Phase 1 study)
Minnesota	Minneapolis	Variable speed limit and Managed Lanes on I-94
		Variable speed limit and Managed Lanes on I-94
Missouri	St. Louis	Variable advisory speed limits on I-270
Illinois	NW Surburban Chicago	Variable speed limit on I-90 from O'Hare to Elgin, IL (24 miles)
Michigan	Oakland County	I-96 Active Traffic Management
Tennessee	Knoxville	Low visibility warning system on I-75
Alabama	Mobile County	Low Visibility warning system on I-10
Georgia	Atlanta	Variable Speed Limits on I-285
Virginia	Alexandria	Variable speed limit on I-95/495 (Woodrow Wilson Bridge)
	Hampton	Variable Speed Limit on I-64 and I-664
	Hillsville	I-77 Active Traffic and Safety Management System (ATSMS)
New Jersey	Trenton	Variable Speed Limit on I-95 and I-78
Delaware	Wilmington	Speed harmonization system (weather related) on I-495
Maine	Portland	Variable Speed Limit on I-95 and I-295
Netherlands	Bussum-Muidberg	A1
	Amsterdam-Utrecht	A2
	Voorburg	A12
	Bodegraven-Woerden	A-12
	Rotterdam	A20
	Tilburg	A58
Finland	Helsinki	E18
Denmark	Copenhagen	M3
Northern Ireland	Belfast	M1, M2, M3
England	Birmingham	M25, M42
Sweden		General Practices
Greece	Athens	Attiki Odos
Germany	Rural Applications	Autobahns 8, 3, and 5

Source: https://www.itskrs.its.dot.gov/its-map ; https://active-traffic.tti.tamu.edu/resources/speedharmonization

2.4 Dynamic Lane Use Control

Dynamic lane use control (DLUC) utilizes DMS to signal the closure or blockage of individual mainline traffic lanes. DMS is typically installed above each lane, but has also been mounted near the roadway shoulder. A DLUC system continuously monitors real-time incident and congestion state collected from ITS sensors of the freeway and dynamically manage lane usage upstream of closures or blockages. By doing so, it is possible to guide traffic out of blocked lanes during incidents or construction activities, in order to mitigate congestion-related crashes. Another potential use of DLUC is to display caution messages on lane use signs over the right lane upstream of on-ramp merge area, which promotes traffic shifting out of the right lane to facilitate smoother entry for on-ramp traffic joining the freeway. This is especially critical when large platoons of vehicles enter from unmetered ramps.

Since the 1990s, a series of human factor-related studies were performed to investigate how drivers react to freeway lane-control signals to identify the effects of various traffic signs on driver behavior based on interviews, driving simulator experiments, and field experiments (Ullman et al., 1993; 1996; Harder and Bloomfield, 2012). In general, it was found that a downward diagonal yellow arrow is more effective in terms of indicating downstream lane closure rather than the yellow X currently specified in the MUTCD. The text-only “MERGE” sign performed the worst. Finley et al. (2013) summarized the DLUC systems in the U.S. and in Europe and concluded that the steady diagonal downward yellow arrow and sequential yellow chevron provided a clear message about the proper action to be taken when a motorist is in a lane under this symbol. Dutta et al. (2017) conducted a stated preference survey to evaluate driver understanding of LUCSs on I-66 and on I-95 Express lanes in Virginia. Similar to the prior reference study, survey results indicated that participants preferred the diagonal downward yellow arrow indication to the yellow X-indication. Chrysler et al. (2022) reviewed the various forms of dynamic lane-use control signal and pavement marking that have been used by different agencies, and summarized that using a downward diagonal yellow arrow to indicate downstream lane closure is more effective, and recommended using the legend SLOW in conjunction with the downward yellow arrow.

Currently, there is limited information regarding the operational and safety performance of DLUC systems. Mehr and Eskandarian (2021) investigated the effects of a lane change advance warning system on delay reductions at congested freeway diverge areas via simulation modeling. The results indicated that the proposed probabilistic prediction model that advises vehicles approaching a freeway diverge area on when to change lanes to reach an off-ramp has the potential of reducing the average delay by up to 83 percent depending on traffic flow and the ratio of vehicles exiting at the off-ramp. A Michigan before-after study (Kassens-Noor et al., 2022) revealed that the use of dynamic inside shoulder lane reduced crashes by 17 percent and yielded a benefit-cost ratio of up to 3.01. The maximum throughput increased up to 35.4 percent; during peak periods, travel times decreased by 11.2 to 16.5 percent. Gu and Han (2025) evaluated the safety and mobility effects of the combination of Variable Speed Limit (VSL) and Lane Control Signs (LCS) on the Tennessee I-24 smart. Results show that the implementation of VSL/LCS reduced serious injury, minor injury, possible injury, and PDO crashes by 18.1%, 10.4%, 17.5% and 3.8%, respectively. Moreover, the average speed increased by up to 19.2 percent with a corresponding buffer time index reduced by up to 58.9 percent.

An overview of the states that have deployed dynamic speed control projects is illustrated in Figure 2-5 and detailed in Table 2-5.

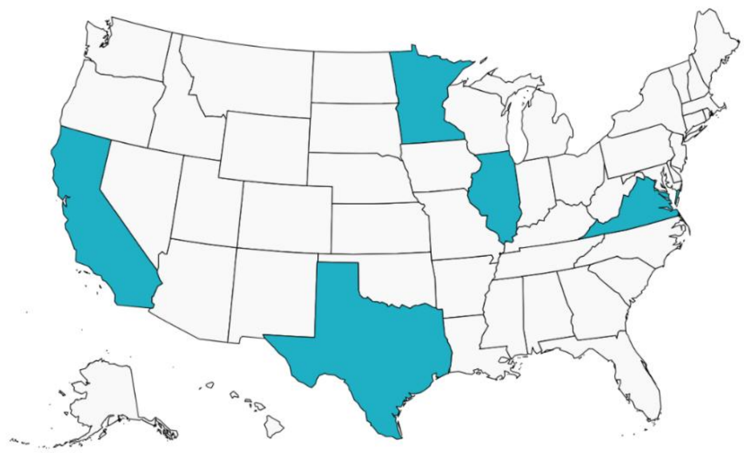

Figure 2-5. States that Deployed Dynamic Lane Use Control Projects

Table 2-5. Summary of Dynamic Lane Use Control Projects in the U.S. and Worldwide

State / Country	City/Region	Dynamic Lane Use Control Project(s)
Virginia	Centreville	I-66 ATM Project
	Falls Church	Route 267 (Dulles Access Highway)
Illinois	NW Suburban Chicago	Dynamic Lane Use on I-90 -O'Hare to Barrington, IL (16 miles)
	Southeast of Chicago	I-80 TSMO project (Indiana Borman Flex Road)
Minnesota	Minneapolis	Managed Lanes - I-394 Minneapolis; I-94, I-35W
	Minneapolis-St. Paul	US 169, MN 312, MN 5, MN 36, US 10, MN 62, I-494, MN 252, MN 77, MN 51, MN 7, MN 13
Texas	Austin	Tolled managed lanes on FM734
	College Station	SH 6
California	SF Bay Area	I-580 Dynamic Shoulder Use
	San Diego	I-805/SR 52
Georgia	Alpharetta	GA 400
Maryland	Bethesda	I-495
	Burtonsville	US-29
Massachusetts	Boston	I-93, SR 3, I-95
Ohio	Columbus	I-71
Hawaii	Honolulu	H1
Florida	Miami	I-95, SR-826, SR-836, Florida Turnpike
New Jersey	Mountainside	Route 22
	Old Bridge	US-9
Washington	Seattle	SR 520, US 2, SR 522
Tennessee	Nashville	I-24 Smart Corridor
Delaware	Wilmington	US-202
England	Not reported	M42
Ireland	Dublin	Various
New Zealand	Auckland	Northern, Northwest and Southern Motorways
Ontario	Ottawa	Regional Road 174, Regional Road 417
	Toronto	Highway 403
British Columbia	Vancouver	Route 1

Source: https://www.itskrs.its.dot.gov/its-map ; https://active-traffic.tti.tamu.edu/resources/shoulderuse

2.5 Hard Shoulder Running

Hard shoulder running (HSR), also known as dynamic shoulder use or managed shoulder lanes, is an active traffic management strategy that involves temporarily opening the hard shoulder or emergency lane of a freeway to traffic during periods of high demand or congestion. The hard shoulder is typically reserved for emergency use, breakdowns, or other unforeseen circumstances. HSR enables dynamic use of hard shoulders as an extra driving lane with the aim to increase road capacity in times of high traffic demand to help alleviate congestion and improve traffic flow. Under some circumstances, HSR is also categorized as Dynamic Lane-Use Control, since HSR is typically initiated through the use of variable message signs, lane control signals, and other intelligent transportation system (ITS) technologies. The goal is to maximize the efficiency of existing road infrastructure and provide additional capacity during peak periods.

An overview of states that have deployed hard shoulder running projects in the U.S. is listed in Table 2-6.

Table 2-6. Summary of Hard Shoulder Running Projects in the U.S.

State	City/Region	Hard Shoulder Running Project(s)
Virginia	Northern	I-66 ATM Project
		Virginia - I-495 North Shoulder Lane Use
	Virginia Beach	I-264 shoulder lane
North Carolina	Wake, Durham Counties	Bus On Shoulder System (BOSS)
Georgia	Atlanta	Dynamic Shoulder Lanes on I-85
		Shoulder lane on GA 400
Michigan	Washtenaw, Livingston County	US-23 Active Traffic Management
California	San Diego	San Diego I-805 Transit Only Lanes (TOL)
Washington	Canyon Park	Dynamic shoulder lane on NB I-405 from SR 527 to I-5
	Everett	Static shoulder lane on EB US 2 between Everett and Lake Stevens/Snohomish
	Vancouver	WB SR 14 dynamic shoulder lane
Colorado	Idaho Springs	Dynamic peak period tolled shoulder lane on I-70
Texas	Dallas Ft. Worth area	Peak period shoulder lane on State Highway 121
	Irving	Static part time shoulder lane on State Highway 161
Ohio	Columbus	Eastbound I-670 shoulder lane
New Jersey	Trenton	Static shoulder lane on NB NJ 29
Massachusetts	Boston	Static shoulder lane on I-93, I-95, SR3

Sources: https://www.itskrs.its.dot.gov/its-map

In terms of the safety and mobility benefits of HSR, Aron et al. (2010) highlighted the importance of traffic flow patterns on the safety impacts of HSR. Based on data from a French urban motorway in a before-after study, it was found that HSR resulted in a decrease in crash risks expect for dense traffic upstream and downstream of the weaving section during opening hours. Kononov et al. (2012) examined the relationship between freeway flow parameters and crash rate on selected freeways in Colorado, and found that as flow increased, the crash rate initially remained constant until a certain critical threshold combination of speed and density is reached, after which crash rate rose rapidly. This suggests that running traffic on hard shoulders during peak periods can lead to a decrease in crash rates due to the lower traffic volume or density per lane. In Germany, Geistefeldt et al. (2012) found that using the hard shoulder as an additional lane during peak hours can increase freeway capacity by 20-25%. In Italy, Guerrieri et al. (2016) found that HSR can increase capacity by up to 35% without significantly affecting safety conditions. Ma et al. (2016) proposed the concept of dynamic hard shoulder running (D-HSR) as a traffic incident management strategy; based on simulation modeling, they found that opening shoulders during incidents can significantly reduce average delay and increases total throughput. Moreover, the length of opened shoulders downstream of the incident does not affect travel delay and throughput, and short openings of the shoulder (e.g., a length of 0.5 mi) can best utilize the shoulder's capacity for incident management. Farrag et al. (2020) examined the mobility effects of dynamic hard shoulder running via simulation. The results show that implementing a hard shoulder running strategy resulted in a reduction of 50%-57% in delay, a decrease of 41%-44% in fuel consumption and emissions, and an increase of 15%-18% in bottleneck throughput, with an overall B/C ratio of 1.53. Waleczek et al. (2021) examined the long-term safety effects of HSR using accident data from seven German freeway sections with HSR over a 13-year period. Results show that the crash occurrence depends on the prevailing traffic conditions, with congestion being the most critical traffic state in relation to safety. Therefore, safety improvements upstream of HSR segments can be related to the improved traffic flow and the reduction of congestion. Although the analysis of crash frequencies subsequent to the implementation of HSR showed no significant reduction or increase in annual crash rates or crash cost rates, the study nevertheless provided evidence that the implementation of HSR can improve road safety if state-of-the-art traffic control technology is applied and congestion can be relieved.

3. Use Case Implementation

3.1 VicRoads Managed Motorway

The Victoria Department of Transport and Planning (VicRoads) introduced the managed motorways concept in Melbourne, Australia starting operations with the M1 Motorway in the early 2000's. The M1 motorway is a 47-mile-long managed facility that includes coordinated adaptive ramp metering at 62 ramps, variable speed limit (VSL), lane use control system (LUCS), priority ramp bypass for transit, and traveler information (Burley et al., 2013).

The M1 Managed Motorway focuses on preventing breakdown of flow and congestion, where the planning and design of the managed motorway system mainly considered the following principles:

- Understanding how many motorists will use the motorway, particularly during peak periods
- Providing adequate, balanced capacity along the route to best accommodate traffic needs
- Designing the mainline geometry to minimize turbulence including weaving areas, entry ramp merging and diverges to exit ramps
- Providing ramp designs that operate safely and enable management of traffic entering the motorway

Vicroads (Gaffney et al., 2017) described the operating principles and design guidelines and recommended the best practices for installing gantries, signs, and signals for different components of the managed freeway. Later, Vicroads (Hall et al., 2019; Gaffney et al., 2019; Burley et al., 2019) published a series of more holistic and standardized guidebook, describing the planning, design, and anticipated benefits of a comprehensive list of managed freeway components. An iterative design process that requires a route-based system approach to gain an understanding of current and future needs was developed for the managed motorway system design. The decisions relating to one component of design can have implications for other components, as well as costs. An overview of the main components of the system is presented in Figure 3-1 (VicRoads, 2023).

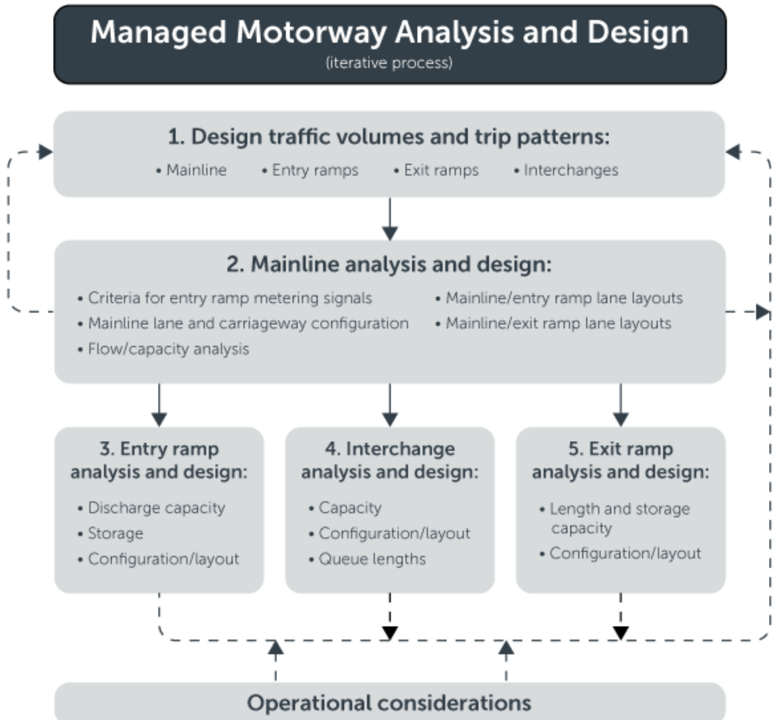

Figure 3-1. Overview of the Managed Motorway Design Process (VicRoads, 2023)

The M1 Managed Motorway system deploys coordinated ramp metering signals as the primary control, which employs an integrated approach that embraces design, intelligent transport systems, and optimized operation to prevent flow breakdown and congestion. Moreover, the system enables assisting in congestion recovery and limiting the spillback of queues upstream via real-time incident and travel time information. Ramp signals are used at entrance ramps to regulate and space out vehicles entering the motorway, which eventually will improve merging and downstream motorway operations. On the motorway mainline, CCTV cameras and vehicle sensors were deployed to monitor breakdowns and accidents. Real-time travel times and incident information are displayed at on-road message signs and overhead lane signs to manage traffic flow, particularly in an incident or when traffic congestion occurs.

Based on a series of performance assessments (Faulkner et al., 2014), it was found that the M1 Managed Motorway system yielded an increase in traffic throughput by 5-8 percent, improved traffic speeds by 35-59 percent, and improved overall travel time reliability by 150-500 percent during peak periods. In addition, the five-year average crash rate on the M1 Motorway declined by 31 percent in a comparison of before and after project completion (9.15 to 6.31 crashes per 100 million Vehicle Kilometers travelled).

3.2 Birmingham M42 Managed Motorways

The Birmingham M42 (shown in Figure 3-2) which is operated by the Highway Agencies of the UK, became a managed motorway in November 2005 when mandatory variable speed limits (VSL) came into use. Later, in September 2006, the managed motorway system included using the hard shoulder as a running lane (HSR) between junctions under controlled conditions. The hard shoulder use as an extra running lane during busy peak periods or incidents was introduced as the final phase of the ATM project.

The system includes Motorway Incident Detection and Automatic Signaling (MIDAS) loop traffic detection sensors beneath each lane at regular intervals and CCTV cameras to detect traffic speed and density. Depending on the volume of traffic and the duration of the congestion, a computer calculates the optimum speed to keep traffic moving. The advisory speed is then displayed on the electronic signs on the gantry overhead (Road Traffic Technology, 2023).

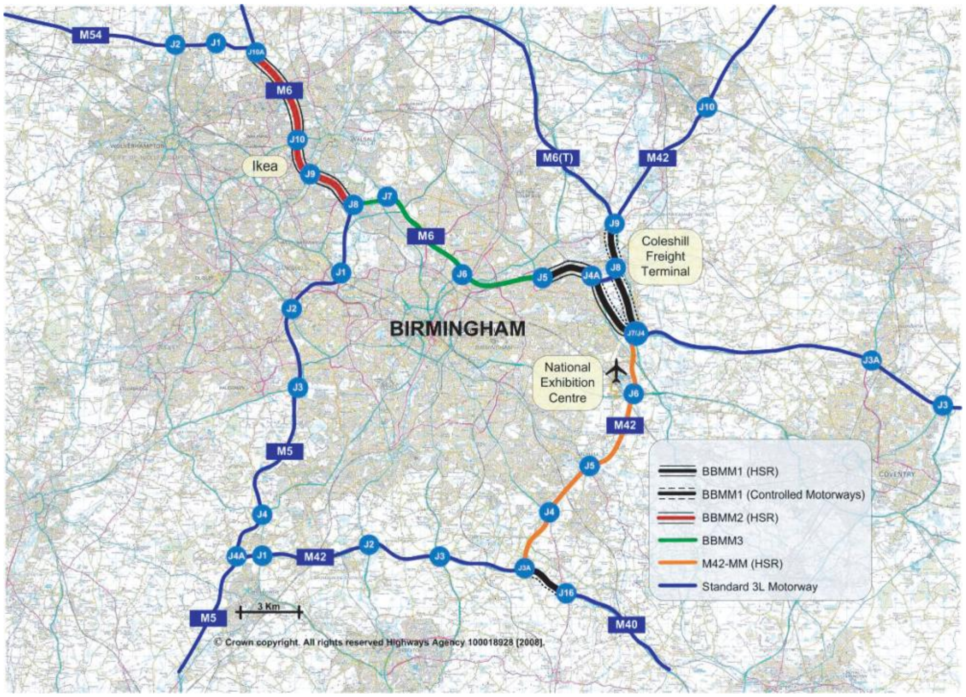

Figure 3-2. The Birmingham M42 managed motorway, United Kingdom (van Vuren et al., 2012)

Several studies assessed different aspects of this program via both field data and microsimulation modeling (Mott MacDonald, 2008; Millard and Unwin, 2009; van Vuren et al., 2012).

Mott MacDonald (2008) originally completed an evaluation of the impact of the M42 managed freeway system; his evaluation results show that throughput increased during HSR operation by an average of 7% to 9%. Moreover, travel time variability was reduced by an average of 27%, which enabled drivers to more accurately estimate their travel times. Driver compliance with hard shoulder use was 97% when the posted speed limit was 50 mph, compared to 93% when the speed limit was reduced to 40 mph. In general, the occurrence of severe congestion was reduced during HSR operation.

Millard and Unwin (2009) developed and calibrated a VISSIM model for this managed freeway to assess the operational benefits under different scenarios. The existing hard shoulder was modeled as a regular lane that is closed to all vehicles by using signals and lane closures. With the activation of HSR, these signals change to green, and lane closures are lifted dynamically according to the program. Virtual detectors were coded in VISSIM to mimic the MIDAS loops and measure the flow and speed during the simulation. The simulation algorithm also dynamically changes driver behavior to simulate VSL through a COM interface. Specifically, the variable speed limits were set by changing the speed distributions used at desired speed decisions. The hard-shoulder was opened by changing signal heads to green and de-activating reduced speed areas; the hard-shoulder was closed by reversing this process. The performance metrics used were the number of completed journeys in the simulation period, vehicle-miles traveled, network throughput, total travel time, and average number of stops, all of which the managed freeway had a positive effect, particularly at locations where large volumes of traffic exit the motorway.

Van Vuren et al. (2012) summarized the evaluation efforts of 5 years of monitoring and modeling of the Birmingham M42 managed motorway. Consultation results show that users had a positive opinion of managed motorways due to the perceived benefits in terms of driver comfort, journey time, and safety. Moreover, an analysis of wider economic benefits indicated that an additional 15% can be expected over and above traditional travel time benefits, driven largely by the motorway users with commuting, business, and commercial trip purposes. This implies that managed motorways and hard shoulder running will be most beneficial elsewhere in congested urban situations.

3.3 Colorado Smart 25 Managed Motorways Pilot

Following the success of the Melbourne managed motorway system, the Colorado Department of Transportation (CDOT), in association with Transmax and VicRoads, deployed a pilot test of Coordinated Adaptive Ramp Metering (CARM) on 23 ramps of I-25 near Denver (Figure 3-3). The goal was to reduce travel times, improve travel time reliability and increase traffic flow without the major expense and community impacts of further freeway physical capacity expansion (CDOT, 2023).

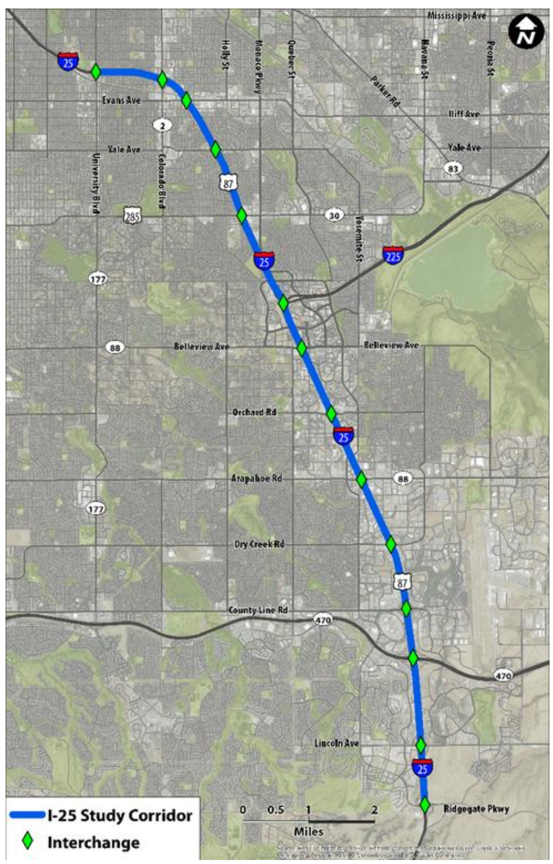

Figure 3-3. The I-25 Managed Freeway corridor (CDOT, 2023)

The CDOT CARM system utilizes the HERO ramp metering algorithm along with real-time data from advanced traffic detection systems. This allows the system to adjust the entry of vehicles onto the freeway, with the objective of minimizing the formation of bottlenecks. The monitoring components include collecting and processing mainline and entrance ramp demands, monitoring ramp and ITS device functional status, ramp signal control, queue warning, emergency access, and incident detection. All ramp meters coordinate and work cooperatively to respond to real-time data, minimize flow breakdown, and prevent the likelihood of negative impacts to adjacent arterials (CDOT, 2023; WSP; 2023). Ramp signals switch on only when needed according to the prevailing traffic flow conditions. The CARM system determines the optimum metering rate for each entry ramp to balance queues and waiting time; signal timings change every 20 seconds in response to freeway conditions. Moreover, ramp queues are managed to be accommodated by the storage space provided by all ramps to prevent queue backups into any upstream arterials.

The CARM system was activated incrementally since 2021 to allow for an adjustment period for drivers and enable system tweaks. Performance assessment efforts were made after the activation of the CARM system (CDOT, 2021; WSP; 2023). The following measures of effectiveness (MOEs) were employed: average traffic flow, speed, travel time reliability, productivity, and crash frequency. In general, the implementation of the CARM system yielded multiple benefits (WSP; 2023), including:

- A 19% reduction in overall travel time during PM peak period
- Decrease in the severity and duration of congested periods by an average of 14.3%
- Increase in travel time reliability. The average reduction in the 95th percentile travel time was 5.4 minutes
- A 42.2 % decrease in total travel delay
- Increase the maximum flow of traffic through known bottleneck locations, making traffic flow more stable and resilient to breakdown
- Improve safety by reducing the formation of bottlenecks, flow breakdown, and the likelihood of rear-end and side-swipe crashes.

3.4 Virginia I-66 Active Traffic Management System

In 2015, an active traffic management (ATM) system was installed on a 12.4-mi corridor of Interstate 66 from Centreville (Exit 52) to the Capital Beltway (Exit 64) in Virginia, which aimed at improving safety and operations on I-66 without physically expanding the existing roadway. Twenty-two gantries were installed in each direction at an average spacing of 0.6-mi to continuously deliver information to drivers (Dutta et al., 2019). The key four ATM strategies that were deployed include:

- Advisory Variable Speed Limit signs (AVSL) posting dynamic advisory speed limits on overhead gantries above each lane. The posted speed limit on this corridor is 55 mph, but the AVSL can post advisory speeds between 35 mph and 50 mph based on the prevailing traffic flow conditions. An algorithm determines the desirable posted speed limit on the basis of observed traffic speeds from sensors, which are then processed, smoothed, and grouped to create transitions into and out of congestion.
- Lane-use control signals (LUCS) that provide information on lane availability when an incident or work zone is present. Drivers are advised that a lane is open (down green arrow), a lane is closed ahead (diagonal down yellow arrow), or a lane is closed (red X).
- Dynamic hard shoulder running (HSR), which dynamically opens and closes the shoulder to traffic on the basis of traffic conditions. Prior to the activation of HSR, the right shoulder on I-66 was open to the peak traffic direction during the workday peak period. After HSR activation, the

shoulders were kept open during those times of the day, but could also be dynamically opened and closed outside those times whenever congestion formed.

- Queue warning systems (QWSs) provide advanced notice to drivers on the occurrence of congested roadway conditions downstream using variable message signs, and working in tandem with AVSLs to provide early notice of slow or stopped traffic ahead.

Sections of the corridor had all four components' gantries, while others did not have HSR, as depicted in Figure 3-4.

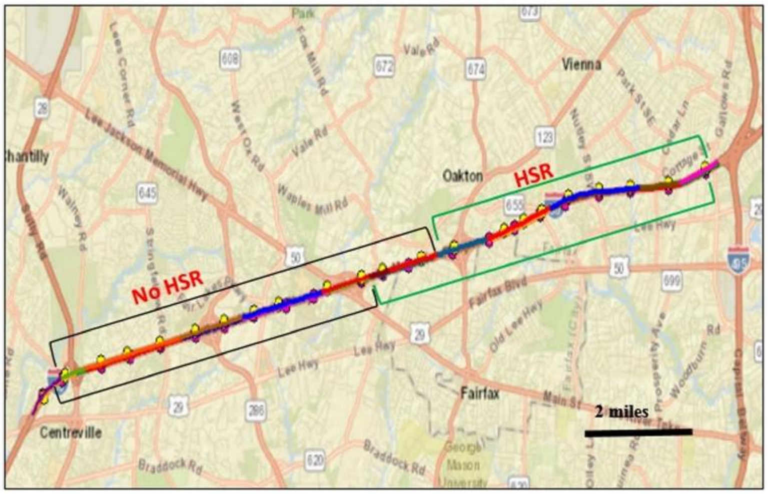

Figure 3-4. HSR locations on I-66 (Reprinted from Dutta et al., 2019)

Chun and Fontaine (2016; 2017) and Dutta et al. (2018; 2019) assessed the operational and safety benefits of the I-66 managed freeway. For traffic operational assessment, the following performance measurements were employed: ATM utilization, average travel times, and travel time reliability. To acquire segment travel times, probe-based travel time data from INRIX were used for the operational analysis. The evaluation revealed a significant improvement in average travel time and travel time reliability during weekday off-peak periods and on weekends, which was mainly attributed to HSR. However, the peak hour performance remained virtually unchanged (Chun and Fontaine, 2017).

The safety effects of the I-66 ATM system were evaluated using the Empirical Bayes methodology with safety performance functions (SPFs) using crash data collected three years before and two years after system activation. Local SPFs with annual calibration factors such as driving behavior and weather were used to generate predictions (Dutta et al., 2019). The safety evaluation results showed 4%, 4%, and 6% reductions in total, multiple-vehicle, and rear-end crashes across the entire corridor. Segment-level analysis showed that the most safety benefits were observed at locations with HSR (31% to 38% crash reductions), and no significant reductions were found in the sections with only AVSLs and LUCS.

3.5 Minneapolis I-35W Variable Advisory Speed Limit System

The Minneapolis ATM system was deployed on Interstate 35 West (I-35W) in two phases between 2009 and 2010. The original system covered sixteen miles of I-35W located south of Minneapolis and was extended by two miles in 2011 (MnDOT, 2023). The system consists of electronic signs over lanes of traffic to provide real-time information to help motorists make informed decisions about their commute. The signs display pertinent information about road conditions to improve traffic flow, reduce congestion, and improve safety.

The ATM system included loop detection, dynamic message signs, ramp meters, cameras, and Intelligent Roadway Information System (IRIS). A series of overhead signs (a.k.a. Intelligent Lane Control Signals, ILCS) above each lane were used to inform drivers of upcoming conditions or controls in place, such as which lanes are closed or blocked because of a crash or obstruction, and which events impact adjacent lanes.

The ATM system essentially serves four key purposes:

- Inform drivers when the left lanes are open to High Occupancy Vehicle (HOV) and/or HOT lanes;
- Inform drivers of advisory speed limits to slow traffic that is approaching stopped traffic or the safety speed during inclement weather;
- Inform drivers of lanes closed, such as when dynamic shoulder lanes are closed or when general purpose lanes are closed due to a crash, work zone or stalled vehicle;
- Inform drivers of hazards such as standing water or debris on the roadway and encouraging travelers to merge away from hazardous lanes.

Kwon and Park (2015) evaluated the effectiveness of the I-35W variable advisory speed limit (VASL) system on the improvement of the traffic flow using real traffic data. The analysis results showed that the average maximum deceleration was decreased by 22%, indicating a decreased risk of collisions. Moreover, the traffic time reliability was improved by up to 32% during the peak hour period after the VASL activation.

3.6 Seattle I-5 Variable Speed Limits and Lane Control

The Seattle NB I-5 section in the vicinity of the Boeing Access Road to Interstate 90 was the first modern Active Traffic and Demand Management system in the state of Washington. It was activated in August 2010. Fifteen overhead gantries, spaced approximately 0.5 miles apart through the 7-mile project area, along with 97 electronic signs, were deployed to display dynamic speed limits, lane closures, merge arrows, warning information, and variable messages (WSDOT, 2023). By providing real-time advanced notice of traffic conditions ahead, it is expected that drivers are alerted earlier and can respond sooner, reducing the need for last-second avoidance maneuvers or emergency braking, both of which are primary factors that contribute to collisions.

Speed adjustments are automated and based on measured changes in traffic states from embedded sensors. Lane control changes are human-initiated, but software automates the process of updating the signage and messages. When traffic flows normally, the overhead signs are blank, and the standard 60 mph speed limit is displayed on the large signs on each side of the highway. Speed limits can be reduced - sometimes lane by lane - to address congestion or backups and encourage drivers to slow down early, rather than making a sudden stop or braking maneuver. Variable speed limit signs show a minimum of 40 mph, even if traffic is moving slower. Similar to VSL, Lane control changes are human-initiated with software automating the process of updating the signage and messages. The lane management signs can display messages warning of slowdowns, backups, or incidents ahead, including if the road is blocked in some way; eventually, direct drivers to change lanes before they reach an incident site (Nisbet and Hammond ,2013; Ishimaru et al., 2014).

4. Summary

Managed freeways represent a modern and adaptive approach to transportation management that leverages advanced technologies to improve freeway performance without requiring costly physical capacity expansions. These systems utilize advanced traffic management and control strategies to dynamically regulate traffic demand, adjust speed limits, manage lane usage, and provide real-time information to motorists. Typical managed freeway components include ramp metering, variable message signs, electronic tolling systems, intelligent transportation system (ITS)–based traffic and environmental monitoring devices, and other coordinated traffic control strategies.

A defining characteristic of managed freeways is their ability to actively manage traffic conditions in real time. By continuously monitoring roadway conditions, these systems can respond to congestion, incidents, or fluctuating demand and optimize the use of existing infrastructure. Through the coordinated application of these technologies, managed freeways aim to reduce congestion, improve travel time reliability, and enhance the overall driving experience. In addition, real-time monitoring and control can contribute to improved roadway safety by enabling quicker responses to changing traffic conditions.

Despite their potential benefits, the implementation of comprehensive managed freeway systems in the United States has been relatively limited. To date, only one pilot implementation of a fully integrated managed freeway system has been deployed in the country. In contrast, Active Traffic Management (ATM) strategies, which are key components of managed freeways, have received considerably greater attention and have been implemented in several metropolitan areas (see Table 4-1). Notable deployments include locations such as Centreville, VA; Minneapolis, MN; and Seattle, WA, where strategies such as Dynamic Lane Use Control, Dynamic Speed Limits, Queue Warning, and Adaptive Ramp Metering have been applied. These applications demonstrate the diverse ways ATM strategies can be implemented across the country, supported by technologies including roadway sensors, dynamic message signs, and automated control algorithms.

Table 4-1. Implementation of Managed Freeway Strategies in the United States

Location(s)	ATM Strategy(ies)	Technologies Used
Centreville, VA	Dynamic Lane Use Control; Dynamic Speed Limits; Queue Warning; Hard Shoulder Running	Roadway sensors, Lane control/Dynamic speed limit signals, Dynamic message signs, TMC algorithms and control
Minneapolis, MN	Dynamic Lane Use Control; Dynamic Speed Limits; Queue Warning; Adaptive ramp metering	Roadway sensors, Lane control signs, Dynamic speed limit signals, Dynamic message signs, TMC algorithms and control
Seattle, WA	Dynamic Lane Use Control; Dynamic Speed Limits; Queue Warning; Adaptive ramp metering	Roadway sensors, Lane control signs, Dynamic speed limit signals, Dynamic message signs, TMC algorithms and control
Los Angeles, CA; Denver, CO; Portland, OR; Houston, TX	Adaptive Ramp Metering	Roadway sensors, Ramp meter signals, Dynamic message signs, TMC algorithms, TMC control
Mobile County, AL; Flagstaff, AZ;	Weather-driven Dynamic Speed Limits	TMC control, Variable speed limit signs, Atmospheric sensors,

Portland, ME; Truckee River, NV; Pittsburgh, PA; Knoxville, TN; Cheyenne, WY		Visibility sensors, Pavement condition sensors, Dynamic message signs

The literature review revealed that among the various ATM strategies, Hard Shoulder Running (HSR) has been shown to provide significant operational benefits under congested conditions by temporarily adding capacity to the freeway system. Dynamic Speed Advisories (DSA) can also improve safety and operations, particularly when implemented in conjunction with Queue Warning (QW) systems. In such cases, Dynamic Speed Advisories provide drivers with actionable speed guidance to reduce crash risks and severity, while Queue Warning systems enhance drivers' awareness of downstream congestion.

Ramp metering represents another widely implemented managed freeway strategy. Numerous local and coordinated ramp metering algorithms have been developed worldwide to improve freeway operations. Among local ramp metering approaches, the Asservissement Linéaire d'Entrée Autoroutière (ALINEA) algorithm has become the most widely deployed due to its simplicity, robustness, and relatively low implementation cost. Several extensions and modifications have been proposed to further refine the applicability and performance of the ALINEA algorithm. For coordinated ramp metering, several adaptive approaches are currently used in practice, including zone-based metering algorithms, fuzzy logic–based algorithms, the System-Wide Adaptive Ramp Metering (SWARM) algorithm, and the Heuristic Ramp-metering Coordination (HERO) algorithm. In addition, several proactive control approaches, such as the Advanced Real-Time Metering System (ARMS), have been evaluated in simulation environments and show promise as potential tools for preventing congestion before it occurs.

Recognizing the potential benefits of these strategies, the North Carolina Department of Transportation (NCDOT) is exploring the implementation of managed freeway concepts within the state. Managed freeway systems offer the potential to enhance traffic flow, improve roadway safety, and maximize the efficiency of existing transportation infrastructure. By learning from the successes and experiences of other deployments, NCDOT aims to apply these innovative approaches to address emerging traffic challenges and improve the mobility and safety of North Carolina's transportation network.

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Appendix B - Queue Warning and Lane Use Control Literature Synthesis and Modeling Justification

Microsimulation Modeling of Freeway Queue Warning Systems

NCDOT Research Project #2024-001

January 2025

Institute for Transportation Research and Education (ITRE)

North Carolina State University

ITRE

Executive Summary

This document provides a state-of-the-practice review on the use of microsimulation modeling to assess the effectiveness of freeway queue warning systems (QWS). In current practice, microsimulation modeling serves as a valuable alternative to field data for researchers and practitioners to simulate various traffic conditions and test the effectiveness of a proposed traffic management strategy. While microsimulation modeling offers valuable insights into the system performance under various scenarios, there are several challenges, such as the need for model calibration and validation to ensure realistic simulation outputs.

The literature review revealed that most studies have focused on implementing QWS within freeway work zones, with limited attention to managed freeway systems. Furthermore, effectiveness assessments have predominantly emphasized the reliability of QWS algorithms rather than evaluating their operational or safety performance. Key challenges in microsimulation modeling of QWS mainly include accurately capturing queue dynamics and calibrating driver behavior in response to queue warning messages.

As a result of these challenges, the application of microsimulation modeling for assessing the operational performance of QWS remians limited. In summary, although microsimulation modeling has demonstrated as a valuable tool for analyzing and optimizing freeway queue warning systems under a variety of traffic flow conditions or traffic control strategies, addressing challenges such as data requirements and driver behavioral modeling will be critical for advancing this field.

Contents

1. Background

Queue warning systems are advanced traffic management solutions designed to detect and communicate the presence of traffic queues to upstream drivers. These systems typically employ vehicle sensors situated within and just upstream of freeway segments prone to congestion and queues caused by frequent unexpected events such as incidents, adverse weather, poor pavement conditions, or construction. Upon detecting slowed or stopped traffic, an automated warning message is displayed on a Dynamic Message Sign (DMS), often complemented by flashing lights to signify an active warning, enabling drivers to prepare to decelerate when they approach the slowed traffic. The system can be automated or controlled by a traffic management center operator. Work zones also benefit from queue warning with portable dynamic message sign units placed upstream of expected queueing points.

In current practice, field collected traffic performance data remains the primary and most reliable source for evaluating the operational and safety impacts of managed freeway strategies, including queue warning systems *(Halmark, et al., 2020; Sakhare, et al., 2022; Hourdos et al., 2023)*. Field data provides direct, real-world insights into traffic conditions, enabling accurate assessments of system effectiveness in reducing congestionand enhancing safety. However, practical challenges such as the high cost, time consumption, and logistical complexities associated with collecting field data have led to growing interest in alternative methods. Integrating microsimulation modeling as a complement or alternative to field-collected traffic data has become increasingly popular. Microsimulation offers a versatile and cost-effective solution for analyzing the potential performance of freeway management strategies under a wide range of scenarios. This method is particularly valuable when field data is unavailable, such as during the planning phase of new projects, or when collecting it is prohibitively expensive or operationally infeasible.

The U.D. Department of Transportation (USDOT) and similar agencies often leverage simulation technologies to enhance their project' planning, design, and implementation. By integrating microscopic simulation modeling with traditional field-collected data, transportation agencies can better assess, design, and implement managed freeway strategies, which supports more informed decision-making and resource allocation.

2. FHWA Analysis Modeling and Simulation (AMS) Programs

The U.S. Federal Highway Administration (FHWA) has devoted significant research efforts in developing Analysis, modeling, and simulation (AMS) frameworks and investigate the application of AMS methodologies to address transportation system challenges. One primary focal area is Congestion Management and Mitigation, where various AMS tools were developed to demonstrate the benefits of alternative operational strategies in reducing congestion and improve mobility at bottleneck locations. Projects in this area mainly include testing conceptual freeway and arterial designs that never implemented in practice, traffic bottlenecks identification and solutions, and active traffic management strategies *(FHWA, 2021)*.

The AMS program also seeks to develop and disseminate best practices on the appropriate application of AMS tools and methodologies as well as explore practical improvements to the state-of-the-practice. *Silverstein et al. (2024)* conducted a literature review and expert survey to summarize the simulation tools that have been commonly used in the USDOT's AMS programs. A total of 22 AMS for ITS tools used in practice were categorized into the following categories: sketch planning, travel demand models, analytical models, traffic optimization tools, macroscopic simulation, mesoscopic simulation, and microsimulation simulation models. For each AMS tool, the following information were summarized: tool description; tool category(ies); primary purpose(s)/objective(s); scope; data inputs/requirements; performance measures; known entities that use the tool and select use cases; and a summary of the strengths and gaps/challenges in regard to practitioner use. The state-of-the-practice literature review and expert interview reveal that mobility and environmental impact performance measures were the most prevalent outputs from AMS for ITS tools; while safety measures were largely absent from both stakeholder discussions as well as the literature review. However, although most AMS tools are well-equipped to model the operations of managed freeway systems and consequently being employed by various transportation agencies for operational analysys, there appears to be a lack of explicit methodologies or frameworks designed for the simulation modeling of queue warning systems.

3. Microsimulation Model Calibration

Previous stdies show that microsimulation modeling allows researchers to replicate and analyze the interactions between vehicles under various scenarios. By simulating individual vehicle movements, these models provide a detailed understanding of how Queue Warning System (QWS) impact traffic dynamics, safety, and efficiency. In practice, a crucial step in the development of a reliable microsimulation model involves calibrating the model using field-observed traffic flow and driver behavior data. Additionally, the simulation results must be validated by comparing the simulated performance measures with corresponding field observations.

In terms of calibration and validation methodologies for microsimulation models of queue warning systems, *Zhao et al. (2022)* developed a framework that can emulate traffic conditions and their impacts on freeway Intelligent Work Zone (IWZ) equipped with Automatic Queue Detection (AQD) systems. The proposed framework includes four steps: (1) developing the base work zone model, (2) replicating the AQD warning messages, (3) calibrating the key simulation parameters using three types of data, and (4) validating the resulting simulation model using segment travel times.

Specifically, the initial development of the simulation model started with developing a basic model of a generic AQD work zone including lane geometry and configuration, work zone merging locations, initial speed distributions by vehicle type, traffic demand, and so forth. Then, the AQD system logic was coded within VISSIM using the VisVAP through a series of surrogate settings such as a pair of loop detectors were placed in each lane at the beginning of the work zone taper and two desired speed decision points were added upstream of the loop detectors. The driving behavior in response to the Portable Dynamic Message Signs (PDMS) warning message was achieved through the use of desired speed distributions that are input by the users. Other simulation parameters used for model calibration include car-following headway data (such as the standstill distance, the time headway, and follow variation) and heavy vehicle acceleration/deceleration parameters.

Finally, the proposed simulation calibration framework was demonstrated on a real-world IWZ test site with AQD systems, which was a 4-mi section of I-80 in Nebraska. The simulation model was validated using two approaches. First, the simulated AQD logic was checked for the reproduction of the warning patterns. Second, the traffic volumes at the PDMS locations were compared using data from the test site. Welch two sample t-test results show that there are no significant differences between field observation and simulated output for passenger cars and heavy vehicles, indicating that the simulation model with the calibrated parameters reliably reproduces the traffic conditions at the test site.

Luo (2023) developed a three-step framework for freeway work zone queue warning system simulation model development, calibration, and validation. The process starts with the base condition model development, where work zone-related data such as study site geometry and traffic data (speed and volume), as well as relevant traffic operation data were collected and processed. Additionally, field-collected performance measures such as traffic flow and speed were collected to evaluate the quality of the developed simulation models. The second step emphasized the calibration of driver-behavior parameters under base condition by modifying three main parameters: desired standstill distance, desired time headway, and the desired truck acceleration. To achieve a well-calibrated simulation model, this process was iteratively performed until obtaining an acceptable set of parameters that ensure the simulation outputs align with field observations.

4. Microsimulation Modeling of QWS Reliability

Although the adoption of microsimulation modeling for freeway queue warning systems by state Departments of Transportation (DOTs) remains limited in practice, several research initiatives have been undertaken to evaluate the reliability and operational or safety performance of these systems.

An earlier study conducted by *Khan (2007)* developed a queue-end warning system based on a combination of traffic dection sensors and an artificial neural network (ANN) model-based algorithm for predicting queue-end location and issuing warning messages displayed on portable variable message sign (PVMS) boards at freeway workzones. A VISSIM microsimulation model was developed and calibrated for assessing the performance of the ANN-based queue-end prediction model. The research did not assess the traffic operational or safety performance associated with the the queue warning systems; however, model validation results show that the microsimulation and the ANN modelling approaches were suitable for simulating traffic operations in a work zone, and the calibration and verification steps were successful and applications of the developed models produce logical results.

Pesti et al. (2013) evaluated the performance and reliability of a queue warning system deployed at a freeway work zones using microscopic traffic simulation. Various combinations of design parameters, such as speed thresholds, data aggregation intervals, update intervals, and detector spacing were tested for identifyingthe most appropriate setting of these parameters. Simulation results indicated that a queue warning system with a detector spacing of half a mile could detect the back-of-queue more accurately than a system with a one-mile detector spacing. In addition, 35 mph was identified as the queue detection threshold which corresponds to a “Stop” traffic condition, and 55 mph as the speed threshold for a “Slow” traffic stream which is the same as the freeway applications recommendation.

5. Microsimulation Modeling of QWS Effectiveness

Several studies have used microsimulation tools such as VISSIM to assess the impact of QWS on freeway operations.

Aung (2011) developed a VISSIM simulation model of a queue warning system of Interstate 35 in Austin, Texas for demonstrating the use of a queue warning system to notify vehicles of approaching queues, and the possible use of the frontage roads as an additional route diversion. The tested freeway segment has a sharp horizontal curve, as well as vertical sight distance issues. The simulation modeling aimed to identify three factors to improve the queue warning system design: location of the detector(s), occupancy rate to trigger the queue warning system, and vehicle diverging rates. Moreover, the simulation tested the operational performance of the queue warning system by primarily measuring the travel time across different routes. Simulation results indicated that an occupancy rate of 0.20 and a speed limit of 45 mph resulted in the shortest travel time on the tested freeway segment. Improvements in the operational performance may contribute to a decrease in abrupt stops, which may be a factor in rear end and side car collisions.

Dowling et al. (2015) evaluated the effect of dynamic speed harmonization with queue warning prototype using VISSIM microscopic simulation for the US 101 freeway corridor in San Mateo, CA. Simulation results show that the Prototype significantly reduces the magnitudes of the speed drops between vehicles, which is considered to benefit safety by reducing the probability of collisions where free-flowing traffic meets the back of a queue. Nevertheless, no simulation results were generated for queue warning system, mainly due to the lack of information on the drivers’ expectancy and reaction to the queue warning messages.

Ayala (2017) employed VISSIM microsimulation model to investigate the effects of QWS on freeway traffic. A freeway work zone with a lane closure was modeled using field-gathered speed data, volumes, and vehicle length. This study proposed a set of six different speed reduction schemes in advance of the lane closure that simulated the QWS effect on three sets of non-compliant driver proportions. The QWS effects were measured according to the largest and negative speed average difference between the average traffic speeds in two consecutive 5-min intervals, which was identified to be the moment where vehicles would be more likely to be involved in an end-of-queue crash. While the research recommended that speed differential measures could be used for future safety enhancement using QWS application and deployment on freeway work zones, no measurement of operational effectiveness was conducted.

Bashir (2020) assessed the operation and safety performance of connected vehicle (CV) based queue warning applications on signalized intersection through VISSIM microsimulation modeling. For traffic operation modeling, queuing, total delays, and vehicle spacing were investigated at intersection. For traffic safety assessment, Time-to-Collision (TTC) and Post-Encroachment Time (PET) were used as Surrogate Measures of Safety (SMoS). The results revealed that the CV-based queue warning systems were effective in reducing vehicle delays by an average of 2%-5%. The simulated vehicle spacing in the vicinity of the intersections were higher (up to 134%) in CV queue warning models due to the earlier start of deceleration and lower lane speeds, which could be considered a safety benefit to prevent rear-end conflicts.

Luo (2023) developed a series of traffic simulation models with the ability to replicate field-observed work zone traffic operation under specific situations to estimate the effect of QWS on traffic flow parameters within freeway work zone. Based on the evaluation of effectiveness measures (i.e., travel time, vehicle delay, and queue length), this study concluded that with the deployment of a QWS, midnight (low-volume conditions) incident tends to have less impact on travel times, vehicle delays, and queue lengths. Beside, incident occurred around noon (higher-volume conditions) resulted in increased travel times and queue lengths through the work zone segment. Moreover, PM peak period incident did not significantly affect travel times, vehicle delays, and queue lengths.

6. Conclusions and Discussions

Overall, previous emerical data-based studies have found that queue warning systems are effective in reducing both primary and secondary crashes by alerting drivers to impending congested conditions *(Ullman et al., 2016; Hsieh et al., 2017)*. Additionally, these systems contribute to a reduction in incident severity by enabling drivers to anticipate and respond to downstream congestion in advance *(Nowakowski et al., 2012; Son et al., 2020)*.

In terms of simulation-based queue warning system performance assessment, it was found that while microsimulation modeling offers valuable insights into the system performance under various scenarios, it is not without challenges. Calibration and validation of models require high-quality data, which may not always be available. Additionally, the representation of driver behavior in response to the queue warning messages is inherently simplified in the simulation models, which potentially limits the accuracy of simulation outputs. Besides, it was found that an effective queue warning system needs to account for rapidly fluctuating queue dynamics. This indicates that warning signs positioned too close to the tail of a queue may potentially result in drivers encountering the queue before noticing the warning. Conversely, placing warning signs too far from the queue may provide drivers with inaccurate information, which could adversely influence driver behavior and subsequently impact the operational efficiency and safety effectiveness of the system.

Given these challenges, in practice the use of microsimulation modeling for assessing the operational performance of QWS remians limited. Most existing studies have concentrated on QWS implementations within freeway work zones rather than on managed freeway systems. Furthermore, effectiveness assessments have predominantly emphasized the reliability of QWS algorithms rather than evaluating their operational or safety performance.

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Appendix C: VSL and CARM Control Algorithm and Queue Flush Override Logic

CARM MODEL DEVELOPMENT

This section presents the formulation of the proposed capacity-constrained CARM algorithm and its simulation-based implementation, including the ramp clustering strategy and queue-flushing logic. The capacity-constrained CARM algorithm is designed to maximize metered on-ramp throughput, which is mathematically equivalent to maximizing freeway throughput. In simpler terms, the on-ramp flows always respect their capacities and downstream segment capacities while pushing more vehicles into the freeway. Thus, the freeway remains near-capacity and maximizes throughput. The proposed framework integrates a mixed-integer linear programming (MILP)-based CARM formulation with a queue override mechanism to effectively mitigate queue spillback conditions.

From a practical perspective, coordinating all on-ramps without accounting for their heterogeneous demands, spatial separations, communication delays, and agency-specific requirements set by the Department of Transportation (DOT) is unrealistic. To address these challenges, the framework introduces a clustering scheme that enables effective and implementable ramp coordination. After forming clusters, the proposed algorithm is applied within each cluster to optimize overall system performance.

Clustering of the Freeway Facility

A "cluster" is defined as a specific portion of the freeway with multiple on-ramps that require coordination. Each cluster is anchored by a metered on-ramp and may contain a combination of additional metered ramps, unmetered ramps, and off-ramps. For example, Figure 5 shows a typical cluster that contains three metered on-ramps, one un-metered on-ramp, and two off-ramps. There are six segments in this example cluster, and each segment's capacity varies depending on the segment lane configuration.

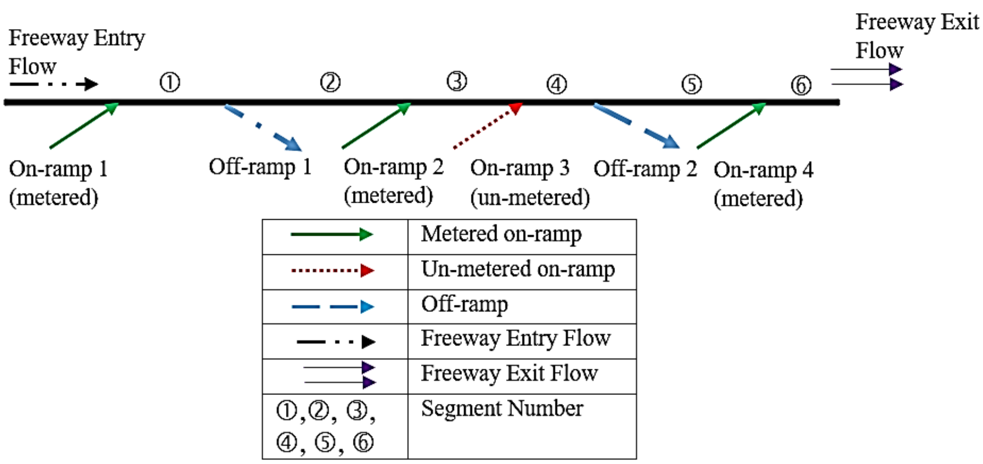


Figure 5: Typical Cluster Representation

As mentioned before, coordinating all metered on-ramps along an extended freeway corridor would be impractical; therefore, a corridor can have multiple clusters for CARM operation. Figure 6 shows a schematic diagram of a case study freeway with 3 defined clusters. The selection of a cluster depended majorly on spatial separations, high on-ramp demands that affect downstream operations, and the presence of system interchanges. For instance, the freeway facility in Cluster 2 includes system interchanges with collector-distributor (C-D) roads, adding complexity to the system.

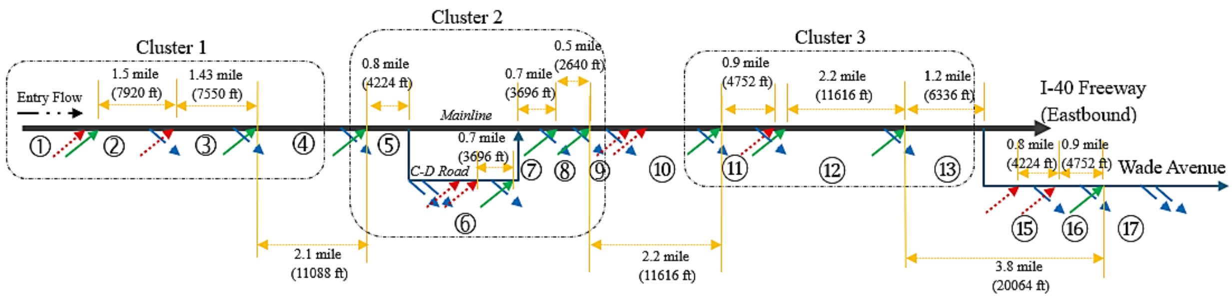


Figure 6: Schematic Diagram of Freeway Under Study

Model Formulation

CARM model is formulated as an optimization problem that determines on-ramp metering flows to maximize overall throughput while preventing freeway breakdown and ramp queue spillback. The model takes initial inputs (on-ramp demand, mainline demand) from the system, estimates future states up to a certain period, and generates optimal ramp metering solutions.

The decision variables are ramp metering flow R_i^t of on-ramp $i \in I$ at timestep $t \in T$, where I is a set of all on-ramps and T is the set of time steps. The details of sets and variables are listed in Table 2.

Table 2: List of Notations Used in the Study

Sets	
T	set of time steps
I	set of metered on-ramps
L	set of un-metered on-ramps
M	set of off-ramps
K	set of segments
I^k	set of metered on-ramp upstream of segment $k \in K$
L^k	set of un-metered on-ramps upstream of segment $k \in K$
M^k	set of off-ramps upstream of segment $k \in K$
Decision (control) variables	
R_i^t	ramp flow of on-ramp at location $i \in I$ at time step $t \in T$
Z_i^t	binary control variable of on-ramp flow at location $i \in I$ at time step $t \in T$
Variables	
F^t	entering freeway flow rate into the cluster in time step t
D_i^t	demand flow rate at ramp $i \in \{I, L\}$ in time step t
U_i^t	unmet demand at ramp $i \in I$ in time step t
E_m^t	off ramp flow at ramp $m \in M$ in time step t
Parameters	

R_m	minimum metering rate at ramp i
C_i	capacity of segment i
	A large number

The objective function (1) maximizes ramp flow R_i^t for all ramps $i \in I$ for all timesteps $t \in T$. Also, a penalty term is present to avoid infeasibility of the optimization program during off-peak hours (e.g., if demand D_i^t falls below the minimum metering rate R_m).

$$Z = \max \sum_{i}^{I} \sum_{t}^{T} \{R_i^t - M(U_i^t + 1 - Z_i^t)\} \tag{1}$$

Objective function is subject to the following constraints.

$$R_i^t \leq D_i^t + U_i^{t-1} + M(1 - Z_i^t) \qquad \forall i \in I, \forall t \in T \tag{2}$$

$$U_i^t \geq D_i^t - R_i^t \qquad \forall i \in I, \forall t \in T \tag{3}$$

$$R_i^t \geq R_m Z_i^t \qquad \forall i \in I, \forall t \in T \tag{4}$$

$$D_i^t \geq R_m Z_i^t \qquad \forall i \in I, \forall t \in T \tag{5}$$

$$F^t + \sum_{i \in I^k} R_i^t + \sum_{l \in L^k} D_l^t - \sum_{m \in M^k} D_m^t \leq C_{d,k} \tag{6}$$

$$\forall k \in K, \forall i \in I^k, \forall l \in L^k, \forall m \in M^k, \forall t \in T$$

$$Z_i^t \in \{0,1\} \qquad \forall i \in I, \forall t \in T \tag{7}$$

$$D_i^t, F^t, U_i^t \geq 0 \qquad \forall i \in I, \forall t \in T \tag{8}$$

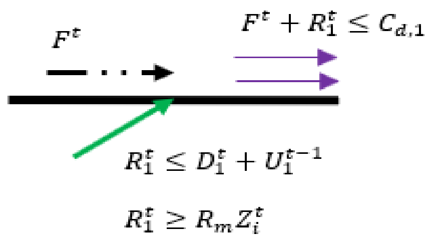
Constraints (2) ensure that the ramp flow does not exceed the accumulated demand on the subject on-ramp. The accumulated demand is the summation of the current on-ramp demand D_i^t and accumulated unmet demand from the previous timestep (U_i^{t-1}). The added term MZ_i^t simply ensures that ramp flow is maximized (restricted to capacity by constraints (*6*) when D_i^t falls below minimum metering rate R_m).

Constraints (3) ensure that $U_i^t = \max\{D_i^t - R_i^t, 0\}$. The penalizing term MU_i^t ensures that the value is restricted to $D_i^t - R_i^t$ when $D_i^t \geq R_m$ and restricted to zero when $D_i^t < R_m$. Thus, it calculates unmet demand for on ramp i at timestep t. This is added to the next time step $t + 1$.

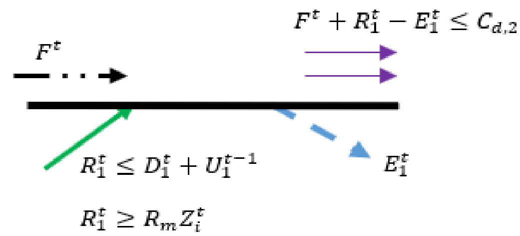
Constraints (4), (5) and (7) ensure ramp metering rate R_i^t meets the minimum rate (R_m) when $D_i^t \geq R_m$, but relaxes it when $D_i^t < R_m$, using a binary control variable Z_i^t and a large number M. Z_i^t is set to 1 to avoid large penalty when $D_i^t \geq R_m$. In case of $D_i^t < R_m$, Z_i^t is set to 0 to avoid infeasibility and to ensure ramp metering is set to capacity.

Constraints (6) ensure ramp metering rate R_i^t at timestep t obeys all downstream segment capacities of a cluster. The capacity constraints mechanism associated with each cluster is depicted in Figure 7. Figure 7.a shows that ramp flows from each metered on-ramp remain within the downstream segment capacities ($C_{d,k}$) at any timestep t. Each on-ramp flow at time t (R_i^t) is also constrained by the cumulative demand flow rate up to time t (i.e., on-ramp demand D_i^t at time t plus the unmet demand UD_i^{t-1} from time period $t-1$).

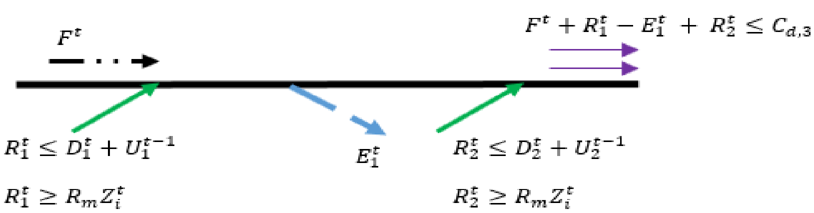
Again, demand from on-ramp 1 is not only constrained by segment 1 capacity, but it is rolled over to the downstream segments to avoid exceeding segment capacities. For instance, in time period t, the inflows to segment 3 (Figure 7.c) are freeway flow (F^t), two on-ramp flows ($R_1^t + R_2^t$) with one exiting off-ramp flow (E_1^t). Therefore, the total demand ($F^t + R_1^t + R_2^t - E_1^t$) must not exceed the capacity limit $C_{d,3}$ of segment 3. Similar constraints are shown till the end of cluster (Figure 7.f).



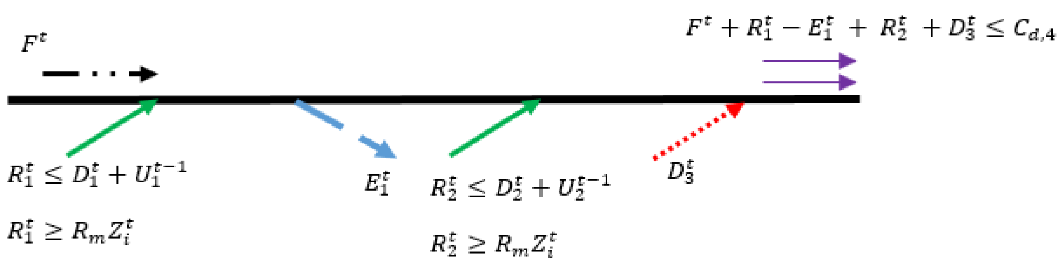
a. Constraints, segment 1



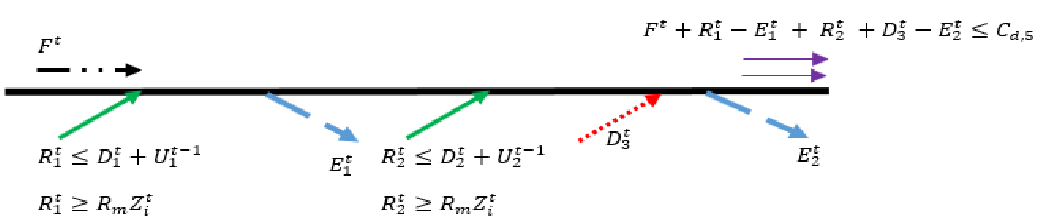
b. Constraints, segment 2



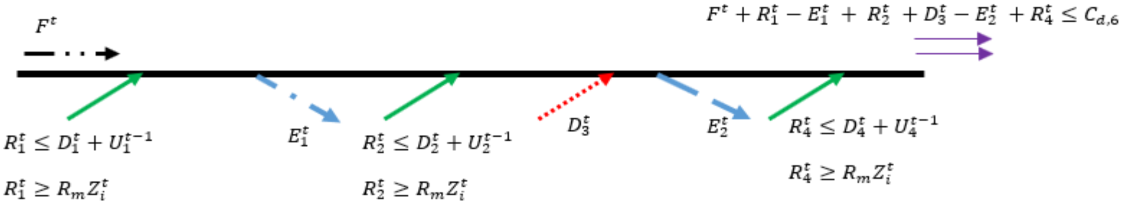
c. Constraints, Segment 3



d. Constraints, Segment 4



e. Constraints, Segment 5



f. Constraints, Segment 6

Figure 7: Ramp Metering Rate Constraints for a Typical Cluster at time t

This mechanism ensures maximum possible ramp flows from each metered on-ramp as well as maximum flows within each segment. Taking all timesteps T into account, the algorithm generates optimal ramp metering decisions for each on-ramp up to T. Therefore, a maximum throughput during the study period T is expected at the cluster exit.

On Ramp Queue Flushing Mechanism

The proposed algorithm aims to improve freeway throughput by maximizing on-ramp throughput. While it pushes vehicles from on-ramps, clearing out queues from each on-ramp may not always be possible, particularly for ramps with a high demand and limited queue storage capacity. This condition may cause queues to spill onto the feeding arterial if unaddressed. Thus, for practical implementation, agencies often add a flushing mechanism that overrides the "optimal" metering rates by activating a continuous green phase (i.e., disabling metering) at the ramp. Therefore, in this study, a flushing strategy is proposed and presented in Figure 8.

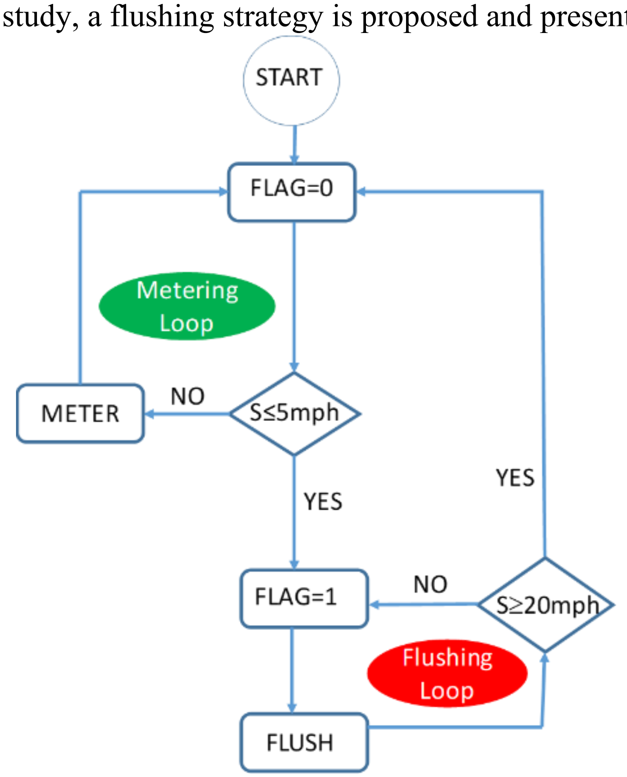


Figure 8: Flushing Logic to Prevent Queue Spillback

Queue spillback is usually measured from the field using detectors installed near the upstream end of the on-ramp. Whenever queues grow and the detector reports that the flushing metric has reached its threshold (either occupancy or speed), the ramp meter goes to full green (or off) until the metric value rises to the point where resuming metering is warranted (Chaudhary et al., 2004).
A simulated loop detector near the ramp upstream end tracks traffic speed. If the speed drops below the threshold speed of 5 mph, flushing occurs (see Figure 8) and overrides the current ramp metering rate. Once an on-ramp enters the flushing loop, the condition is checked every time step t.
To ensure that queuing has ended, a speed threshold of at least 20 mph must be met. This strategy thus avoids frequent changes from metering to flushing and vice versa.

CARM MODEL IMPLEMENTATION IN SIMULATION

The proposed CARM algorithm (*1*) - (*8*) is an MILP in nature and solvable using available commercial solvers. However, ramp metering is mostly operated during peak hours when demand usually exceeds the minimum metering rates. Therefore, it is possible to reduce the complexity of the model from MILP to LP, which is more efficient and requires significantly less computational resources. Under the strict condition $D_i^t \geq R_m$ at any timestep t, the proposed MILP with $Z_i^t = 1$ can simply be formulated as an LP model (*9*) - (*15*).

$$\mathrm{Z} = \max \sum_{i}^{I} \sum_{t}^{T} (R_i^t - MU_i^t) \tag{9}$$

Objective function is subject to the following constraints.

$$R_i^t \leq D_i^t + U_i^{t-1} \qquad \forall i \in I, \forall t \in T \tag{10}$$

$$U_i^t \geq D_i^t - R_i^t \qquad \forall i \in I, \forall t \in T \tag{11}$$

$$R_i^t \geq R_m \qquad \forall i \in I, \forall t \in T \tag{12}$$

$$D_i^t \geq R_m \qquad \forall i \in I, \forall t \in T \tag{13}$$

$$F^t + \sum_{i \in I^k} R_i^t + \sum_{l \in L^k} D_l^t - \sum_{m \in M^k} D_m^t \leq C_{d,k} \qquad \forall k \in K, \forall i \in I^k, \forall l \in L^k, \forall m \in M^k, \forall t \in T \tag{14}$$

$$D_i^t, F^t, U_i^t \geq 0 \qquad \forall i \in I, \forall t \in T \tag{15}$$

As mentioned earlier, the proposed CARM algorithm described in (*9*) - (*15*), along with a queue over-riding mechanism (shown in Figure 8), is implemented in the VISSIM simulation platform through COM programming. The overall implementation framework is depicted in Figure 9**.**

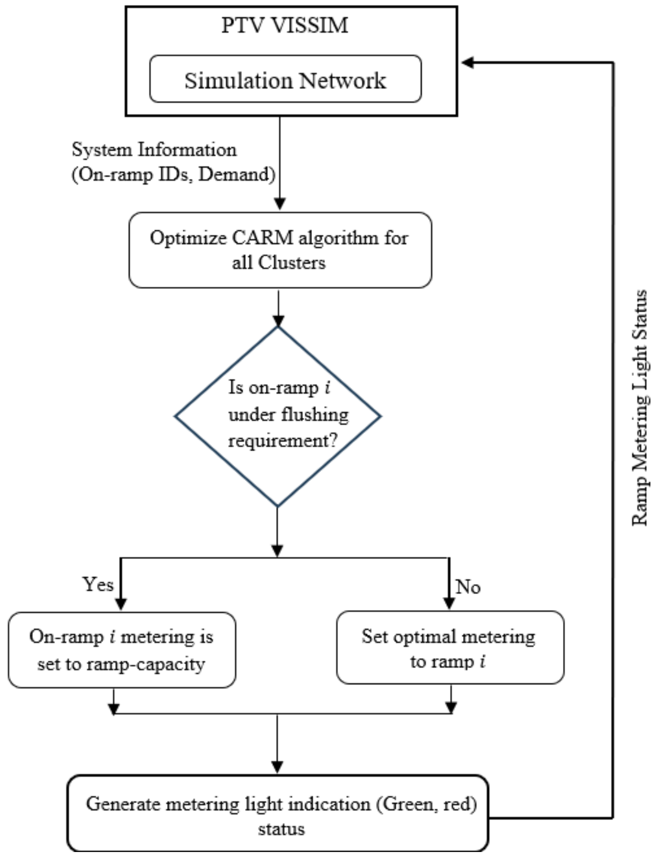


Figure 9: CARM Implementation Framework

At time t, traffic demand data along with ramp IDs are collected from the simulation model. Ramp ID information is used to cluster on-ramps based on a preselected list. Next, the CARM algorithm is solved for each cluster up to a certain period T. A flushing check for each on-ramp is performed based on the flushing strategy. The optimal decision for any on-ramp varies if it is in the flushing loop, as shown in Figure 9. Based on the decisions, ramp metering signal indications are generated. Such indications are transmitted to the simulation model for execution. The system implements the decisions and rolls over to the next time step $t + 1$ to iterate the same process until the end of the study period.

VSL MODEL DEVELOPMENT

The Variable Speed Limit (VSL) control strategy implemented in this study is based on the rule-based framework proposed by Allaby et al. (2006). This approach was selected due to its practicality, limited data requirements, and proven effectiveness in smoothing traffic flow under recurrent congestion based on multiple studies. Among the five modified VSL logics presented in Allaby et al., the recommended version was adopted.

The recommended version in mph speed unit is presented in Figure 10.

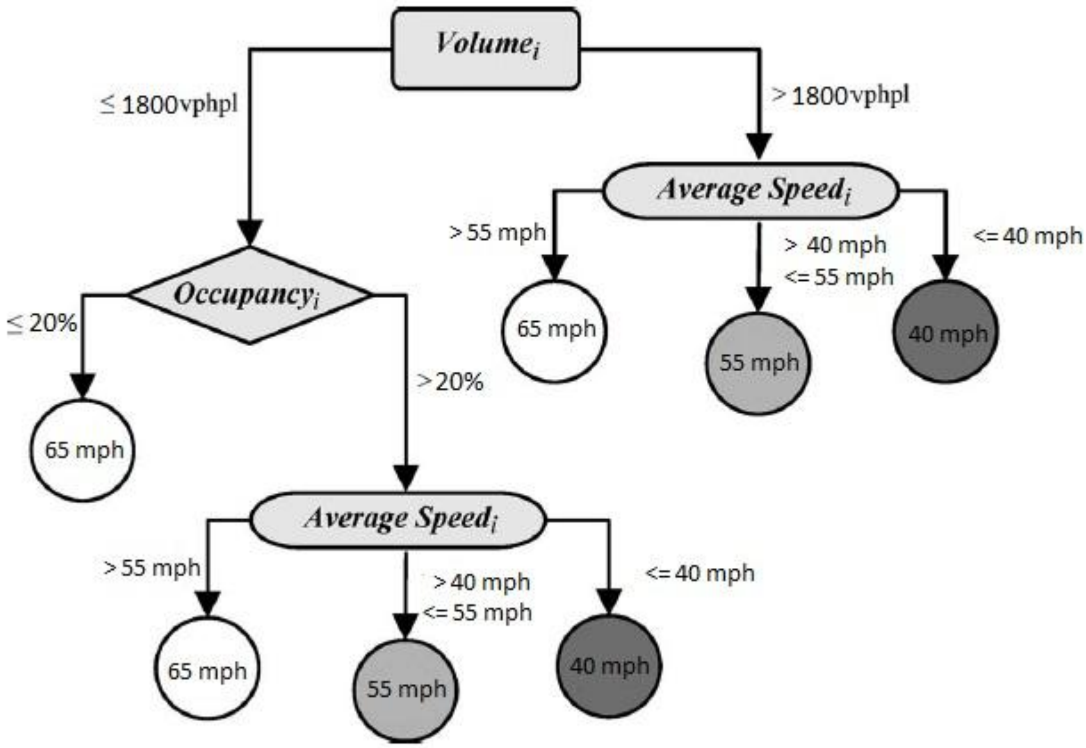


Figure 10: VSL Logic Used in the Study

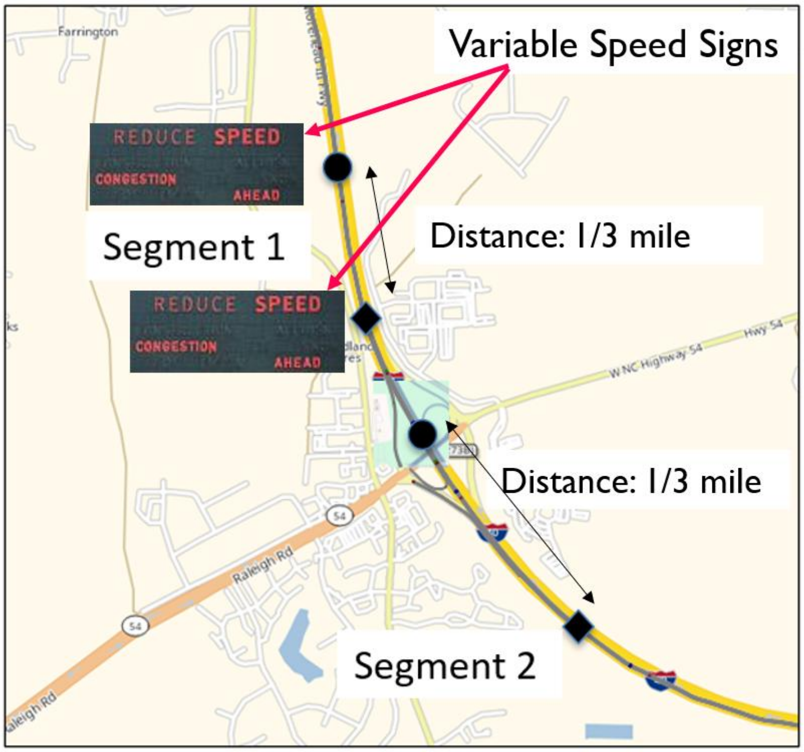
The VSL strategy (Figure 10) relies on real-time measurements of traffic volume, occupancy, and average speed collected at key locations along the managed freeway facility. These measurements are obtained from on-road detectors installed upstream of bottleneck locations.

The control logic follows a hierarchical decision structure that first evaluates traffic demand conditions and then refines speed limit decisions based on congestion indicators.

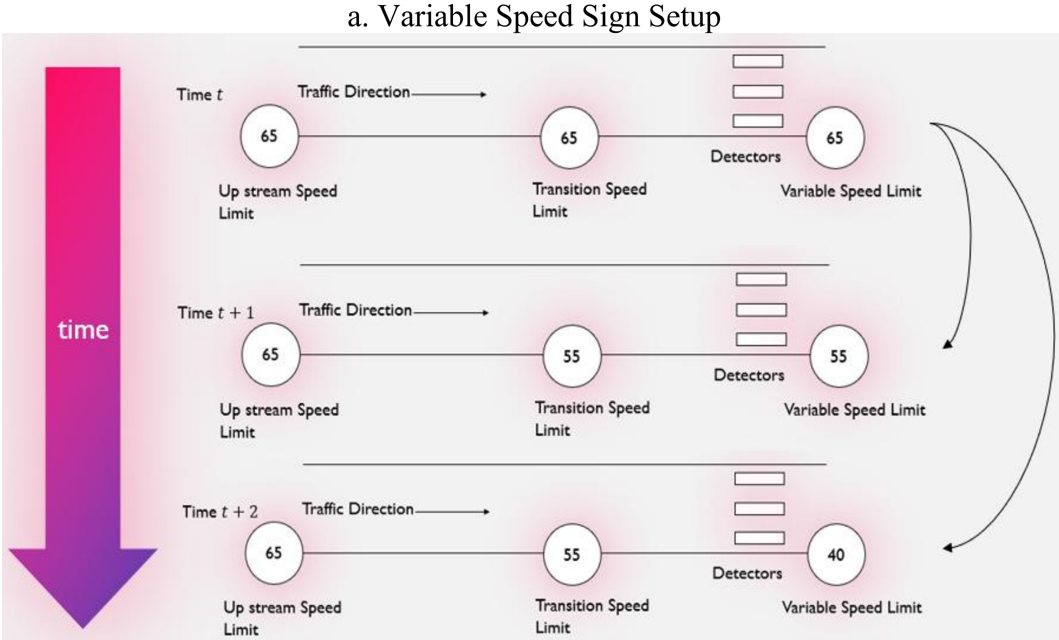
Specifically, traffic volume is used as the primary trigger to distinguish between uncongested and potentially congested regimes. When the observed volume exceeds a predefined threshold (1,800 veh/h/ln), the algorithm further evaluates traffic states using average speed and occupancy to determine the appropriate speed limit.

The VSL decision process can be summarized as follows: For low-volume conditions (≤ 1,800 veh/h/ln): Occupancy is evaluated to assess traffic stability. If occupancy is ≤ 20%, the posted speed limit remains at 65 mph, indicating free-flow conditions. If occupancy exceeds 20%, average speed is evaluated to determine whether a speed reduction is necessary. On the other hand, in high-volume conditions (> 1,800 veh/h/ln): Average speed becomes the primary indicator of congestion severity. If average speed is greater than 55 mph, the speed limit is set to 65 mph. If average speed is between 40 and 55 mph, the speed limit is reduced to 55 mph. If average speed falls below or equal to 40 mph, a further reduction to 40 mph is applied.

To ensure smooth transitions and minimize driver confusion, the VSL system is implemented using two consecutive variable speed limit signs placed upstream of the congestion-prone segment. This staggered deployment (See Figure 11) allows speed reductions to be introduced gradually over distance, reducing the likelihood of sudden breaking and flow instability.



a. Variable Speed Sign Setup



b. Transition VSL Logic

Figure 11: VSL Settings

The VSL control is updated at regular intervals based on detector data, enabling the system to respond dynamically to evolving traffic conditions. The transition speed limit logic (Figure 11) ensures a smooth and gradual reduction of speeds between the upstream free-flow segment and the downstream variable speed limit (VSL) controlled segment. When a speed reduction is triggered based on downstream traffic conditions, the VSL is first lowered at the upstream location in a stepwise speed reduction (e.g., 65 → 55 → 40 mph) over space and time, rather than an abrupt drop at a single location. By acting as an intermediate buffer zone, the transition speed limit reduces speed differentials between adjacent freeway segments, improves driver compliance, and mitigates the formation of shockwaves and capacity drop.

VSL SIMULATION IMPLEMENTATION

VSL strategy was implemented in PTV VISSIM using the attribute modification functionality to dynamically update posted speed limits in response to evolving traffic conditions. The attribute modification functionality allows to adjust desired speed decisions depending on the traffic conditions as measured through detectors. Therefore, speed limit changes follow realistic VSL implementations.

A total of seven key VSL locations were identified along the freeway facility based on observed recurrent speed drops in the 2035 Base scenario (Figure 12). At each location, speed limits were modified in real time according to detector-based measurements of traffic speed, following the predefined VSL logic.

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	15	16	17
03:00-3:15	63	54	13	12	23	63	61	49	58	56	61	63	59	55	55	55
03:15-03:30	63	21	9	12	23	63	50	34	56	55	59	63	45	55	55	55
03:30-03:45	61	13	9	12	23	64	32	29	57	54	60	63	38	55	55	54
03:45-04:00	53	11	9	12	22	63	37	29	55	56	60	63	38	55	55	47
04:00-04:15	29	10	9	12	24	62	22	26	54	55	57	63	35	54	55	49
04:15-04:30	20	10	9	12	24	44	14	24	53	55	58	63	26	54	55	52
04:30-04:45	15	10	9	12	23	28	14	24	55	56	59	53	20	55	55	54
04:45-05:00	12	10	9	12	23	22	14	25	55	56	57	46	18	54	55	54
05:00-05:15	12	10	9	12	23	19	13	23	50	54	55	36	17	55	56	53
05:15-05:30	11	10	9	12	25	16	12	22	49	53	46	26	17	54	55	45
05:30-05:45	11	10	9	12	17	13	11	21	49	55	51	18	17	55	56	36
05:45-06:00	11	10	9	11	9	8	13	23	50	55	53	14	17	54	56	39
06:00-06:15	11	10	8	9	7	8	15	27	56	57	44	12	17	54	55	48
06:15-06:30	9	8	7	9	10	11	17	28	57	57	58	12	17	55	55	54
06:30-06:45	8	7	7	9	13	13	17	26	55	58	62	13	17	55	55	55
06:45-07:00	8	8	8	10	15	13	18	27	57	57	62	14	17	56	55	55

Figure 12: 2035 Base Speed Profile and Key VSL Locations

CARM (+FLUSH) + VSL SCENARIO IMPLEMENTATION

The integrated CARM + VSL strategy was implemented in VISSIM by combining external control through the COM interface for CARM and VISSIM attribute modification for VSL. CARM was coded in Java while VSL uses VISSIM's attribute modification languages.

Overall, this scenario simply combines ramp metering control and variable speed limit control in parallel.

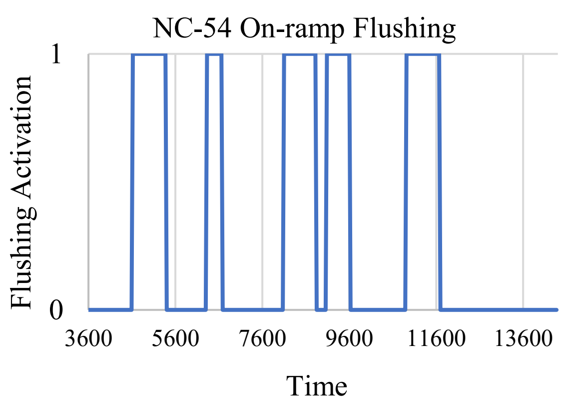
ADDITIONAL RESULTS

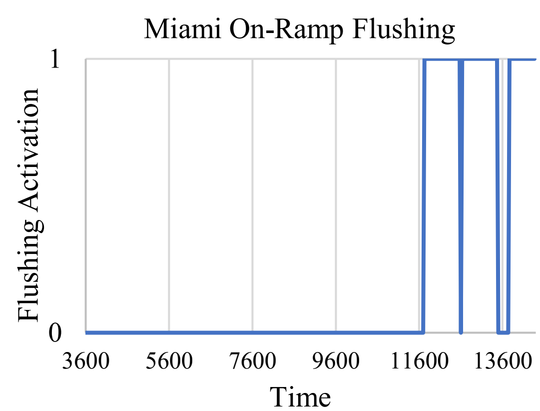
The main report contains some key performance measures, and more information is attached here for all simulation-based scenarios.

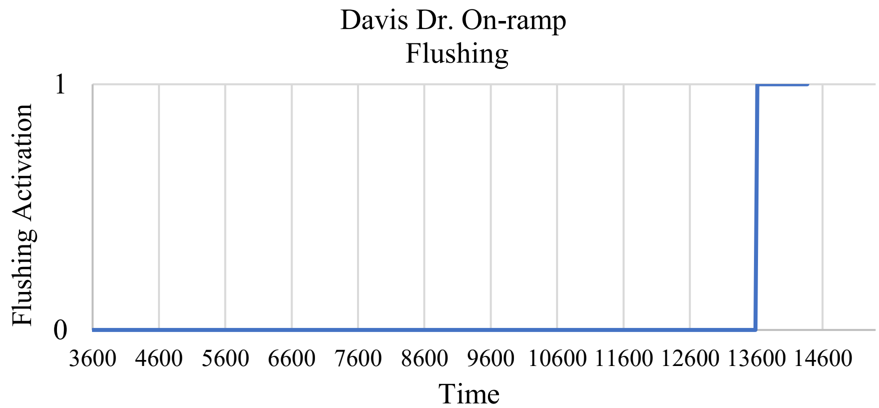
Flushing Activation Overriding CARM Operation

Metered ramps, Flushing activation of metered on-ramps during the peak period are analyzed to assess the effectiveness of CARM. Three of the metered on-ramps with high demand exhibited flushing activation (see **Error! Reference source not found.**). Here, the "1" represents flushing activation and 0 denotes regular CARM operation.

Due to heavy on-ramp demand (i.e., more than 2,000 veh/h flow rate), the NC-54 on-ramp activated flushing multiple times. The other two on-ramps exhibited flushing only during the last hour of congestion (06:00 – 07:00 pm).





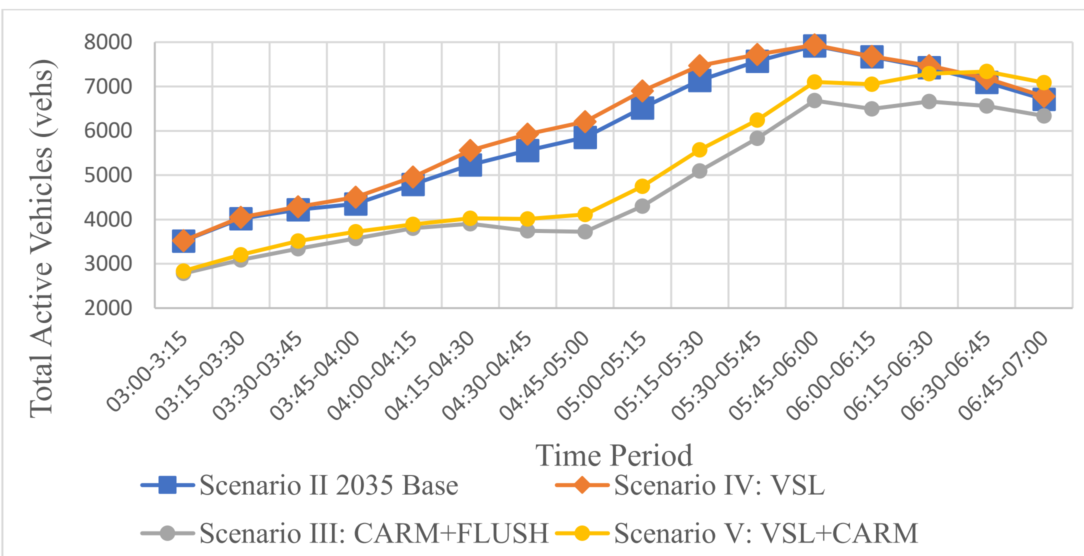


: Flushing Activation of Metered On-Ramps During Peak Period

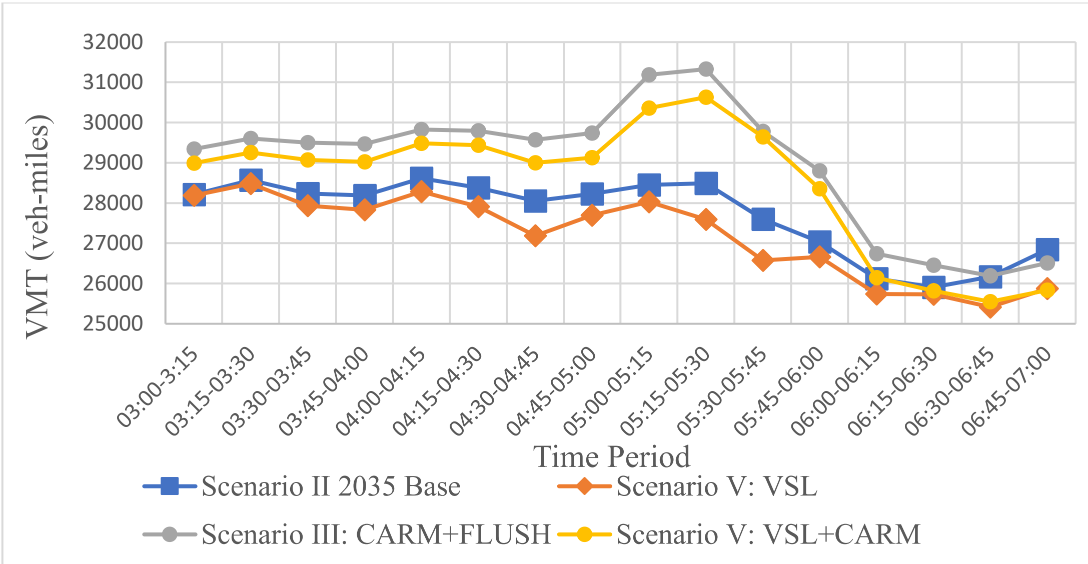
Overall, flushing interruptions occur at only three out of the ten metered on-ramps, and flushing was activated nearly 6% of the simulated peak period. This shows that CARM was, in fact, remained active during most of the peak period.

Facility-wide Performance for Simulation-based Scenarios

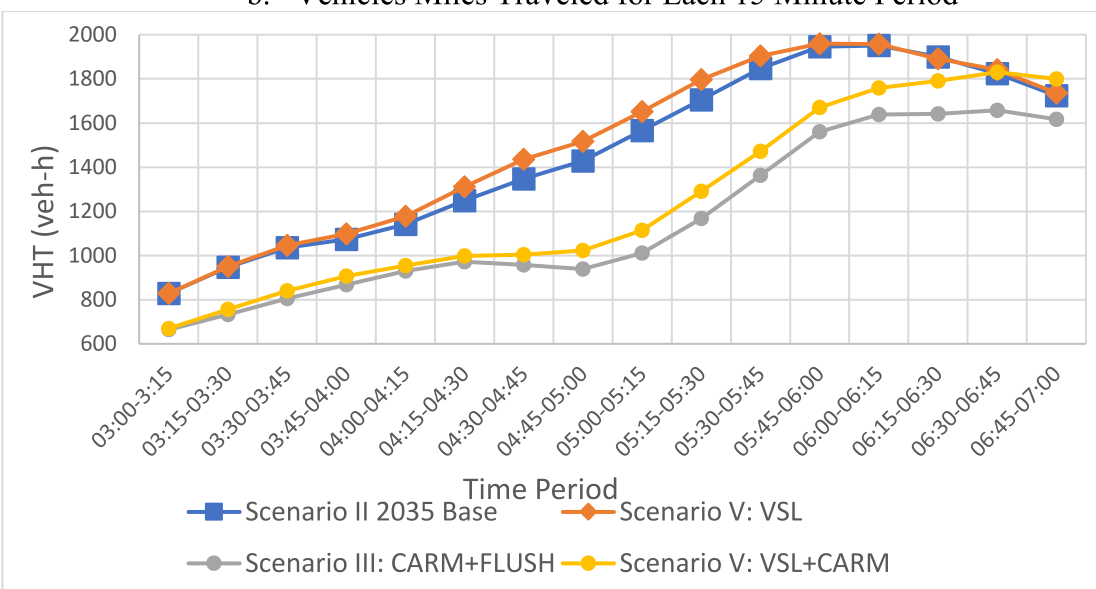
Facility-wide performance measures for each scenario (active vehicles, VMT, VHT, Unmet Demand, total on-ramp throughput) during the peak period are shown in . Also, cumulative speed distribution (c.d.f.) for the scenarios is also shown in Figure 13.



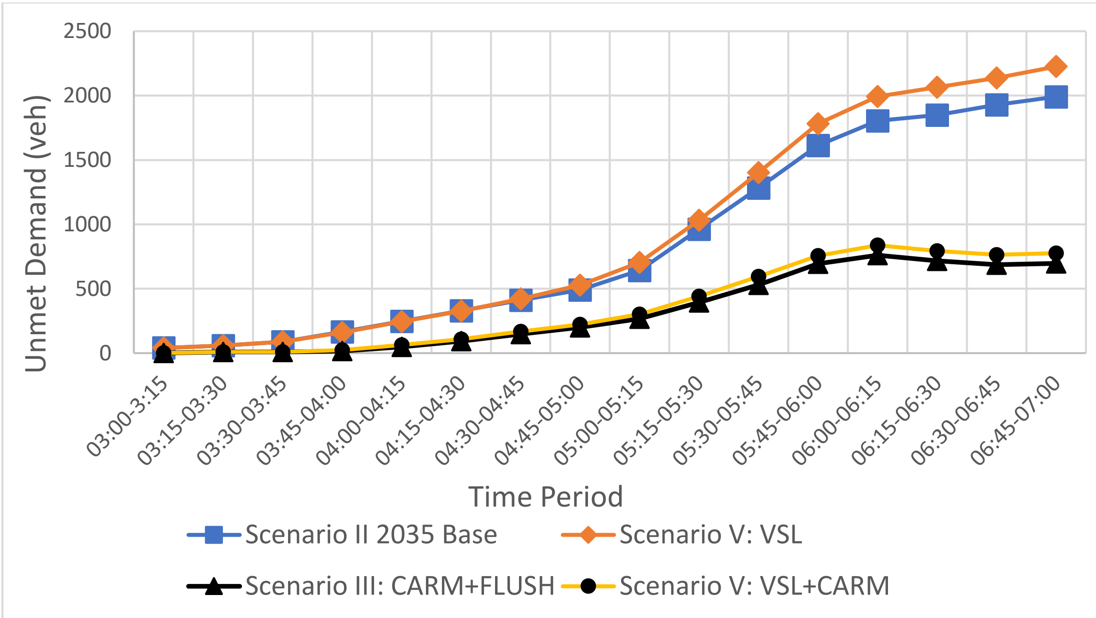
a. Total Number of In-System Vehicles for Each 15 Minute Period



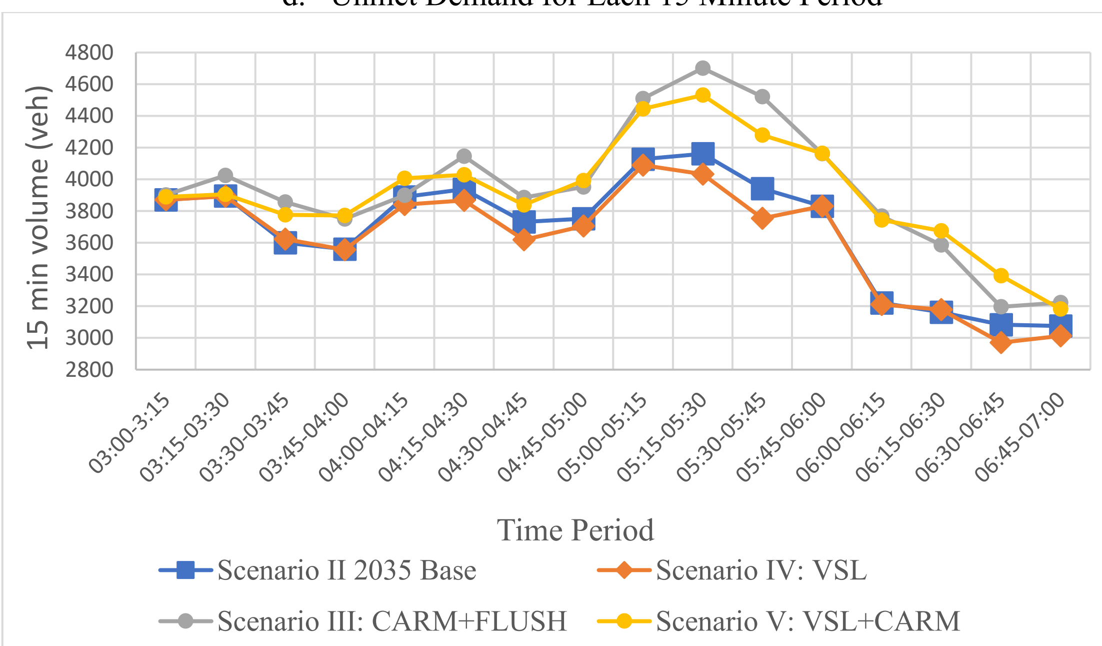
b. Vehicles Miles Traveled for Each 15 Minute Period



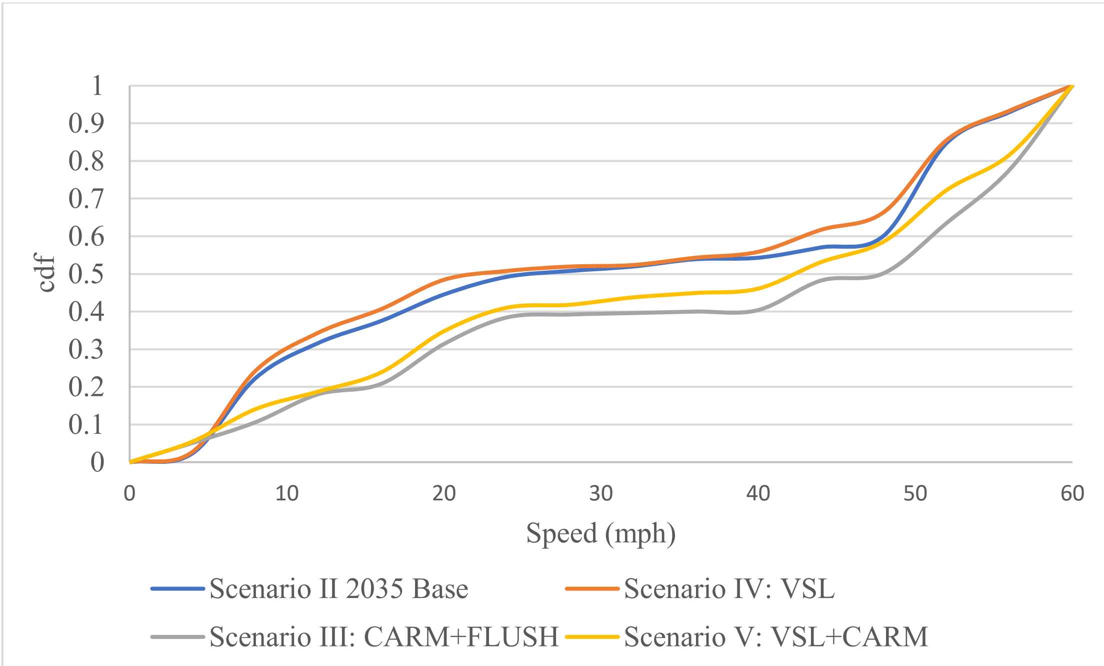
c. Vehicles Hours Traveled for Each 15 Minute Period



d. Unmet Demand for Each 15 Minute Period



e. On-ramp Throughput Volume for Each 15 Minute Period (Total Network)



f. Speed Distribution (cdf)

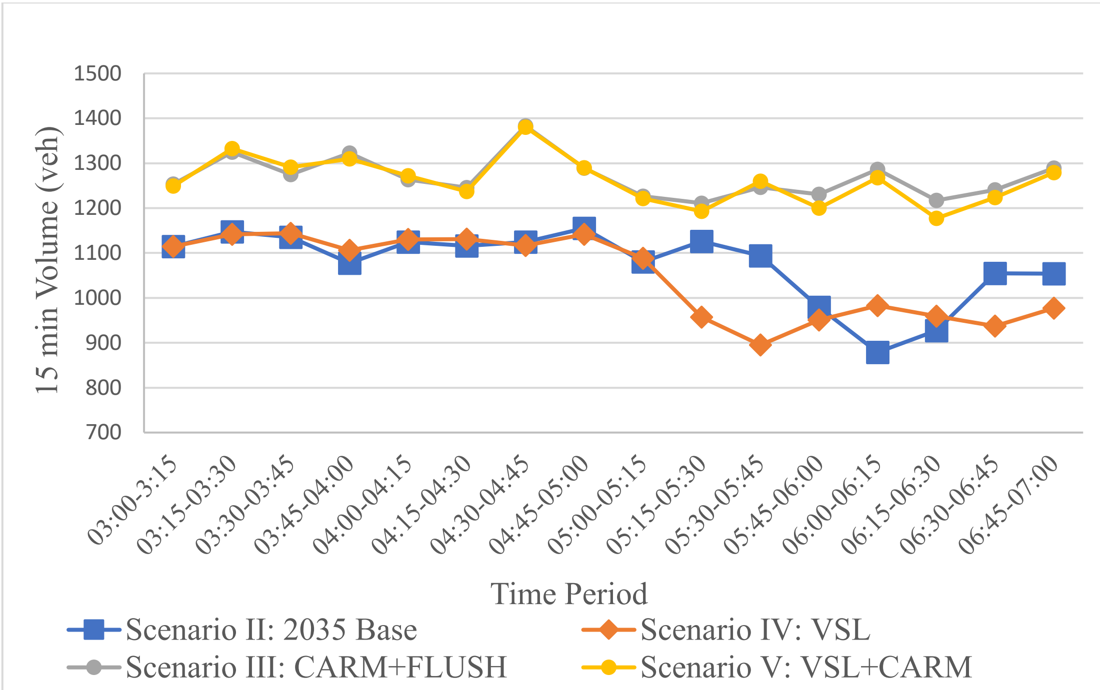
	1	2	3	4	5	6	7	8	9	10	11	12	13	15	16	17
03:00-3:15	63	59	63	24	25	57	19	23	53	55	61	63	63	54	56	46
03:15-03:30	63	59	63	24	27	33	12	21	53	56	61	63	63	51	56	46
03:30-03:45	63	57	63	24	26	21	12	21	53	57	63	63	63	51	56	46
03:45-04:00	63	59	63	22	26	17	12	21	53	57	62	63	63	54	55	46
04:00-04:15	62	58	63	21	27	14	11	21	52	57	61	63	62	54	55	46
04:15-04:30	63	57	63	20	26	11	11	21	52	56	62	63	62	49	55	46
04:30-04:45	63	58	63	21	28	9	11	21	53	58	62	63	62	28	55	46
04:45-05:00	63	56	63	24	26	8	11	22	52	56	61	63	50	21	56	46
05:00-05:15	60	52	62	24	26	8	11	21	51	54	60	60	37	22	55	46
05:15-05:30	54	49	61	19	25	7	10	21	50	52	58	46	33	15	54	46
05:30-05:45	56	48	51	15	26	6	9	20	50	55	58	36	27	10	54	45
05:45-06:00	57	47	27	13	23	6	10	20	50	55	48	30	17	7	55	46
06:00-06:15	62	47	16	13	18	5	10	21	53	57	36	20	12	7	52	45
06:15-06:30	64	30	11	13	17	5	11	22	53	58	35	18	12	7	49	44
06:30-06:45	64	19	10	12	20	5	12	22	53	58	34	17	12	7	48	43
06:45-07:00	64	17	10	13	21	6	12	22	53	58	33	16	12	7	48	43

g. Average Segment Speed for Scenario V: CARM (+ FLUSH) + VSL

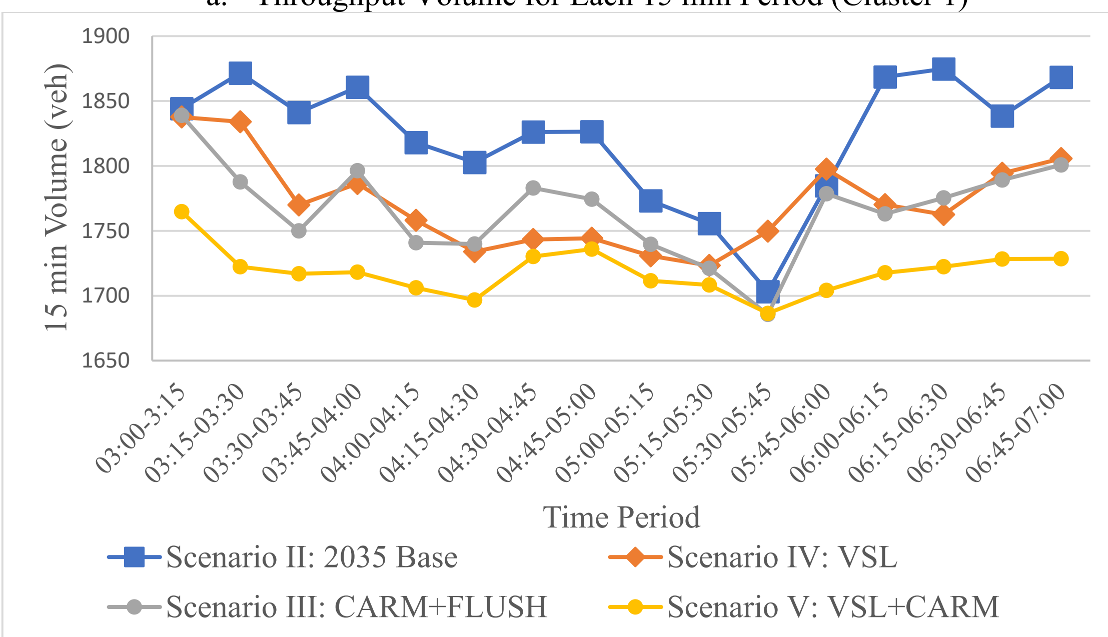
Figure 13: Performance Measures

Cluster-wide Performance

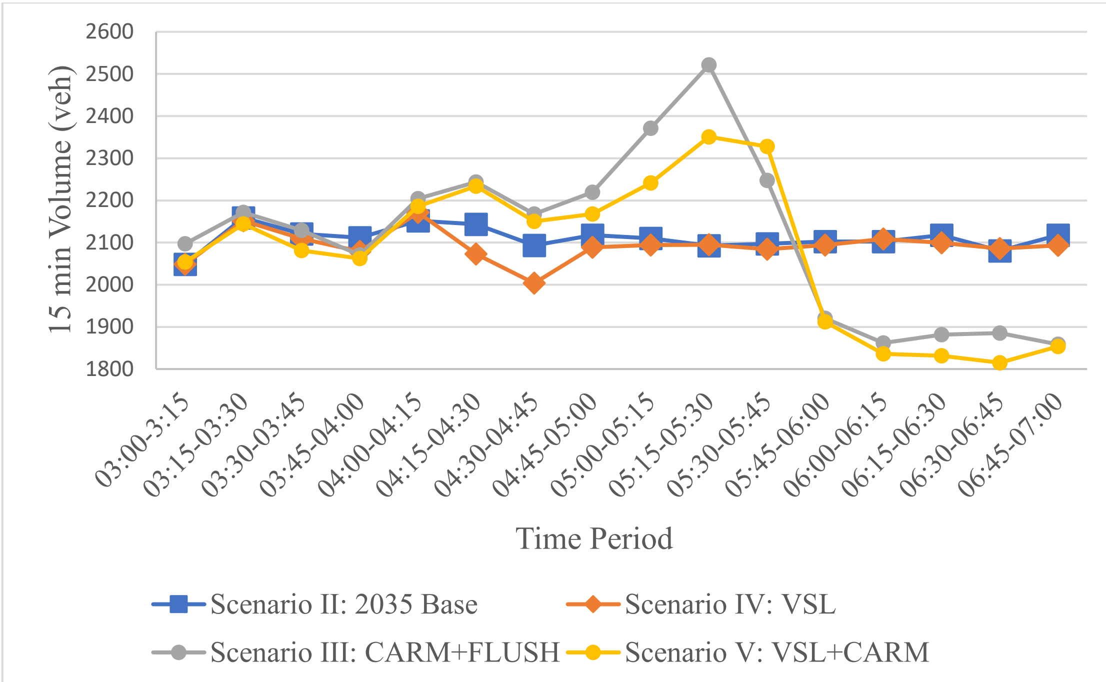
In addition to facility-wide performance, analysis was conducted segment-wise to get better insights into the effects of the technologies.



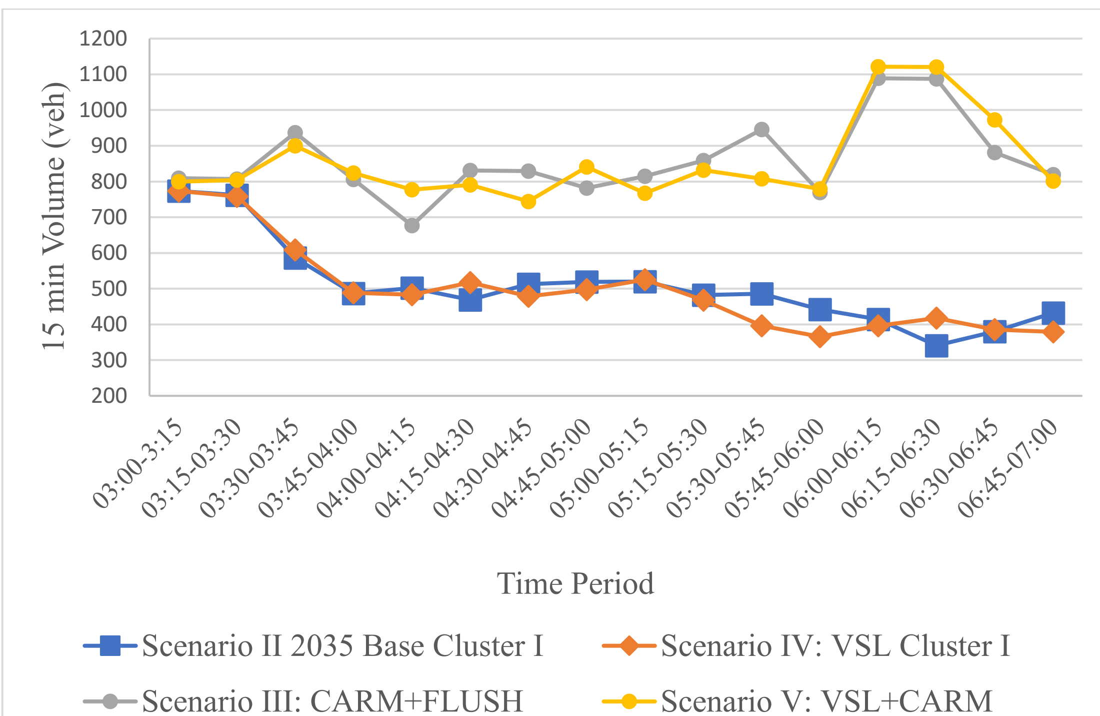
a. Throughput Volume for Each 15 min Period (Cluster 1)



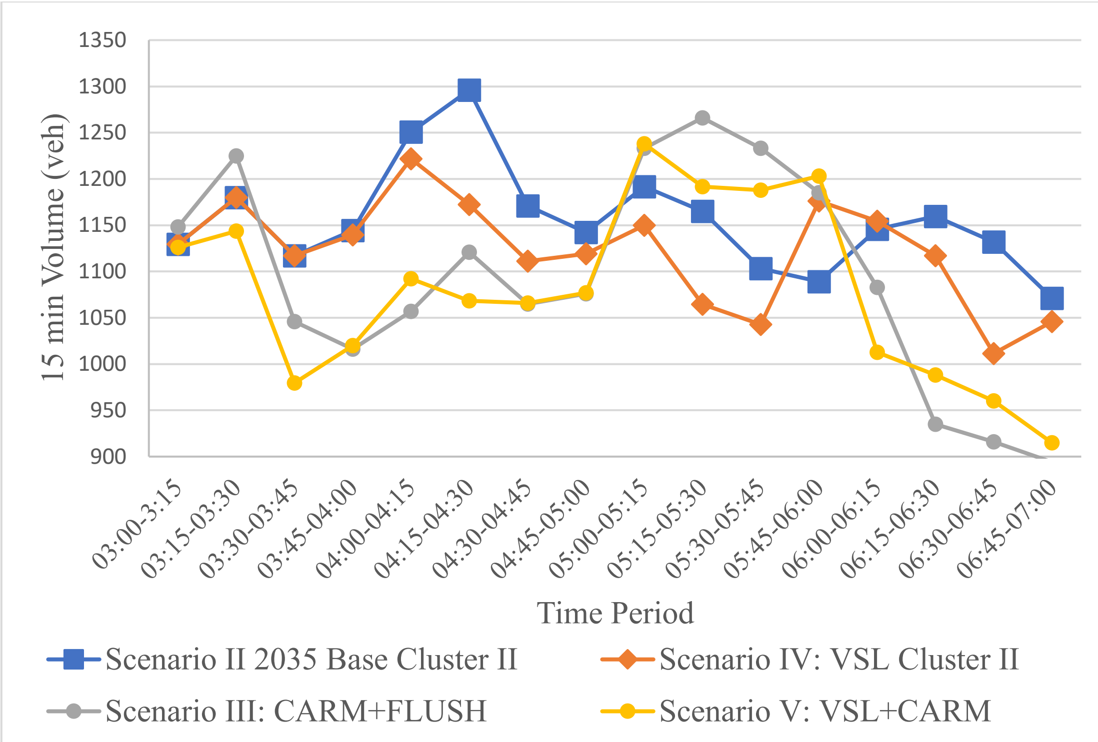
b. Throughput Volume for Each 15 min Period (Cluster 2)



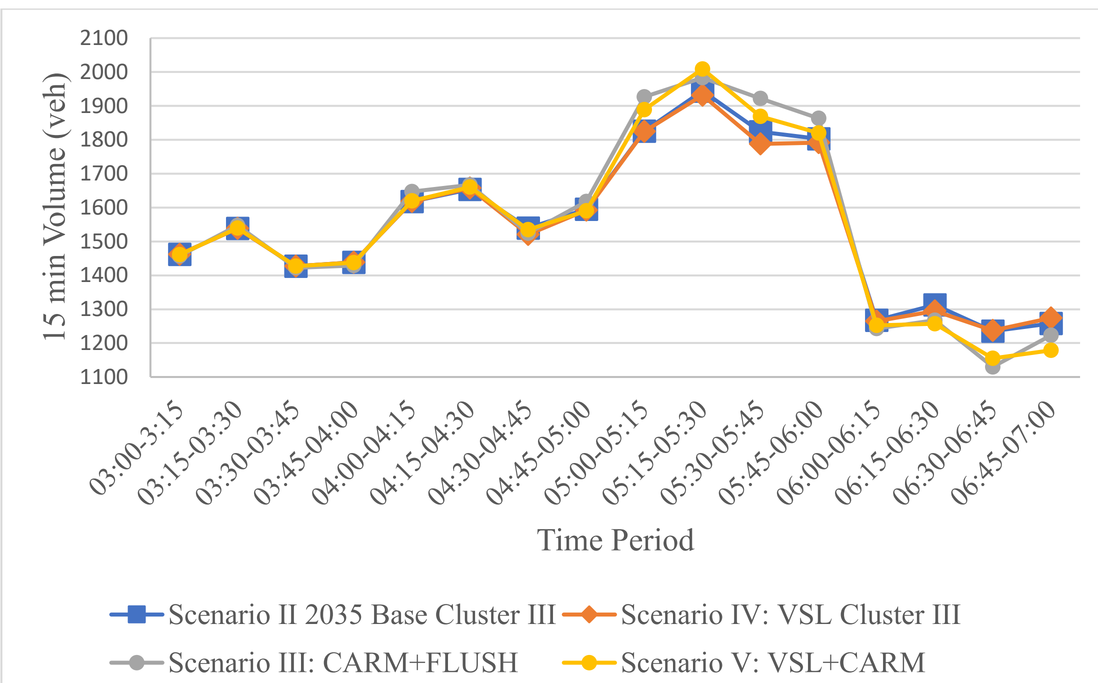
c. Throughput Volume for Each 15 min Period (Cluster 3)



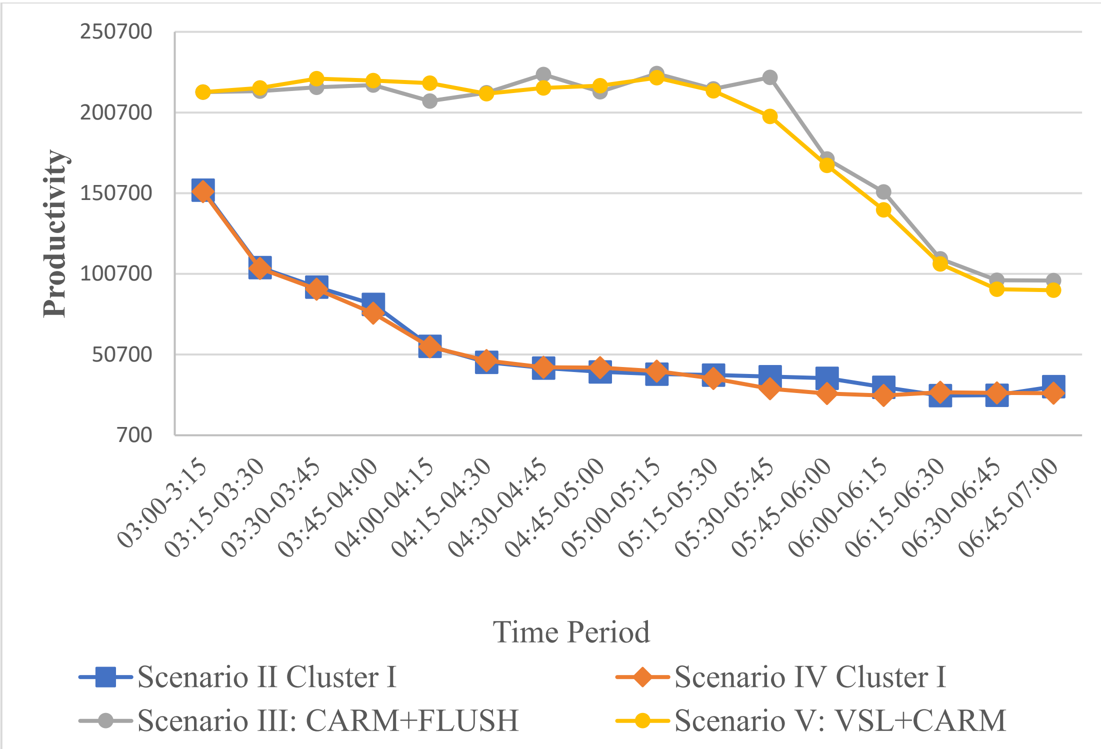
d. On-ramp Throughput Volume for Each 15 min Period (Cluster 1)



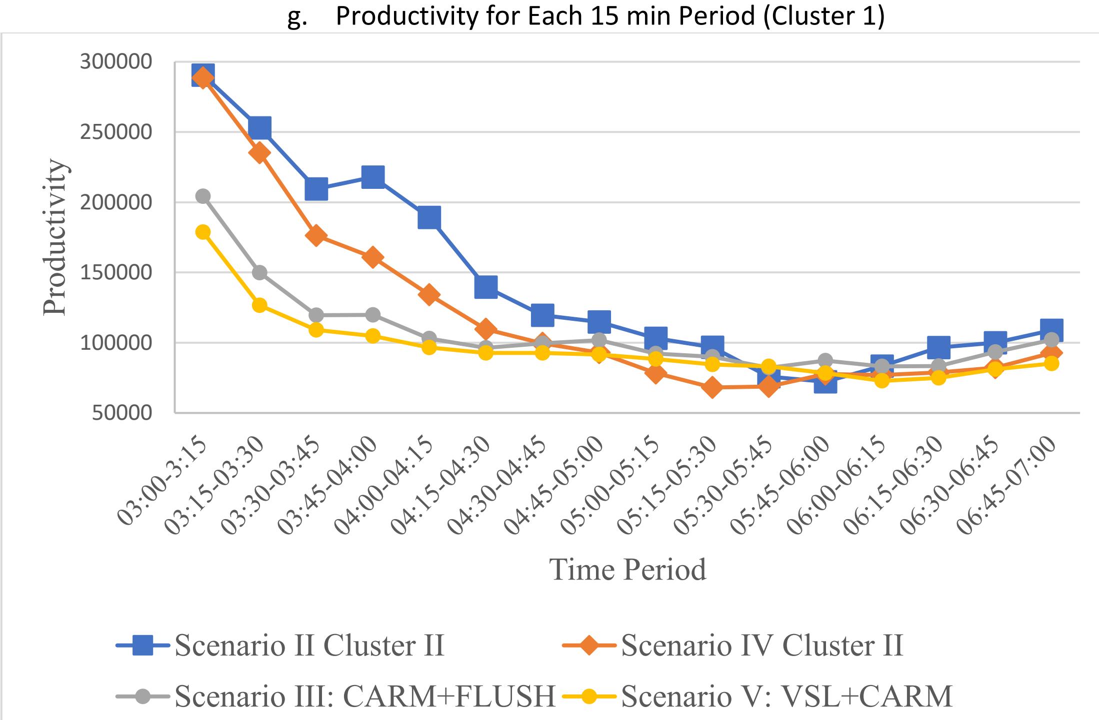
e. On-ramp Throughput Volume for Each 15 min Period (Cluster 2)



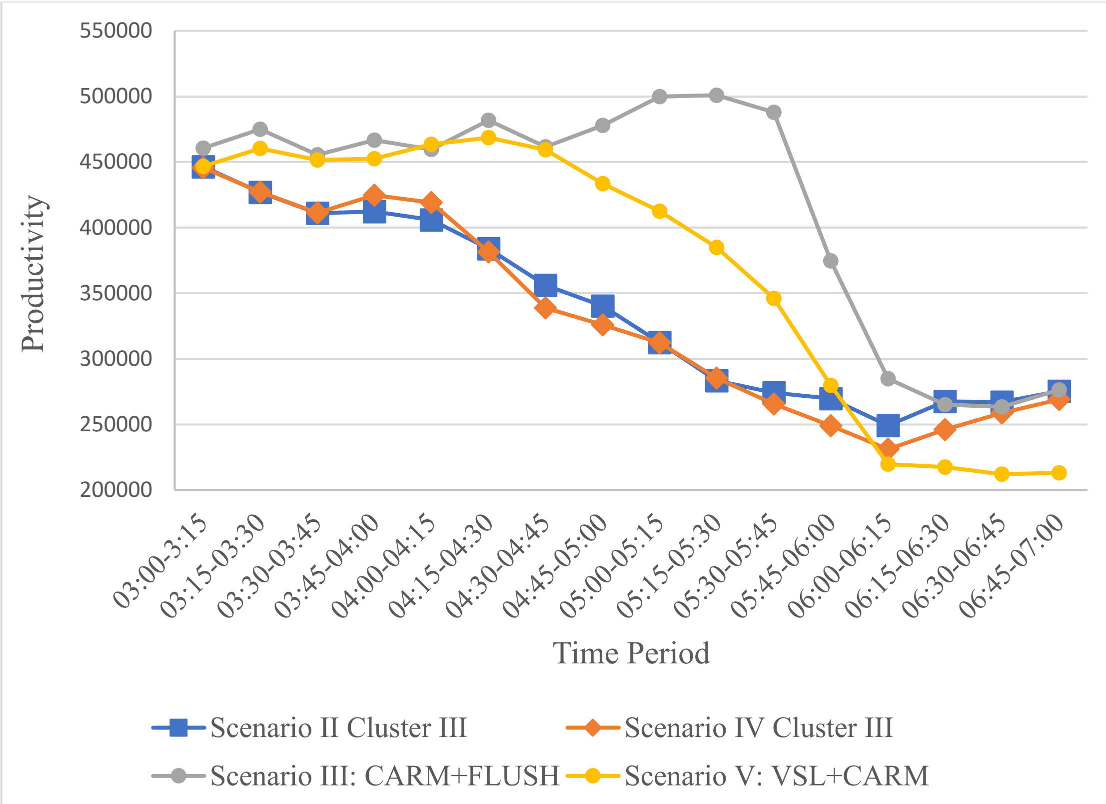
f. On-ramp Throughput Volume for Each 15 min Period (Cluster 3)



g. Productivity for Each 15 min Period (Cluster 1)



h. Productivity for Each 15 min Period (Cluster 2)



i. Productivity for Each 15 min Period (Cluster 3)

Figure 14: Cluster-wide Performance Measures

Appendix D: Benefit-Cost Analysis Methodology and Calculator Documentation

Benefit-Cost Analysis Framework

1. Introduction

The Benefit–Cost Analysis (BCA) framework described herein provides an overview of the methods and data sources used to evaluate the economic efficiency of the proposed managed freeway project. This framework outlines the key components and methodologies used to estimate project costs, mobility benefits, safety benefits, and ultimately the overall benefit–cost ratio (BCR).

Project costs include both capital and ongoing operations and maintenance (O&M) expenses over the analysis period. Mobility benefits are estimated using traffic simulation and operational performance data from literature review to quantify savings in travel time, delay, and network efficiencies. The value of time (VOT) is applied to monetize these travel time savings for different user groups (e.g., passenger vehicles, freight), supported by relevant traffic data inputs such as peak hour traffic volume and Annual Average Daily Traffic (AADT). Safety benefits are derived from expected crash reductions using available AADT, Safety Performance Functions (SPFs), Crash Modification Factors (CMFs), and unit crash costs by severity level. These estimates reflect anticipated improvements in safety performance following project implementation. Finally, the BCR integrates all three components: costs, mobility benefits, and safety benefits, to provide a comprehensive measure of project performance. A BCR greater than 1.0 indicates that the expected benefits exceed the costs, supporting the project's economic justification and prioritization for investment.

This document is organized as follows: Section 2 presents estimations of project capital and O&M costs. This is followed by an assessment of anticipated mobility benefits and safety benefits associated with the proposed strategies, documented in Sections 3 and 4, respectively. Then, Section 5 conducts scenario-based benefit–cost analyses to evaluate the economic performance of the alternatives under varying assumptions. Finally, the I-40 managed freeway case study is presented in Section 6 to demonstrate the application of the proposed framework and to illustrate its practical implications.

2. Estimated Project Costs

The AASHTO Redbook *(AASHTO, 1978)* states "Project costs should include the present value of any obligation to incur costs (or commit to incur costs in the future) that burden the authority's funds." Therefore, under this definition the present value of construction, operating, and maintenance costs over the service life of the project are included in the assessment of expected project costs. AASHTO Redbook provides additional guidance regarding the categories of costs and their proper treatment in a benefit-cost or economic appraisal. Categories discussed in the Redbook include:

- Construction and other development costs
- Adjusting development and operating cost estimates for inflation
- The cost of right-of-way
- Measuring the current and future value of undeveloped land
- Measuring current and future value of developed land
- Valuing already-owned right-of-way
- Maintenance and operating costs
- Creating operating cost estimates

The Redbook also recommends that project costs need to be expressed as present values for use in economic evaluation. Project construction or implementation costs are typically already in present values form, but any annual or future costs need to be converted to present values.

2.1 Capital Costs Estimation

This research adopted the civil costs for I-40 between SR54 and I-440 in Raleigh-Durham area (i.e., NCDOT project I-6006), which was estimated by WSP using the NCDOT Design Estimate Work Sheets *(WSP, 2024)*. The assumption was made that civil costs could be reasonably applied throughout the network, as shown in Tables 1 to 3. Civil costs were converted to a per mile basis for CARM implementation. For Queue Warning and Variable Speed limits technologies, the number of gantries per mile was determined based on those required on I-40 between SR54 and I-440 and was subsequently applied to the entire study corridor.

Table 1 presents the baseline costs for CARM deployment alone, including radar detection, fiber network installation, and a 25% contingency. The total estimated cost is approximately $43.9 million, corresponding to an average of $1.19 million per entrance ramp and $1.10 million per directional mile, with an overall ramp density of 0.93 ramps per mile. Costs vary by corridor, with I-40 segments accounting for the majority of expenditures due to greater mileage and infrastructure needs. Table 2 summarizes the added costs associated with adding queue warning and dynamic speed advisory systems, primarily driven by gantry installations. Across the study corridors, a total of 22 gantries are assumed, resulting in an additional cost of approximately $20.0 million, including contingency. Average costs per directional mile range from approximately $384,000 to $853,000, depending on gantry density, with no additional costs assumed for the Wade Avenue westbound segment. Table 3 lists additional costs for implementing lane use control signals, which require a larger number of additional gantries. A total of 57 additional gantries are assumed, resulting in an estimated $48.1 million in added costs, including contingency.

Table 1. Costs for Coordinated Adaptive Ramp Metering Deployment Alone *(WSP, 2024)*

Conditions		Sum Interchanges and Devices (Radar)	FIBER Network Install	Contingency (25%)	Total Per Segment	Cost Per Entrance Ramp	Cost Per Directional Mile	Ramp Density (Ramp per Mile)
Route	Directional Miles							
I-40 EB	**16.8**	$13,511,000	$4,357,600	$4,467,150	$22,335,750	$1,489,050	$1,332,682	0.89
I-40 WB	**16.6**	$8,344,000	$4,316,000	$3,165,000	$15,825,000	$1,130,357	$953,313	0.84
Wade EB	**3.2**	$1,545,000	$832,000	$594,250	$2,971,250	$742,813	$928,516	1.25
Wade WB	**3.2**	$1,390,000	$832,000	$555,500	$2,777,500	$694,375	$867,969	1.25
Total	**39.8**	**$24,790,000**	**$10,337,600**	**$8,781,900**	**$43,909,500**	**$1,186,743**	**$1,104,364**	**0.93**

Table 2. Additional Costs for Queue Warning and Dynamic Speed Advisory *(WSP, 2024)*

Conditions		Number of Gantries	Total Gantry Cost	Contingency (25%)	Total Per Segment	Cost Per Entrance Ramp	Cost Per Directional Mile	Ramp Density (Ramp per Mile)
Route	Directional Miles							
I-40 EB	**16.8**	12	$8,100,000	$2,820,000	$10,920,000	$728,000	$650,000	0.71
I-40 WB	**16.6**	7	$4,725,000	$1,645,000	$6,370,000	$455,000	$383,735	0.42
Wade EB	**3.2**	3	$2,025,000	$705,000	$2,730,000	$682,500	$853,125	0.94
Wade WB	**3.2**	0	$0	$0	$0	$0	$0	0.00
Total	**39.8**	**22**	**$14,850,000**	**$5,170,000**	**$20,020,000**			**0.55**

Table 3. Additional Costs for Lane Use Control Signals *(WSP, 2024)*

Conditions		Additional Gantries	Total Additional Gantry Cost	Contingency (25%)	Total Per Segment	Cost Per Entrance Ramp	Cost Per Directional Mile	Additional Ramp Density per Mile	Total Ramp Density per Mile)
Route	Directional Miles								
I-40 EB	**16.8**	22	$14,850,000	$3,712,500	$18,562,500	$1,237,500	$1,104,911	1.3	0.71
I-40 WB	**16.6**	26	$17,550,000	$4,387,500	$21,937,500	$1,566,964	$1,321,536	1.6	0.42

Wade EB	**3.2**	3	$2,025,000	$506,250	$2,531,250	$632,813	$791,016	0.9	0.94
Wade WB	**3.2**	6	$4,050,000	$1,012,500	$5,062,500	$1,265,625	$1,582,031	1.9	0.00
Total	**39.8**	**57**	**$38,475,000**	**$9,618,750**	**$48,093,750**			**1.43**	**0.55**

2.2 Operations and Maintenance Costs

The FHWA Operations Benefit/Cost Analysis Tool (TOPS-BC) *(Sallman et al, 2013)* pointed out that compared with more traditional infrastructure improvements, estimating the life-cycle costs of TSM&O strategies (such as ATM) is usually more complex, since TSM&O improvements typically incur a greater proportion of their costs as continuing operations and maintenance costs, as opposed to upfront capital costs. Much of the equipment associated with TSM&O strategies also typically *has a much shorter anticipated useful life* than many traditional improvements, and must be replaced as it reaches obsolescence. With this concern, a B/C analysis must fully consider and account for all the costs of TSM&O strategies to more reasonably assess the applicability of deploying TSM&O improvements.

In addition to the initial capital costs, FHWA recommends the operations costs include the following *(Sallman et al, 2013; Neudorf and McCabe, 2015)*:

- Labor costs tied to the operation of ATM strategies - This can include staffing for TMC operations. To effectively manage the ATM techniques, the TMC will need to be staffed with operators 24 hours a day, 7 days a week. Depending on the size of the ATM system implemented, existing TMC operators' workload might increase, and additional staff might be required. However, the system should be largely automated with operators mainly being responsible for monitoring or possible manual override.
- Updating and maintaining operating procedures - Besides the additional labor costs associated with operations, there are other costs tied to operation of ATM strategies. These include updating standard operating procedures and the system rules based on operating experience.
- Replacement costs - The periodic cost of replacing and/or redeploying system equipment as it becomes obsolete and reaches the end of its expected useful life to ensure the continued system operation.

The research team obtained the annual O&M cost estimates from the NCDOT Transportation Mobility and Safety Division. Along with the capital costs extracted from Tables 1-3, the O&M costs are shown in Table 4. In practice, instead of applying monetized values directly, a rule-of-thumb is to approximate operating costs as a percentage of capital costs. The combined capital cost for the three ATM strategies is estimated at approximately $112 million with an annual O&M cost of approximately $2.7 million, representing 2.4 percent of the capital cost. Among the strategies, Lane Use Control Signals exhibit the highest O&M cost intensity, with annual O&M costs account for 3.12 percent of the capital cost. The Queue Warning and Dynamic Speed Advisory and Coordinated Adaptive Ramp Metering strategies demonstrate comparatively lower O&M ratios of 2.8 percent and 1.4 percent, respectively.

Table 4. Estimated Annual O&M Costs for the Study Corridor

ATM Strategy	**Capital Cost**	**Annual O&M Cost**	**O&M Cost as a Percentage of Capital Cost**
Coordinated Adaptive Ramp Metering	$43,909,500	$625,000	**1.4%**
Queue Warning and Dynamic Speed Advisory	$20,020,000	$560,000	**2.8%**
Lane Use Control Signals	$48,093,750	$1,500,000	**3.12%**

Total	**$112,023,250**	**$2,685,000**	**2.4%**

2.3 Converting Annual Monetary Value to Present Value

The following data are needed to convert annual monetary value to present value:

- Annual monetary benefits associated with the operational (i.e., delay savings) and safety (i.e., crash reductions);
- Service life of the countermeasure(s); and,
- Discount rate (minimum rate of return).

When the annual benefits are uniform over the service life of the project, the present value of project benefits can be calculated using the Equation below.

$$PVB_v = [TotalAnnualMonetaryBenefits \times (P/A, i, y)]^x$$

Where,

PVB_v = Present value of the project benefits for a specific site, v

x = Average annual inflation rate at which prices increase over time (a 5% inflation rate was adopted by this research, or $x = 0.05$)

$(P/A, i, y)$ = Conversion factor for a series of uniform annual amounts to present value

$$(P/A, i, y) = \frac{(1.0 + i)^y - 1.0}{i \times (1.0 + i)^y}$$

Where,

i = Minimum attractive rate of return or discount rate (i.e., if the discount rate is 2%, the $i = 0.02$)

y = Number of years in the service life of the countermeasure(s) (y = 10)

3. Estimated Mobility Benefits

3.1 Peak Hour Travel Time Savings

Table 5 presents the network performance metrics derived from VISSIM microsimulation modeling conducted for the Year 2035 baseline (no treatments) and three alternative operational improvement scenarios: CARM, VSL, and VSL + CARM. The results, obtained from model outputs, include Vehicle Hours of Delay (VHD), Vehicle Miles Traveled (VMT), Vehicle Hours Traveled (VHT), and Throughput, which collectively reflect the efficiency and capacity of the transportation network under each scenario. Overall, the CARM and VSL+CARM scenarios demonstrate substantial reductions in delay (−38.0% and −29.0%, respectively) and notable increases in throughput, indicating enhanced traffic operations relative to the baseline. In contrast, the VSL-only scenario yields limited performance improvements. Vehicle Hours Traveled (VHT) was the selected performance metric for mobility benefits.

Table 5. Freeway-wide Operational Performance in representative PM peak period (3-7 PM)

Scenario	VHD (veh-hr)	VMT (veh-miles)	VHT (veh-hr)	Throughput (veh)	Productivity
I: 2022 Existing Conditions	2,616	396,509	8,759	65,985	15680675
II: 2035 Base	30,522	443,097	23,515	73,145	10466384
III: 2035 CARM	18,926 (-38.0%)	463,946 (+4.7%)	18,521 (-21.2%)	77,998 (+6.6%)	13091966 (+25%)
IV: 2035 VSL	31,605 (+3.6%)	435,130 (-1.8%)	24,097 (+2.5%)	72,123 (-1.4%)	9920053 (-5.2%)
V: 2035 VSL + CARM	21,672 (-29.0%)	455,734 (+2.9%)	19,877 (-15.5%)	76,976 (+5.2%)	11976202 (+14.4%)

3.2 Extrapolate PM Peak Period Benefits to Annual and Bi-directional Facility

To ensure that the estimated project benefits represent system wide impacts, the 4-hr peak-period results are extrapolated to future year daily, annual and bi-directional conditions. The initial benefit estimates, typically derived from traffic simulation or performance data during the peak analysis period in one direction, are adjusted to account for both directions of travel and for all weekdays of the year. This extrapolation process incorporates factors such as the proportion of daily traffic represented by the peak hours (K-factor), directional traffic splits (D-factor), and the number of effective analysis days per year (N-factor). By applying these adjustments, the analysis provides a more comprehensive representation of the project's total annual mobility and safety benefits across the full facility.

K-factor - Percentage of I-40 Eastbound corridor, day-time traffic volume (6 am to 8 pm) which occurs during the simulated PM peak 4-hours (K=0.338 for the study freeway corridor) based on available 2035 hourly demand volumes

D-factor - Percentage of I-40 Eastbound corridor, day-time traffic volume (6 am to 8 pm) of bi-directional traffic flow traveling on the corridor (D = 0.491 for the study freeway corridor)

N-factor – Number of weekdays per year benefits occur (N = 250 days/year)

O-factor – Weighted average vehicle occupancy per trip (O = 1.5 occupants/vehicle; per NCDOT guideline)

EB PM Peak Period VHT Savings = VHT Base Scenario (2035) – VHT Treatment Scenario (2035)

Corridor Annual Benefits = (Simulated Eastbound Peak Period VHT Savings × N × O) / (K × D)

3.3 Value of Time in North Carolina

The input values of time align with current NCDOT benefit-cost analysis guidance and ensure consistency in evaluating the economic impacts of proposed transportation investments *(NCDOT, 2025)*. As shown in Table 6, vehicle occupancy rates are assumed to be 1.52 persons per vehicle for passenger travel and 1.00 person per vehicle for commercial trucks. The value of time is estimated at $21.10 per hour for passenger vehicles and $35.70 per hour for trucks. Operating costs are set at $0.56 per mile for light-duty vehicles and $1.27 per mile for commercial trucks. All monetary values are expressed in 2023 dollars.

Table 6. Value of Time for Benefit-Cost Analysis

Input	Value
General Assumptions	
Analysis Period (Years)- Projects Involving Full Reconstruction of Highways or New Location Roadways	30
Discount Rate	3.1%
Discount Rate for Reductions in CO_2 Emissions	2%
Dollar Year	2023
Auto Occupancy (Passenger Vehicles, All Travel)	1.52
Auto Occupancy (Trucks) [1]	1.00
Truck Value of Time (Hourly Value)	$35.70
Passenger Vehicle Value of Time (Hourly Value)	$21.10
Operating Costs per Mile (Light Duty Vehicles)	$0.56
Operating Costs per Mile (Commercial Trucks)	$1.27

4. Estimated Safety Benefits

4.1 Predicting Baseline Number of Crashes

The baseline number of crashes for the analysis is estimated using the Safety Performance Function (SPF) developed specifically for roadway facilities in North Carolina *(Srinivasan and Carter, 2011)*. This SPF provides a predictive relationship between crash frequency and key roadway and traffic characteristics such as segment length, traffic volume, and facility type, depicted as follows:

$$Y = L \times (Annual\ Factor) \times exp\{\alpha + \beta \times \ln AADT\} = L \times (Annual\ Factor) \times e^{\alpha} \times AADT^{\beta}$$

Where,

- Y is the predicted number of crashes per year for a segment with length L (in miles)
- α is the intercept; $\alpha = -15.3488$ for KABC crashes, and $\alpha = -17.0107$ for PDO crash for the study freeway corridor
- β is the coefficient for $\ln AADT$; $\beta = 1.5478$ for KABC crashes, and $\beta = 1.7596$ for PDO crash for the study freeway corridor
- $Annual\ Factor$ is a multiplier applied to the SPF-predicted number of crashes to align the model's predictions with observed crash data for a specific year; $Annual\ Factor = 1.06$ for the study freeway corridor *(Saleem et al., 2021)*.

The baseline SPF serves as the foundation for estimating expected crash occurrences under existing conditions, prior to the implementation of any proposed improvements. It is important to note that the SPFs are developed separately for two crash categories: Fatal and Injury (KABC) crashes, and Property Damage Only (PDO) crashes. Accordingly, the total number of baseline crashes are calculated as the sum of crashes predicated from the two SPFs within the study freeway segment.

4.2. AADT Estimation

In crash prediction modeling, AADT serves as the principal exposure variable within SPFs, quantifying the relationship between traffic volume and expected crash frequency. It reflects the level of vehicular activity influencing the likelihood of crash occurrence across facility types. It represents the estimated average number of vehicles traveling on a road segment per day, averaged across all days of the week over an entire year.

The baseline (i.e., Year 2022) AADTs for each interchange are extracted from the NCDOT AADT map; a screenshot is shown in Figure 1. For each interchange, AADTs of the arterial, on-ramp, and freeway mainline are summarized in Table 7.

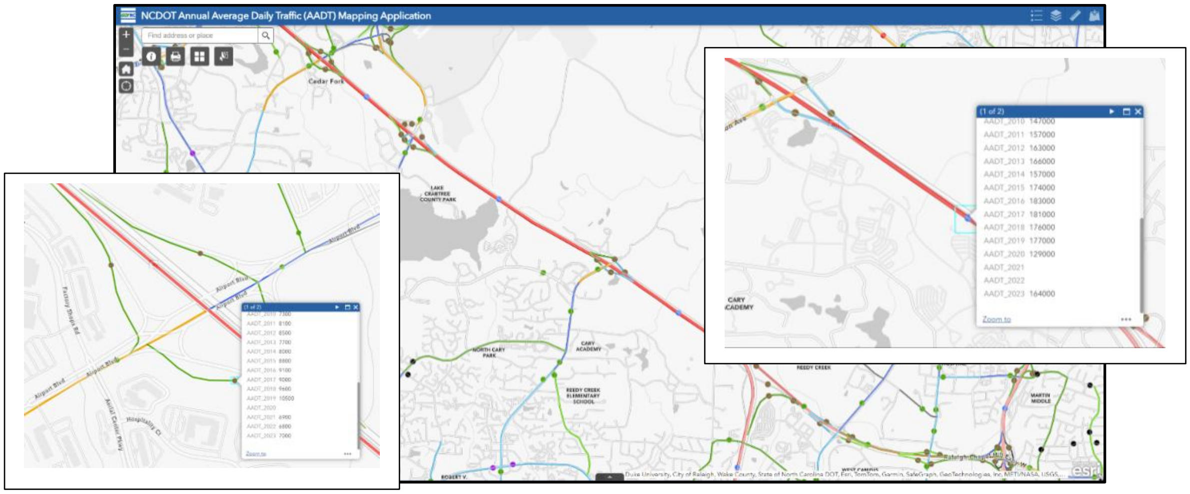


Figure 1. NCDOT AADT Map

Source: https://www.arcgis.com/apps/webappviewer/index.html?id=964881960f0549de8c3583bf46ef5ed4

Table 7. System AADT Under Baseline Condition (Year 2022)

Interchange	AADT (veh)			Segment Length (mile)
	Arterial	On-Ramp	Freeway Mainline Segment	
SR54 (Raleigh Rd)	43,000	EB: 18,000	123,000	1
NC751	16,000	EB: 5,200	127,000	2
Fayetteville Rd	30,000	EB: 11,000	127,000	2
NC55	32,000	EB: 9,300	135,000	1
Durham FWY	67,500	EB: 27,500	91,000	1
Davis Dr.	24,000	EB: 6,300	173,000	1
Miami Blvd	18,000	EB: 8,700	184,000	1
Page Rd.	16,500	EB: 7,200	188,000	1
State Hwy 540	42,500	NB: 7,600; SB: 7,500	152,000	1
Airport Blvd	34,500	EB: 7,000	148,000	1
Aviation Pkwy	34,000	EB: 6,800	169,000	2
Harrison Ave	37,000	EB: 11,000	164,000	2
Weighted Average Facility AADT			**147,125**	

The Growth Factor Method is employed for predicting future year AADT:

$$AADT_{Future} = AADT_{Baseline} \times (1 + r)^n$$

Where,

$AADT_{Baseline}$: known AADT in the base year (Year 2022)

- r: annual growth rate (e.g., 1.5%)

- n: number of years into the future (13 years in this case)

Estimated average freeway mainline AADT for Year 2035:

$$AADT_{2035} = AADT_{2022} \times (1 + 0.015)^{13} = 147125 \times 1.015^{13} = 178{,}500$$

4.3 Crash Modification Factors (CMFs) for Selected ATM Strategies

This section summarizes the Crash Modification Factors (CMFs) used to evaluate the safety impacts of the selected freeway ATM strategies, including Coordinated Adaptive Ramp Metering (CARM), Variable Speed Limits (VSL), Queue Warning (QW), and Lane Use Control Signals (LUCS). CMFs represent the expected change in crash frequency associated with the implementation of a specific treatment and are critical inputs for quantifying safety benefits in project evaluation and decision-making. The selected CMFs, as listed in Table 8, are based on peer-reviewed literature and authoritative agency reports that specific to safety performance outcomes on urban freeways. Where multiple CMFs were reported for a treatment, the values most appropriate for fatal and injury crashes (i.e., K, A, B, and C) and PDO were chosen to ensure consistency and conservativeness in the safety analysis.

Table 8. Summary of CMFs for Various ATM Treatments (Extracted from Literature Review)

Treatment	Crash Type	Crash Severity	CMF	Study Region / Source
Coordinated Ramp Metering	All	All	1.07	I-80 in the San Francisco Bay Area, California. *(Molan et al., 2023)*
		K,A,B,C	0.95	
		PDO	1.12	
Install Variable Speed Limit (VSL)	All	All	0.708	A 30.6-mi corridor in GA. *(Avelar et al., 2021)* * I-5 in the greater Seattle region, Washington State. *(Chakraborty and Mahmud, 2024)*
		K,A,B,C	0.89	
		PDO	0.918*	
	Rear-end	All	0.648	
		K,A,B,C	0.82	
Install changeable "Queue Ahead" warning signs	Rear-end	K,A,B,C	0.84	*Elvik and Vaa, 2004*
		PDO	1.16	
	All	K,A,B,C	0.56	
Implementation of LUCS	All	All	0.96	I-66, VA. *(Dutta et al., 2017)*
		K,A,B,C	0.94	

CMF for Coordinated Adaptive Ramp Metering

The CMF for CARM was derived from a before-after safety evaluation using the Empirical Bayes method, conducted by *Molan et al. (2023)* on I-80 in California. As shown in Table 8, a CMF of 0.95 was reported for fatal and injury crashes (K, A, B, and C), reflecting a 5% reduction in crash frequency following CARM implementation. In addition, a CMF of 1.12 was reported for property damage only (PDO) crashes, indicating that the implementation of CARM increased PDO crashes by 12%.

CMF for Variable Speed Limit

Avelar et al. (2021) conducted a safety evaluation of VSL systems on a 30.6-mile corridor in Georgia. The study reported a CMF of 0.89 for fatal and injury crashes, indicating an 11% reduction after VSL deployment. However, this study did not specify a CMF for PDO crashes. Alternatively, we referenced another study conducted on I-5 in the greater Seattle region of Washington State, which reported a PDO crash CMF of 0.918, indicating an 8.2% reduction in PDO crashes.

CMF for Queue Warning

Based on findings from *Elvik and Vaa (2004)*, the installation of changeable "Queue Ahead" warning signs was associated with a CMF of 0.84 for rear-end crashes resulting in fatalities or injuries. This indicates a 16% reduction in crash occurrence under queuing conditions. Although more significant reductions were noted for all injury crashes (CMF = 0.56), given that the primary objective of a queue warning system is to mitigate rear-end crash risk, the 0.84 value was selected as a conservative and focused estimate for injury-related rear-end crash reduction. With respect to PDO crashes, an 16% increase in rear-end PDO crashes was reported, indicating a CMF of 1.16.

CMF for Lane Use Control Signals

Dutta et al. (2017) evaluated the impact of diagonal yellow arrow LUCS on I-66 in Virginia and reported a CMF of 0.94 for fatal and injury crashes. This equates to a 6% reduction in crash frequency following LUCS implementation. However, this study did not report a CMF specific to PDO crashes, and a review of the existing literature did not identify any studies that provided PDO-specific CMFs for this treatment. In the absence of such information, the overall crash CMF of 0.96 was adopted as a proxy measure to reflect the general safety benefits associated with improved lane guidance.

4.4 Predicted Annual Number of Crashes with Active Traffic Management Systems

The general form of the predictive model used to determine the predicted average crash frequency, $N_{predicted}$, is shown below:

$$N_{predicted} = N_{SPFx} \times (CMF_{1x} \times CMF_{1x} \times \ldots \times CMF_{yx}) \times C_x$$

Where,

$N_{predicted}$ = predicted average crash frequency for a specific year for site type x

N_{SPFx} = predicted average crash frequency determined under base conditions (no technology) of the SPF developed for site type x

CMF_{yx} = Crash Modification Factors specific to SPF for site type x

C_x = calibration factor to adjust SPF for local conditions for site type x

$$Combined\ CMF = CMF_{CARM} \times CMF_{VSL} \times CMF_{QW} \times CMF_{LUCS}$$

For fatal and injury (KABC) crashes, the combined CMF is calculated as follows:

$$CMF^{KABC} = CMF_{CARM}^{KABC} \times CMF_{VSL}^{KABC} \times CMF_{QW}^{KABC} \times CMF_{LUCS}^{KABC}$$

$$= 0.95 \times 0.89 \times 0.84 \times 0.94 = 0.668$$

For property damage only (PDO) crashes, the combined CMF is calculated as follows:

$$CMF^{PDO} = CMF_{CARM}^{PDO} \times CMF_{VSL}^{PDO} \times CMF_{QW}^{PDO} \times CMF_{LUCS}^{PDO}$$

$$= 1.12 \times 0.918 \times 1.16 \times 0.96 = 1.145$$

So, the predicted average crash frequency for the tested managed freeway scenario is:

$$N_{predicted} = N_{spfx}^{KABC} \times CMF^{KABC} \times C_x^{KABC} + N_{spfx}^{PDO} \times CMF^{PDO} \times C_x^{PDO}$$
$$= N_{spfx}^{KABC} \times 0.668 \times C_x^{KABC} + N_{spfx}^{PDO} \times 1.145 \times C_x^{PDO}$$

An example of calculating the predicated future year total number of crashes using the baseline SPFs is presented below:

$$N_{predicted} = N_{spfx}^{KABC} \times 0.668 \times C_x^{KABC} + N_{spfx}^{PDO} \times 1.145 \times C_x^{PDO}$$
$$= (39.8 \times 1 \times \exp\{-15.3488 + 1.5478 \times ln178{,}500\}) \times 0.668 \times 1$$
$$+ (39.8 \times 1 \times \exp\{-17.0107 + 1.7596 \times ln178{,}500\}) \times 1.145 \times 1$$
$$= 1155 \times 0.668 \times 1 + 2839 \times 1.145 \times 1$$

$$= 771 + 3251 = 4022$$

4.5 Standardized Crash Cost Estimates in North Carolina

The NCDOT Transportation Mobility and Safety Division adopts the recommendation of FHWA's Crash Costs for Highway Safety Analysis *(Harmon et al., 2018)*. The comprehensive crash costs presented in this guide are based on the values provided in the FHWA report, as shown in Table 9.

Table 9. Cost per crash in North Carolina (Retrieved from *NCDOT, 2024*)

Crash Type	Cost Per Crash (2024 Dollars)
Fatal Crash	$12,894,000
A Injury Crash	$760,000
B Injury Crash	$255,000
C Injury Crash	$149,000
Property Damage Only Crash	**$15,100**
Average Crash	$141,000
Injury Crash (F+A+B+C)	**$495,000**
Non-Fatal Injury Crash (A+B+C)	$220,000
Severe Injury Crash (F+A)	$4,129,000
Moderate Injury Crash (B+C)	$187,000

5. Estimated Scenario-Based Benefit-Cost Ratios

A benefit-cost ratio (BCR) is the ratio of the present-value benefits of a project to the implementation costs of the project (BCR = Benefits/Costs). If the ratio is greater than 1.0, then the project is considered economically justified. When multiple alternative solutions exist, countermeasures that produce the highest BCR for a specific site are the most desirable. The general methodology for calculating BCR is presented as follows:

1. Calculate the present value of the estimated savings in travel time and reductions in average crash frequency.
2. Calculate the present value of the costs associated with the safety improvement project.
3. Calculate the benefit-cost ratio by dividing the estimated project benefits by the estimated project costs.

$$BCR = \frac{PVB}{PVC}$$

Where,

BCR = Benefit cost ratio

PVB = Present value of project benefits

PVC = Present value of project costs

4. If the BCR is greater than 1.0, then the project is economically justified.

6. BCR Calculator for I-40 Case Study

The research team developed a spreadsheet tool to estimate the Benefit-Cost Ratios of the I-40 managed freeway project. The spreadsheet contains two functional tabs: Parameters Used, and Tech Options.

In addition to the functional tabs, the spreadsheet includes an Instructions tab that guides users through the basic steps required to calculate scenario-based BCRs. Specifically, the Parameters Used tab documents the parameters used in the benefit–cost analysis. Parameter values were primarily derived from four sources:

- Outputs from microsimulation modeling;
- Recommendations from the literature or state-of-the-practice guidance;
- Information provided by NCDOT staff or documented in NCDOT databases; and
- Calculations based on data collected from NCDOT roadway facilities.

Users may manually modify these values if more appropriate data are available. Moreover, in the mobility (travel time savings) and safety (crash modification factors) benefit analyses, lower and upper bounds were established alongside literature-recommended values using engineering judgment. The lower bound reflects a conservative estimation.

The Tech Options tab displays the estimated 5-year and 10-year B/C ratios for implementing managed freeway strategy(ies). Users can manually select individual strategies by entering the code "1" in the corresponding cells. Multiple strategies can be selected simultaneously, including all four or any combination. In this system, code "1" indicates inclusion of a strategy, while code "0" indicates exclusion.

Based on North Carolina data, the estimated B/C ratios for the evaluated strategies are presented in Table 10. When all strategies are implemented, the 10-year B/C ratio is estimated at 14.4 under conservative assumptions and increases to 30.0 under favorable assumptions. Among the four strategies analyzed, CARM emerges as the most cost-effective option, yielding a 10-year B/C ratio of 18.8 even under conservative estimates. Under favorable estimates, queue warning appears to be the most cost-effective option with a 10-year B/C ratio of 45.2. In contrast, variable speed limits do not appear to be a priority under conservative assumptions, as prior research indicates that VSL implementation may increase travel time. However, VSL remains a viable investment under aggressive estimation parameters, despite underperforming relative to alternative deployment options.

Table 10. Estimated Benefit-Cost Ratios under Various Scenarios

B/C Ratios using Conservative Estimates					
Analysis Period	**CARM Only**	**QW Only**	**VSL Only**	**LUCS Only**	**All Strategies**
5 Year	10.8	5.3	<1	10.2	8.6
10 Year	18.8	8.7	<1	16.4	**14.4**
B/C Ratios using Favorable Estimates					
Analysis Period	**CARM Only**	**QW Only**	**VSL Only**	**LUCS Only**	**All Strategies**

5 Year	22.1	28.4	13.7	19.6	18.7
10 Year	37.0	45.2	21.8	30.6	**30.0**

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Appendix

Conservative B/C ratios – CARM, VSL, QW and LUCS Implemented

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Conservative Estimate= 0, otherwise 1: 0

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
1	CARM	15.0%	2.5%	-18.0%
1	Queue Warning	-5.0%	10.0%	-20.0%
1	VSL/DSC	-4.5%	5.0%	4.0%
1	Lane Control	15.5%	3.0%	2.0%

	Capital (Year 0)	Maintenance (Annual (Yr1 $))	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate: 2.0% Inflation rate: 5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	31,623,185

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	31,623,185	1155	2839
Future (2035 build)	24,922,226	934	3782
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 144,325,249	$ 109,432,202	$ (14,241,033)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 114,753,250	$ -
1	$ 3,040,368	$ 234,820,017
2	$ 3,129,790	$ 230,215,703
3	$ 3,221,843	$ 225,701,670
4	$ 3,316,603	$ 221,276,147
5	$ 3,414,150	$ 216,937,399
6	$ 3,514,566	$ 212,683,725
7	$ 3,617,936	$ 208,513,456
8	$ 3,724,346	$ 204,424,956
9	$ 3,833,885	$ 200,416,624
10	$ 3,946,647	$ 196,486,886
11	$ 4,062,725	$ 192,634,202
12	$ 4,182,216	$ 188,857,061
13	$ 4,305,223	$ 185,153,981

5yr	Total PV	$ 130,876,004	$ 1,128,950,937
	Benefit/Cost	8.6	
10yr	Total	$ 149,513,384	$ 2,151,476,584
	Benefit/Cost	14.4	

Favorable B/C ratios – CARM, VSL, QW and LUCS Implemented

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Conservative Estimate= 0, otherwise 1: 1

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
1	CARM	21.2%	5.0%	-12.0%
1	Queue Warning	0.0%	16.0%	-16.0%
1	VSL/DSC	-2.5%	11.0%	8.2%
1	Lane Control	21.5%	6.0%	4.0%

	Capital (Year 0)	Maintenance (Annual (Yr1 $))	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate: 5.0% Inflation rate: 5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1.5
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	47,434,777

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	47,434,777	1155	2839
Future (2035 build)	30,075,759	771	3251
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 373,878,518	$ 190,060,649	$ (6,214,734)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 114,753,250	$ -
1	$ 2,891,000	$ 531,166,126
2	$ 2,891,000	$ 505,872,501
3	$ 2,891,000	$ 481,783,335
4	$ 2,891,000	$ 458,841,271
5	$ 2,891,000	$ 436,991,687
6	$ 2,891,000	$ 416,182,559
7	$ 2,891,000	$ 396,364,342
8	$ 2,891,000	$ 377,489,849
9	$ 2,891,000	$ 359,514,142
10	$ 2,891,000	$ 342,394,421
11	$ 2,891,000	$ 326,089,925
12	$ 2,891,000	$ 310,561,833
13	$ 2,891,000	$ 295,773,174

5yr	Total PV	$ 129,208,250	$ 2,414,654,920
	Benefit/Cost	18.7	
10yr	Total	$ 143,663,250	$ 4,306,600,233
	Benefit/Cost	30.0	

CARM Only

Conservative B/C ratios

Conservative Estimate= 0, otherwise 1: 0

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
1	CARM	15.0%	2.5%	-18.0%
0	Queue Warning	-5.0%	10.0%	-20.0%
0	VSL/DSC	-4.5%	5.0%	4.0%
0	Lane Control	15.5%	3.0%	2.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	2.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	31,623,185

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	31,623,185	1155	2839
Future (2035 build)	26,879,707	1126	3350
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 102,165,022	$ 14,294,866	$ (7,717,025)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 43,909,500	$ -
1	$ 707,721	$ 106,610,650
2	$ 728,536	$ 104,520,245
3	$ 749,963	$ 102,470,828
4	$ 772,021	$ 100,461,596
5	$ 794,728	$ 98,491,761
6	$ 818,102	$ 96,560,550
7	$ 842,164	$ 94,667,206
8	$ 866,933	$ 92,810,986
9	$ 892,431	$ 90,991,163
10	$ 918,679	$ 89,207,023
11	$ 945,699	$ 87,457,865
12	$ 973,514	$ 85,743,005
13	$ 1,002,147	$ 84,061,770

5yr	Total PV	$ 47,662,469	$ 512,555,080
	Benefit/Cost	10.8	
10yr	Total	$ 52,000,779	$ 976,792,008
	Benefit/Cost	18.8	

Favorable B/C ratios

Conservative Estimate= 0, otherwise 1: 1

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
1	CARM	21.2%	5.0%	-12.0%
0	Queue Warning	0.0%	16.0%	-16.0%
0	VSL/DSC	-2.5%	11.0%	8.2%
0	Lane Control	21.5%	6.0%	4.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	5.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1.5
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	47,434,777

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	47,434,777	1155	2839
Future (2035 build)	37,378,604	1097	3180
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 216,589,847	$ 28,589,732	$ (5,144,684)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 43,909,500	$ -
1	$ 625,000	$ 228,604,662
2	$ 625,000	$ 217,718,726
3	$ 625,000	$ 207,351,167
4	$ 625,000	$ 197,477,302
5	$ 625,000	$ 188,073,621
6	$ 625,000	$ 179,117,735
7	$ 625,000	$ 170,588,319
8	$ 625,000	$ 162,465,065
9	$ 625,000	$ 154,728,634
10	$ 625,000	$ 147,360,604
11	$ 625,000	$ 140,343,432
12	$ 625,000	$ 133,660,411
13	$ 625,000	$ 127,295,630

5yr	Total PV	$ 47,034,500	$ 1,039,225,479
	Benefit/Cost	22.1	
10yr	Total	$ 50,159,500	$ 1,853,485,835
	Benefit/Cost	37.0	

QW Only

Conservative B/C ratios

Conservative Estimate= 0, otherwise 1: 0

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	15.0%	2.5%	-18.0%
1	Queue Warning	-5.0%	10.0%	-20.0%
0	VSL/DSC	-4.5%	5.0%	4.0%
0	Lane Control	15.5%	3.0%	2.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	
Discount rate	2.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	31,623,185

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	31,623,185	1155	2839
Future (2035 build)	33,204,344	1040	3407
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ (34,055,007)	$ 57,179,464	$ (8,574,473)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra:	1155	2839

Year	PV Costs	PV Benefits
0	$ 11,375,000	$ -
1	$ 317,059	$ 14,264,690
2	$ 326,384	$ 13,984,990
3	$ 335,984	$ 13,710,775
4	$ 345,865	$ 13,441,936
5	$ 356,038	$ 13,178,368
6	$ 366,510	$ 12,919,969
7	$ 377,289	$ 12,666,636
8	$ 388,386	$ 12,418,271
9	$ 399,809	$ 12,174,775
10	$ 411,568	$ 11,936,054
11	$ 423,673	$ 11,702,014
12	$ 436,134	$ 11,472,563
13	$ 448,962	$ 11,247,611

5yr	Total PV	$ 13,056,330	$ 68,580,759
	Benefit/Cost	5.3	
10yr	Total	$ 14,999,893	$ 130,696,464
	Benefit/Cost	8.7	

Favorable B/C ratios

Conservative Estimate= 0, otherwise 1: 1

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	21.2%	5.0%	-12.0%
1	Queue Warning	0.0%	16.0%	-16.0%
0	VSL/DSC	-2.5%	11.0%	8.2%
0	Lane Control	21.5%	6.0%	4.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	
Discount rate	5.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1.5
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	47,434,777

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	47,434,777	1155	2839
Future (2035 build)	47,434,777	970	3294
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ -	$ 91,487,142	$ (6,859,578)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra:	1155	2839

Year	PV Costs	PV Benefits
0	$ 11,375,000	$ -
1	$ 308,000	$ 80,597,680
2	$ 308,000	$ 76,759,695
3	$ 308,000	$ 73,104,471
4	$ 308,000	$ 69,623,306
5	$ 308,000	$ 66,307,911
6	$ 308,000	$ 63,150,391
7	$ 308,000	$ 60,143,230
8	$ 308,000	$ 57,279,266
9	$ 308,000	$ 54,551,682
10	$ 308,000	$ 51,953,983
11	$ 308,000	$ 49,479,984
12	$ 308,000	$ 47,123,794
13	$ 308,000	$ 44,879,804

5yr	Total PV	$ 12,915,000	$ 366,393,063
	Benefit/Cost	28.4	
10yr	Total	$ 14,455,000	$ 653,471,614
	Benefit/Cost	45.2	

VSL Only

Conservative B/C ratios

Conservative Estimate= 0, otherwise 1: 0

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	15.0%	2.5%	-18.0%
0	Queue Warning	-5.0%	10.0%	-20.0%
1	VSL/DSC	-4.5%	5.0%	4.0%
0	Lane Control	15.5%	3.0%	2.0%

	Year 0	Annual (Yr1 $)	
	Capital	Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	2.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	31,623,185

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	31,623,185	1155	2839
Future (2035 build)	33,046,228	1097	2726
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ (30,649,507)	$ 28,589,732	$ 1,714,895

Year	PV Costs	PV Benefits
0	$ 11,375,000	$ -
1	$ 317,059	$ (338,118)
2	$ 326,384	$ (331,488)
3	$ 335,984	$ (324,988)
4	$ 345,865	$ (318,616)
5	$ 356,038	$ (312,369)
6	$ 366,510	$ (306,244)
7	$ 377,289	$ (300,239)
8	$ 388,386	$ (294,352)
9	$ 399,809	$ (288,580)
10	$ 411,568	$ (282,922)
11	$ 423,673	$ (277,375)
12	$ 436,134	$ (271,936)
13	$ 448,962	$ (266,604)

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

5yr	**Total PV**	$ 13,056,330	$ (1,625,579)
	Benefit/Cost	(0.1)	
10yr	**Total**	$ 14,999,893	$ (3,097,917)
	Benefit/Cost	(0.2)	

Favorable B/C ratios

Conservative Estimate= 0, otherwise 1: 1

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	21.2%	5.0%	-12.0%
0	Queue Warning	0.0%	16.0%	-16.0%
1	VSL/DSC	-2.5%	11.0%	8.2%
0	Lane Control	21.5%	6.0%	4.0%

	Year 0	Annual (Yr1 $)	
	Capital	Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	5.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1.5
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	47,434,777

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	47,434,777	1155	2839
Future (2035 build)	48,620,646	1028	2606
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ (25,541,256)	$ 62,897,410	$ 3,515,534

Year	PV Costs	PV Benefits
0	$ 11,375,000	$ -
1	$ 308,000	$ 38,925,417
2	$ 308,000	$ 37,071,826
3	$ 308,000	$ 35,306,501
4	$ 308,000	$ 33,625,239
5	$ 308,000	$ 32,024,037
6	$ 308,000	$ 30,499,083
7	$ 308,000	$ 29,046,746
8	$ 308,000	$ 27,663,567
9	$ 308,000	$ 26,346,255
10	$ 308,000	$ 25,091,671
11	$ 308,000	$ 23,896,830
12	$ 308,000	$ 22,758,885
13	$ 308,000	$ 21,675,129

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

5yr	**Total PV**	$ 12,915,000	$ 176,953,021
	Benefit/Cost	13.7	
10yr	**Total**	$ 14,455,000	$ 315,600,342
	Benefit/Cost	21.8	

LUCS Only

Conservative B/C ratios

Conservative Estimate= 0, otherwise 1: 0

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	15.0%	2.5%	-18.0%
0	Queue Warning	-5.0%	10.0%	-20.0%
0	VSL/DSC	-4.5%	5.0%	4.0%
1	Lane Control	15.5%	3.0%	2.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	2.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	31,623,185

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	31,623,185	1155	2839
Future (2035 build)	26,721,591	1120	2782
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 105,570,523	$ 17,153,839	$ 857,447

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 48,093,750	$ -
1	$ 1,698,529	$ 121,158,637
2	$ 1,748,486	$ 118,782,977
3	$ 1,799,912	$ 116,453,899
4	$ 1,852,851	$ 114,170,489
5	$ 1,907,346	$ 111,931,852
6	$ 1,963,445	$ 109,737,110
7	$ 2,021,193	$ 107,585,402
8	$ 2,080,640	$ 105,475,884
9	$ 2,141,835	$ 103,407,730
10	$ 2,204,831	$ 101,380,127
11	$ 2,269,679	$ 99,392,282
12	$ 2,336,434	$ 97,443,413
13	$ 2,405,152	$ 95,532,758

5yr	Total PV	$ 57,100,875	$ 582,497,854
	Benefit/Cost	10.2	
10yr	Total	$ 67,512,819	$ 1,110,084,107
	Benefit/Cost	16.4	

Favorable B/C ratios

Conservative Estimate= 0, otherwise 1: 1

Tool Input
Simulation output
Literature
NCDOT Provided
NC Data Calculation

Active	Treatment	TT Savings	Crash Reduction (KABC)	Crash Reduction (PDO)
0	CARM	21.2%	5.0%	-12.0%
0	Queue Warning	0.0%	16.0%	-16.0%
0	VSL/DSC	-2.5%	11.0%	8.2%
1	Lane Control	21.5%	6.0%	4.0%

	Year 0 Capital	Annual (Yr1 $) Maintenance	Admin
CARM	$ 43,909,500	$ 625,000	
Queue Warning	$ 11,375,000	$ 280,000	
VSL	$ 11,375,000	$ 280,000	
Lane Control	$ 48,093,750	$ 1,500,000	

Discount rate	5.0%	Inflation rate	5%

2035 Base VHT (3-7 pm weekday)	23,515
Number of weekdays per year	250
Occupancy - Factor	1.5
K - Factor (4hrs to daily)	0.338
Directional - Factor	0.55
Annual Bi-directional VHT	47,434,777

	Simulation VHT	Predicted KABC Crashes	PDO Crashes
Base (2022)			
Future (2035 no build)	47,434,777	1155	2839
Future (2035 build)	37,236,300	1086	2726
Unit benefit	$ 21.54	$495,000.00	$15,100.00
Annual Benefit	$ 219,654,798	$ 34,307,678	$ 1,714,895

	KABC	PDO
2022 AADT	147,125	
Estimated average 2035 AADT	178,544	
Corridor Bi-directional Lengh	39.8	
NC Annual Factor	1	
Alpha	-15.3488	-17.0107
Beta	1.5478	1.7596
e	2.718281828	
Year 2035 No Build Predicted Cra	1155	2839

Year	PV Costs	PV Benefits
0	$ 48,093,750	$ -
1	$ 1,650,000	$ 243,502,258
2	$ 1,650,000	$ 231,906,912
3	$ 1,650,000	$ 220,863,726
4	$ 1,650,000	$ 210,346,405
5	$ 1,650,000	$ 200,329,910
6	$ 1,650,000	$ 190,790,390
7	$ 1,650,000	$ 181,705,134
8	$ 1,650,000	$ 173,052,508
9	$ 1,650,000	$ 164,811,913
10	$ 1,650,000	$ 156,963,726
11	$ 1,650,000	$ 149,489,263
12	$ 1,650,000	$ 142,370,727
13	$ 1,650,000	$ 135,591,168

5yr	Total PV	$ 56,343,750	$ 1,106,949,210
	Benefit/Cost	19.6	
10yr	Total	$ 64,593,750	$ 1,974,272,882
	Benefit/Cost	30.6	