Field Evaluation of At-Grade Alternative Intersection Designs, Volume I—Operations Report

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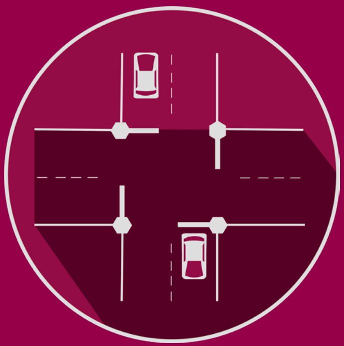

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
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FOREWORD

The research documented in this report was conducted as part of a multiyear research project for the Federal Highway Administration (FHWA). The report may be of interest to transportation practitioners conducting transportation design, operations, and safety research with an objective of better understanding the operational and safety performance of alternative intersection configurations.

The specific objective of this FHWA project was to investigate the operational and safety improvements of innovative intersections. The safety evaluation is included in a separate report. Where possible, the research incorporates operational elements that can help an agency better understand the benefits to pedestrians and bicyclists while maintaining service, where possible, to motor vehicles. These designs are most often referred to as alternative or innovative intersections. This project included identifying and evaluating up to 12 study sites suitable for before-after analyses. In addition to identifying the prospective sites, the research team assessed the suitability of each site, collected in-field data, and reduced the field data for use in comparison studies.

Carl K. Andersen
Director, Office of Safety and Operations
Research and Development

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This report summarizes the before and after operational performance of intersection modifications that have been converted from traditional intersections to alternative configurations. These changes are largely based on reconfiguration of left turns or U-turns. The studied locations were from Arizona, Minnesota, Texas, and Virginia.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or “metric ton”)	Mg (or “t”)
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or “t”)	megagrams (or “metric ton”)	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
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lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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

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LIST OF ABBREVIATIONS

COVID-19	coronavirus
DLT	displaced left turn
DOT	Department of Transportation
EB	eastbound
FHWA	Federal Highway Administration
MnDOT	Minnesota Department of Transportation
MUT	median U-turn
NB	northbound
QRI	quadrant roadway intersections
RCUT	restricted crossing U-turn
RLTCI	reduced left-turn conflict intersection
SB	southbound
vph	vehicles per hour
WB	westbound

CHAPTER 1. INTRODUCTION

In their simplest forms, intersections are locations where two or more roads cross and create the opportunity for roadway users to share a common point, which creates the potential for those roadway users to come into direct conflict. The intersection is a critical roadway element and one of the greatest sources of system delay and, in many cases, severe crashes. As a result, intersection design has historically focused on how best to optimize the characteristics of this common point to increase capacity and minimize crashes.

Intersection design has typically relied on moving all traffic from each intersecting roadway through the common point, referred to as the central intersection. As part of this design, whenever traffic volume levels increased on one or both roadways, stop signs were then replaced with signals, and multimovement shared lanes (i.e., through and right-turn lane) were replaced with exclusive lanes for right-turn, through, and left-turn movements. This change prompted more sophisticated traffic signals (where agencies replaced a single signal phase for each roadway with multiple signal phases for each roadway), and the addition of coordination between adjacent signalized intersections to help reduce delay and improve safety.

Despite continued advances in signal controller and detector technology, the actual physical geometric design of intersections has changed little over the last 50 yr. In the last decade, however, transportation agencies responsible for maintaining operational and safety performance for increasingly congested traditional intersections (where all through and turning movements are served directly at the central intersection) have sought innovative geometric solutions, where select indirect movements are provided to improve capacity and safety for the entire intersection. These new concepts, developed for a collection of unique intersections, are sometimes referred to as alternative intersections or innovative intersections. These alternative intersections create strategic micronetworks around the central intersection to potentially orchestrate traffic movements more efficiently while also reducing and dispersing conflict points for potential safety benefits.

CAPACITY ISSUES WITH TRADITIONAL INTERSECTIONS

A major challenge inherent to intersection design is optimizing high volume maneuvers, particularly left turns crossing opposing traffic. Executing these turning maneuvers involves crossing and conflicting with through and left-turning traffic on both approaches of the crossroad, through and right-turning traffic from the opposing approach on the same roadway, and pedestrian and bicycle crossings when present. Managing these intersection conflicts by separating movements in time can introduce substantial delays, particularly at locations where the left-turn volume is elevated and disproportionate to other movements.

Signalized traditional intersections can range from a simple two-phase intersection to as many as eight signal phases at a complex four-leg intersection. The increase in phasing is usually due to increasing traffic volumes, which require left-turn phases that remove the turn from direct conflict with other simultaneous movements at the intersection. As more signal phases are added, less green time is available for each signal phase. This change in turn requires additional lanes to move the same traffic volume.

Not all congested intersections are signalized. In many cases, high volumes on primary routes may necessitate providing priority to the major intersection approaches, often at the expense of efficient operations for the minor approaches. This scenario is often observed at roads with higher commuter trips and intersections that do not warrant a traffic signal other than during a.m. or p.m. peak periods.

ALTERNATIVE INTERSECTIONS

Alternative intersections can be distinguished from traditional intersections in three ways:

- Alternative intersections reduce the overall number of conflict points, particularly the most dangerous crossing conflict points, and disperse some of the conflicts located at the central intersection to other locations. These changes provide both operational and safety benefits.
- Alternative intersections accommodate left-turn maneuvers and sometimes through movements for one or both intersecting roadways by redirecting the vehicles to new locations before or after the central intersection.
- Alternative intersections, when signalized, typically reduce the number of signal phases from four or more to two phases per cycle.

While reducing signal phases and conflict points should provide apparent operational benefits, actual capacity benefits for individual alternative intersection forms need to be identified and quantified. A signalized alternative intersection, because the intersection has two signal phases per cycle, provides more green time to each of its two phases than traditional intersections with four, six, or eight signal phases per cycle. A decrease in the number of signal phases per cycle results in a corresponding increase in the lane capacity of the intersection. Since each lane in an alternative intersection is designed to move more vehicles than its traditional intersection counterpart, fewer through lanes and turning lanes are required to store and move the same volume of traffic. This streamlined cross section can result in less right-of-way to construct the lanes and less cost to build them. As a result of reduced delay, vehicle emissions generated from idling motor vehicles will also be minimized. Finally, reduced signal phases will decrease the frequency of starts and stops associated with multiphase traditional intersections.

CURRENT STATUS OF ALTERNATIVE INTERSECTIONS

The Federal Highway Administration (FHWA) notes that in 2020, there were 38,824 roadway fatalities in the United States and that 27 percent of the fatalities were at an intersection or were intersection related.(1) Because of the potential capacity benefits associated with alternative intersections, States and local jurisdictions have adopted and applied alternative intersections to help optimize the operational performance of the intersection. The transportation profession has already learned much from existing applications but also needs to analyze the performance of individual intersection configurations and individual roadway elements to better determine optimal design strategies. Alternative intersection concepts continue to evolve, but the optimal intersection geometry (including turn offsets, bicycle and pedestrian paths, and adjacent access points) still must be fully determined.

The general driving population is also unfamiliar with these unique intersections, and the transportation profession has voiced concerns that this unfamiliarity could result in driver confusion and incorrect maneuvers through the intersections. There is a growing need, therefore, to conduct research so that the transportation community can be better educated and informed of the expected capacity, geometry, and cost associated with alternative intersections.

CURRENT STUDY DESIGN

Currently the number of alternative intersections in the United States is limited relative to the number of traditional intersections, although the number of alternative intersections continues to grow. One of the most widely constructed configurations is an adaptation that features median U-turn (MUT) intersections to facilitate indirect left-turn movements. These intersections are often referred to generally as reduced conflict intersections or reduced left-turn conflict intersections (RLTCI).(2) The most common specific types of these intersections are MUT and restricted crossing U-turn (RCUT) designs.(3,4,5) These designs are featured among the FHWA Proven Safety Countermeasures as RLTCI.(6) Other alternative intersections include displaced left-turn (DLT) designs and quadrant roadway intersections (QRI).(7,8) All alternative intersection configurations need to be consistently evaluated, and their operational benefits need to be documented. This information is vital to transportation professionals who need to make informed decisions on whether to select a traditional or alternative intersection and how to potentially choose between competing alternative intersections. Because of the relatively limited number of alternative intersection forms, the geometrics of each alternative intersection are evolving with every new intersection that is constructed. Knowing what works and what does not work from past designs will help improve future designs.

This report summarizes the observed operational performance of constructed alternative intersections. This report includes:

- Chapter 2—A review of related operational performance literature.
- Chapter 3—An overview of the site identification process.
- Chapter 4—An overview of the methodology selected for this study.
- Chapter 5—A summary of the operational analysis l findings that resulted from this study.
- Chapter 6—Information about observed road user behavior.
- Chapter 7—A summary of overall findings and recommendations that result from this study.
- An appendix that provides supplemental information.

A companion report documents the safety findings from this study.(9)

CHAPTER 2. LITERATURE REVIEW—OPERATIONAL PERFORMANCE

Chapter 1 reviewed the growing need of transportation agencies to construct alternative intersections when traditional intersections no longer function effectively. These unique intersections present a viable alternative to compensate for challenges such as an oversaturated left turn.

The field evaluation summarized in this report focused on operational performance. Volume II reviews documented safety performance.[9] In general, the published literature focuses on the operational performance of intersections where the left turn has been modified. The most common modifications include:

- Restricting the left turn or through movement for maneuvers that originate on the minor road by constructing a U-turn so that these vehicles turn right, execute a U-turn, and then either continue straight (for the left turn equivalent) or turn right (for the through movement equivalent). The RCUT intersection is an example of this type of intersection.
- Restricting the left-turn movement for maneuvers originating on the major road by constructing midblock U-turns. The MUT intersection is an example of this configuration.
- Providing enhanced operations by shifting what would have been the left-turning traffic across the opposing traffic at signalized intersections located upstream and downstream of the primary intersection location. This configuration provides priority to the left turn and permits fewer traffic signal phases. The DLT is an example of this configuration.
- Removing conflicting left turns completely by constructing a road that functions similarly to a highway loop ramp. This configuration is known as a quadrant.

In some cases, individual transportation agencies have developed unique alternative intersections that combine features of other intersections. These alternatives are referred to as hybrid intersections throughout this report.

This literature review explored operational performance measures and the known effects of these measures as they relate to the alternative intersection types included in this study. In some cases, the operational performance of the alternative intersection is known, while in other cases little is known. Subsequent chapters include schematics that graphically depict the layout of these intersections, so this information has not been repeated in this summary of the literature. Some of the potential operational performance that is presented in the published literature includes comparison of before and after travel times, comparison of before and after queue lengths, and influence on pedestrian walking paths, where applicable, due to the changes.

MUT INTERSECTION

An MUT is one of the most common alternative intersection treatments. The following sections present a brief description of the MUT, MUT performance measures, and known operational performance.

MUT Description

A typical MUT configuration consists of one main intersection and two median U-turn openings. An MUT is any intersection where direct left turns are replaced with indirect left turns using a U-turn movement at locations with wide medians.(3) The minor road left-turn movements are indirect and rerouted through downstream U-turns. All left turns must occur at one of the three crossover locations. Removal of left-turn movements at the main intersection increases efficiency.(10) Traffic signals at MUT locations are optional, depending on the demand. Variations to the MUT include placing directional crossovers on the major or minor road, adding a loon (an expanded paved apron opposite a median crossover) so that the median can be narrowed, and incorporating a stop-controlled directional crossover upstream of the intersection.

MUT Performance Measures

A 2022 study by Jovanovic and Teodorovic evaluated fixed-time traffic control at alternative intersections.(11) Their study explored the optimization of the fixed-time traffic signal at an intersection location, using an approach based on the bee colony optimization analysis method. Their study focused on assessing how an optimization of cycle lengths performed for an alternative intersection in contrast to the traditional Webster method. Though their research focused on various traffic signal optimization strategies, the research team did conclude that an effective operational performance measure is the experienced travel time in vehicle seconds for all vehicles entering the RCUT. The research team further developed a performance measure that identified the distance from the main junction to the U-turn maneuver and the distance from the U-turn to the main junction.

The enhanced operational performance of an MUT can be expected to improve because the traffic signal can be reduced to a two-phase cycle, resulting in a shorter cycle length.(3) This modification removes the left-turning vehicles from the primary road and relocates the movements to an MUT, which provides additional time for through vehicles, resulting in improved capacity. The simplified traffic signal design also helps to facilitate corridor progression.

In addition, the removal of direct left turns by routing these vehicles through a U-turn eliminates the need for these vehicles to queue at a traffic signal, which can result in travel times that are similar to those observed for conventional intersections.

In addition to travel time and shorter cycle lengths due to simplification of the traffic signal, the FHWA *Median U-Turn Intersection Informational Guide* indicates that additional performance measures could include speed, delay, queues, and number of stops.(3)

MUT Operational Performance

A variety of studies have focused on the operational performance of an MUT.[12–21] The FHWA *Median U-Turn Intersection Informational Guide* summarizes the operational benefits of an MUT in contrast to a conventional intersection as follows:[3]

- Capacity can be expected to increase from 14 up to 18 percent.
- Total throughput should increase from 15 up to 40 percent.
- Vehicles stopping in the network ranged from 20 to 40 percent fewer.
- Critical lane volumes can be expected to reduce by approximately 17 percent.

Additional analysis strategies can be used to assess the MUT, including corridor performance.

RCUT INTERSECTION

The following sections describe RCUT performance measures as published in the literature.

RCUT Description

Rural high-speed highways are one of the most dangerous locations on the road network because a driver must estimate the speed of approaching vehicles and then select a safe gap.[22,23] The implementation of an RCUT at these high-speed locations can help improve safety without compromising operational performance. An RCUT is similar to an MUT, except where the MUT reroutes the left-turn maneuver from the major and minor roads, the RCUT reroutes minor street left turns and through movements.[20] When the RCUT is applied to a corridor, this network of intersections is sometimes referred to as superstreets or synchronized streets.[24] Associated intersection traffic control can range from signalized, to stop-controlled, to yielding or merging configurations.

Researchers have explored a variety of potential measures that can be used to assess the operational performance of a single RCUT or multiple RCUTs (i.e., superstreets).

RCUT Performance Measures

The 2021 study by Appiah evaluated the development of traffic signal warrants at RCUT locations.[25] Appiah used a microsimulation tool widely used by the transportation professional community to develop 72 combinations of geometry and traffic RCUT combinations suitable for assessing recommended traffic control configurations. Their target performance measures included:[20]

- Average system delay.

- 95th percentile queue lengths at U-turns, right-turn lanes, and left-turn lanes (to establish a practical upper limit for queue length vehicle storage).

- Travel time during peak and offpeak periods.

In addition to the more common performance measures, several less common roadway elements may be considered, including speed metrics, geometric elements, and pedestrian and bicycle accommodations. A study by Sun et al. focuses on how acceleration and deceleration lanes should be placed in proximity to the U-turn and primary intersection.(26) The study found that a combination of acceleration and deceleration lanes performed better than just a deceleration lane.

RCUT Operational Performance

The 2021 research paper by Appiah used the RCUT as a case study for developing suitable traffic guidance.(25) The paper noted the following items as part of this evaluation:

- Minor traffic volume is a critical characteristic in considering the need for identifying the need for traffic signals at RCUT intersections.
- RCUT operation can benefit from the placement of traffic signals at all three intersections if the minor road traffic volume exceeds 575 vehicles per hour (vph) and the main road has four lanes. For two-lane roadways, the traffic signal would be recommended for minor road volumes of 450 vph or greater.

Edara et al. determined that the average wait time at an RCUT (i.e., J-turn) of 5 s was approximately half the 11 s Edara et al. observed at the control site.(27)

A Minnesota RCUT case study observed that travel time may slightly increase during offpeak periods, but this observation is offset by a reduced delay during high volume conditions.(23) Similarly, a study in Wilmington, NC, observed that vehicles were moving through the intersection about 20 percent faster, even though the corridor traffic volume increased. Simulations of the site estimated an approximate 25 percent reduction in travel times during peak periods.(28) A study in Holly Springs, NC, observed similar travel time results.(29)

DLT INTERSECTIONS AND INTERCHANGES

The following sections summarize the description and performance of DLT intersections and interchanges.

DLT Description

A DLT features a left-turn movement that crosses over the opposing traffic to a parallel lane where the adjacent traffic flows in the opposite direction. This traffic is then directed to the cross street. This design is intended to optimally facilitate an enhanced left-turn maneuver.(30) This intersection type is also sometimes referred to as a continuous flow intersection. The DLT configuration can occur for at-grade intersections or at interchange locations. The layout of a DLT includes a crossover intersection located upstream of the main intersection.

DLT Performance Measures

The DLT intersection introduces an additional level of complexity for the traffic signal timing and corridor progression. In addition, the design must enable drivers to clearly identify their path without unexpected complications, such as driveways located too close to the crossover intersections.[30] Common measures of effectiveness may include travel time, speed, delay, queue length, and number of stops.

DLT Operational Performance

Hughes et al. conducted comparative simulation studies to assess the expected performance for several DLT performance measures.[31] The studies determined that intersection delay for four DLT intersections was reduced from 10 to 90 percent compared to a conventional intersection (this reduction was 36 to 39 percent for two DLTs). Similarly, the queue length was reduced 34 to 88 percent, and the number of stops was reduced 15 to 30 percent. The throughput, however, increased for four DLTs and for two left-turn lanes. The *Displaced Left Turn Intersection Informational Guide* further recommends the following seven basic measures of effectiveness when performing operational analyses:[30]

- Travel time.
- Speed.
- Delay.
- Queues.
- Stops (based on a minimum vehicle operating speed).
- Density.
- Travel time variance.

QUADRANT INTERSECTION

At the intersection of two major urban or suburban roads, a quadrant may be a suitable way to manage the heavy traffic volumes. This technique reroutes all four of the left-turn movements to a corridor connector for one of the intersection quadrants. The two-phase main intersection does not permit any left turns. The beginning and ending points of the quadrant can be expected to have more common traffic signal configurations (often a three-phase design). Though elements of a quadrant have been constructed in the Unites States at the time of development of this report, there were not any known fully developed quadrant at-grade intersections.[30]

ALTERNATIVE HYBRID INTERSECTION

In many cases, a transportation agency has limited available right-of-way at locations characterized by challenging operational constraints. When this situation occurs, many agencies combine individual elements of other alternative intersections to create a unique composite intersection specifically designed for the location. Because this type of intersection is a variant of multiple alternative intersection configurations, little is currently known about this type of intersection’s operational performance. For this reason, the performance characteristics of hybrid intersections are not available in the published literature. However, two of the intersections included in the field studies that were evaluated for this project should be classified as hybrid. Future research may benefit from an analysis of hybrid options at composite intersection locations.

CHAPTER 3. OVERVIEW OF ALTERNATIVE INTERSECTION CONFIGURATIONS

The design and construction of alternative intersections is largely based on the recognition that a saturated roadway system with disproportionate traffic volumes may not always function in an unsaturated condition by adding or modifying traffic control devices or simply adding extra through lanes. Consequently, the ultimate primary operational goal for constructing an alternative intersection is to relocate movements from the central intersection that does not function optimally, and to relocate these movements so that extra intersection capacity can be provided. Alternative intersections are successful if they increase capacity in terms of overall throughput. To accommodate this goal, a transportation agency must target enhancement of the congested maneuvers and optimize intersection operational performance.

For this research project, the team conducted a multiyear study, identifying candidate alternative intersection sites for which transportation agencies were considering implementation in the immediate future. The research team reached out to these agencies and identified sites where design was nearing completion and construction was eminent. For the identified candidate locations, team members met with project representatives and requested permission to conduct before-construction and after-construction studies of the intersections. This approach enabled the team to directly measure actual operations of alternative intersections in the field and evaluate that information to determine good design practice for each specific type of alternative intersection studied. In some cases, the intersection of interest was isolated, while at other locations the alternative intersections were constructed in a series of consecutive RCUTs. Intersections constructed in a series are often referred to as superstreets.

STUDY SITES SELECTION

In the United States, transportation agencies are usually required to conduct public involvement activities as their design develops. These activities are usually well advertised and a matter of public record, which allowed the research team to develop a preliminary list of potential intersections to consider for this analysis. In addition, the research team worked with FHWA to establish criteria for the selected sites. Ultimately, the team established the following general requirements:

- No sign of construction should be evident at the site. This restriction includes no construction signs, no barrels, and no utility work.
- Data collection should be scheduled, if possible, on weekdays at times when there are no special events occurring. The city of Tucson, AZ, holds an annual gem festival, and the research team had to work around that schedule for this study.
- The stakeholder agencies were supportive of this research activity and willing to assist if needed.
- Construction needed to begin shortly after the completion of the preliminary before-data collection and end at least 2 yr before the proposed after-data collection.

Unfortunately, the last item could not be achieved due to the coronavirus (COVID-19) pandemic. Before the pandemic, the team had completed collection of all of the before data. The project team intended to begin scheduling the after-data collection during the spring of 2020, and then collect the remaining data during 2021. This schedule provided sufficient time for road users to adjust to the new intersection configuration. The associated safety information is included in a companion report and did include some flexibility in the proposed safety data analysis schedule.(9)

A second unexpected challenge occurred when the Texas Department of Transportation (DOT) paused work on the proposed RCUTs at three locations in College Station, TX. Work on those three intersections was later resumed, and the sites were under construction at the time of the writing of this report. A future assessment could be conducted on these sites so that this information is included in the larger knowledge base of innovative intersections.

Because three intersections were not available for the after analysis, the research team worked with FHWA to identify three substitute locations along existing study corridors. Two Minnesota sites (one with the RCUT and one without an RCUT but located along the RCUT corridor) and one indirect left-turn site in Arizona were added.

The research team initially identified 12 different sites to study. Ultimately, 15 sites were selected, with some sites serving as comparison sites to replace the sites where after data could not be collected due to the timing of the contract. The four Arizona sites, five Minnesota sites, four Texas sites, and two Virginia sites are all listed in table 1. This table identifies the type of intersection design for the site and the period of data collection (i.e., before data, after data, or comparison data). Ultimately the team collected before and after data at 12 of the 15 study sites. In some cases, construction was delayed due to the 2020 COVID-19 pandemic. This situation impacted construction schedules as well as data collection schedules for after data. Ultimately, the three College Station, TX, intersections encountered substantial delay, and the team was forced to identify three alternative study locations that could be used for comparison sites in Arizona and Minnesota. The research team did collect before data for the three College Station, TX, sites and would encourage an after study once the sites are completed (the sites are currently under construction). Table 2 provides the construction start and end dates.

Table 1. Sites included in study.

State	Intersection, City	After Period Intersection Design
AZ	Grant Road at First Avenue, Tucson	Signalized main intersection with MUTs (also known as indirect left turn or ThruU in AZ)
AZ	Grant Road at Oracle Road North, Tucson	Signalized main intersection with MUTs (also known as indirect left turn or ThruU in AZ)
AZ	Grant Road at Stone Avenue, Tucson	Signalized main intersection with MUTs (also known as indirect left turn or ThruU in AZ)
AZ	Valencia Road at Kolb Road, Tucson	Signalized hybrid intersection (quadrant road and MUTs)
MN	MN–65 at 157th Avenue NE, Ham Lake	Unsignalized RCUT
MN	MN–65 at 181st Avenue NE (Baltimore Street NE), Ham Lake	Unsignalized RCUT
MN	MN–65 at 187th Lane NE, East Bethel	Unsignalized RCUT
MN	MN–65 at 209th Avenue NE, East Bethel	Traditional two-way, stop-controlled intersection
MN	MN–65 at Viking Boulevard NE, East Bethel	Signalized RCUT
TX	FM–2818 at George Bush Drive West, College Station	Signalized RCUT (future)
TX	FM–2818 at Luther Street West, College Station	Signalized RCUT (future)
TX	FM–2818 at Holleman Drive South, College Station	Signalized RCUT (future)
TX	SH–16 (Bandera Road) at West Loop 1604 Access Road, San Antonio	Signalized DLT interchange
VA	Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165), Norfolk	Signalized intersection with DLT on north and south approaches
VA	Indian River Road at Kempsville Road, Virginia Beach	Signalized hybrid intersection (DLT on two approaches and MUT on two approaches)

NE = northeast.

Table 2. Construction dates and data collection periods for sites included in study.

State, Site	Construction Start to End	Data Collection Periods
AZ, Grant Road at First Avenue	08/2017 to 10/2018	Before and after
AZ, Grant Road at Oracle Road North	08/2017 to 10/2018 (comparison site)	After only, replacement site to use for comparison to region characteristics
AZ, Grant Road at Stone Avenue	08/2017 to 10/2018	Before and after
AZ, Valencia Road at Kolb Road	06/2018 to 09/2019	Before and after
MN, MN–65 at 157th Avenue NE	07/2018 to 8/2019	Before and after
MN, MN–65 at 181st Avenue NE	07/2018 to 08/2019 (comparison site)	After only, replacement site to use for comparison to region characteristics
MN, MN–65 at 187th Lane NE	07/2018 to 8/2019	Before and after
MN, MN–65 at 209th Avenue NE	NA (comparison site)	After only, replacement site to use for comparison to region characteristics
MN, MN–65 at Viking Boulevard NE	07/2018 to 08/2019	Before and after
TX, FM–2818 at George Bush Drive West, College Station	02/2021 under construction at the time of this report	Before only, construction not completed at time of report development
TX, FM–2818 at Luther Street West, College Station	02/2021 under construction at the time of this report	Before only, construction not completed at time of report development
TX, FM–2818 at Holleman Drive South, College Station	02/2021 under construction at the time of this report	Before only, construction not completed at time of report development
TX, SH–16 (Bandera Road) at West Loop 1604 Access Road, San Antonio	09/2017 to 04/2019	Before and after
VA, Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165), Norfolk	08/2016 to 07/2018	Before and after
VA, Indian River Road at Kempsville Road, Virginia Beach	03/2019 to 3/2020	Before and after

NA = no construction dates were available.

INTERSECTION CONFIGURATIONS

The most common type of alternative intersection included in this study is the RLTCI, with both variants represented (the RCUT and the MUT). The Arizona DOD refers to their variant of the RCUT as a ThruU. This configuration generally includes intersections with an RCUT maneuver, but the cross-street traffic is not diverted by an island in the central intersection. The research team's goal was to find a diverse selection of intersections but include at least two of any intersection type where possible. Ultimately, the team identified three common intersection configurations:

- RLTCI.
- DLT.
- Quadrant road.

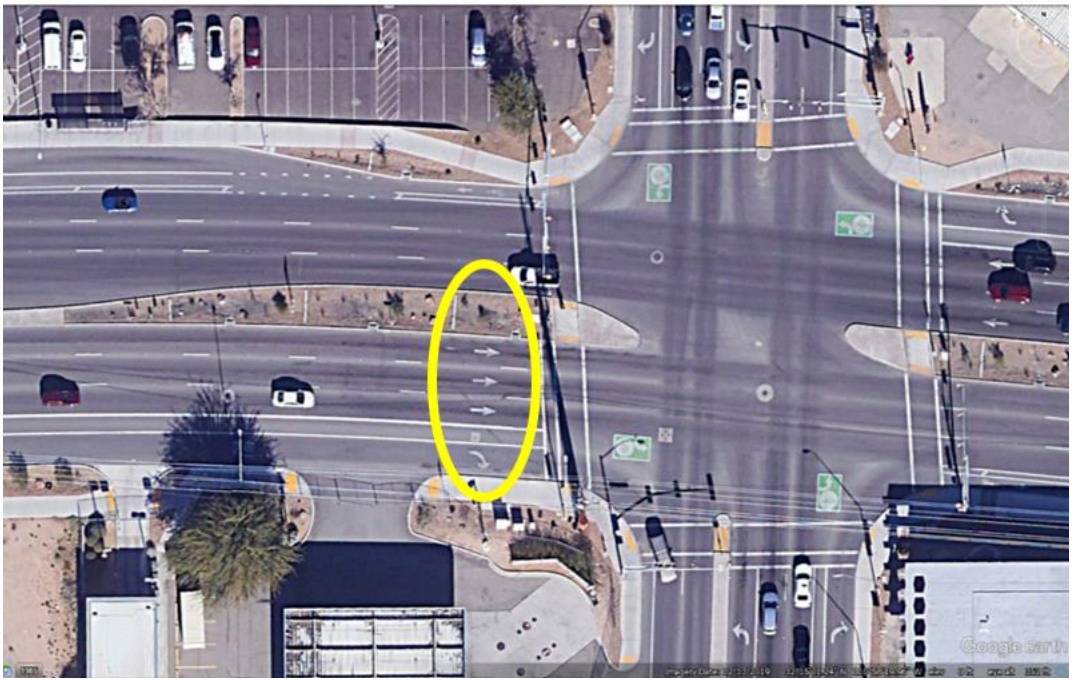
For the RLTCI and DLT, one or more left-turn configurations crossed over upstream of the intersection. The DLT is also known as a continuous-flow intersection. The RLTCI configurations included signalized MUT intersections (primarily in Arizona) and unsignalized RCUT intersections (in Minnesota). In addition, FHWA gave the research team permission to include one interchange with a displaced left upstream of the terminal intersection. Though there was no direct comparison location, the two Virginia sites also included a DLT for at-grade intersections and these data can be contrasted. The Texas site also repurposed an existing Texas turnaround (also referred to as a Texas U-turn) into the path for the displaced left. While the two common treatments were RLTCI and DLT, other intersection forms were present, such as a quadrant road intersection in Tucson, AZ, and hybrid intersections that included more than one alternative intersection treatment type. The following sections highlight the features of each intersection type.

Arizona Signalized Main Intersection with MUT Intersections and Signalized Hybrid Intersections

The research team identified three prospective Arizona intersections deemed suitable for the before-after analysis. In addition, during the after-data collection process, the team added an additional comparison site that was available in the Grant Road corridor. This additional intersection is at Grant Road and Stone Avenue. Because the site was already under construction at the time of the initial before data collection, the Stone Avenue intersection was initially excluded. Due to the delayed construction of the three College Station, TX, sites, the research team was able to include the intersection for after cross section comparisons. The four intersections located in Tucson, AZ, are:

- Grant Road East at First Avenue North.
- Grant Road West at Oracle Road North (comparison site).
- Grant Road (transition from East to West) at Stone Avenue North.
- Valencia Road East at Kolb Road South.

Three of the four Arizona study sites have an indirect left-turn intersection (MUT) that is also sometimes also referred to as a ThruU. The MUT configuration is similar to the RCUT with primary road left-turning vehicles going past the intersection and turning first at a U-turn and then executing a right turn. These configurations differ in that the indirect left-turn intersection does permit through traffic from the minor approach to cross the major street without also executing a right turn followed by a U-turn. This scenario is better demonstrated in the following figure, where the eastbound (EB) approach has white straight arrow pavement markings (circled). The fourth site has a hybrid intersection configuration.



Original Map: © 2017 Google® Earth™. Modified by FHWA (see Acknowledgment section).

Figure 1. Photo. Example of straight arrow pavement markings on an approach where major road left turns are to use a MUT (ThruU intersection).[32]

Grant Road at First Avenue, Arizona

The research team acquired after data at the intersection of Grant Road at First Avenue in Tucson, AZ. The key design features of the Grant Road improvement plan included indirect left turns, eight bus pullouts with shelters and benches, a bike and pedestrian signal at 6th Avenue, 8-ft sidewalks, and 7-ft bike lanes. Figure 2 shows an aerial view of the intersection in the before condition. Figure 3 shows an overview of the main intersection and neighboring U-turns during the after period. Figure 4 shows a closeup of the main intersection in the after period.

© 2017 Google® Earth™.
Note: Grant Road runs east–west and First Avenue runs north–south, north at top.

Figure 2. Photo. Grant Road at First Avenue intersection, before condition in Tucson, AZ.[32]

© 2019 Google® Earth™.
Note: Grant Road runs east–west and First Avenue runs north–south, north at top.

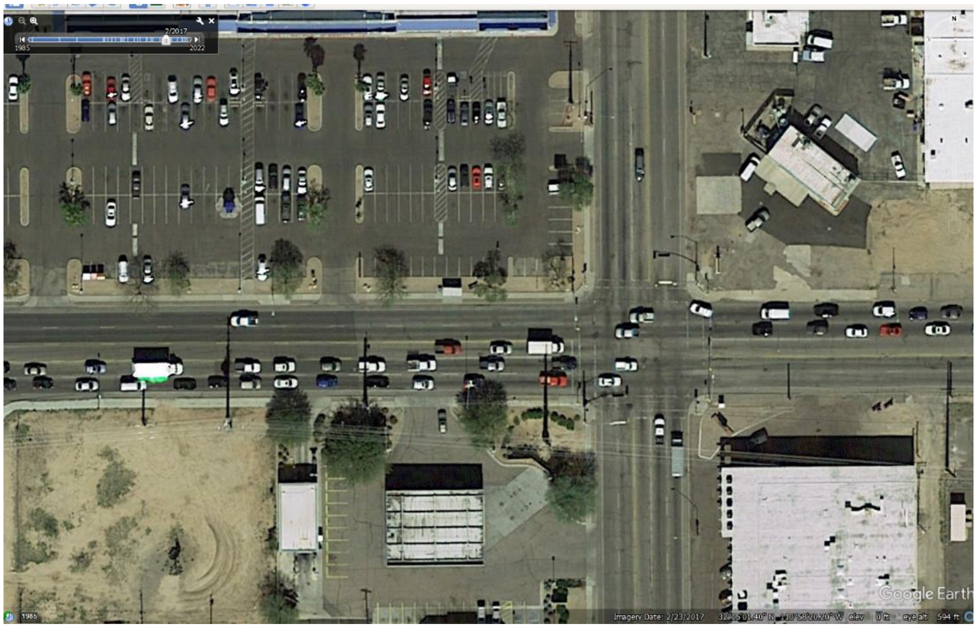
Figure 3. Photo. Grant Road at First Avenue intersection, overview of main intersection and U-turns, after condition in Tucson, AZ.[32]

© 2019 Google® Earth™.
Note: Grant Road runs east–west and First Avenue runs north–south, north at top.

Figure 4. Photo. Grant Road at First Avenue intersection, closeup of main intersection, after condition in Tucson, AZ.[32]

Grant Road at Stone Avenue in Tucson, AZ

The research team acquired after data at the intersection of Grant Road at Stone Avenue in Tucson, AZ. This intersection was part of the second phase of the Grant Road improvement plan. The plan's main objective was improving regional mobility by introducing the indirect left turn. Figure 5 shows an aerial view of the intersection in the before condition. Figure 6 shows an overview of the main intersection and neighboring U-turns during the after period. Figure 7 shows a closeup of the main intersection in the after period.



© 2017 Google® Earth™.
Note: Grant Road runs east–west and Stone Avenue runs north–south, north at top.

Figure 5. Photo. Grant Road at Stone Avenue intersection before condition (photo date February 23, 2017) in Tucson, AZ.[32]

© 2019 Google® Earth™.
Note: Grant Road runs east–west and Stone Avenue runs north–south, north at top.

Figure 6. Photo. Grant Road at Stone Avenue intersection, overview of main intersection and U-turns, after condition in Tucson, AZ.[32]

© 2019 Google® Earth™.
Note: Grant Road runs east–west and Stone Avenue runs north–south, north at top.

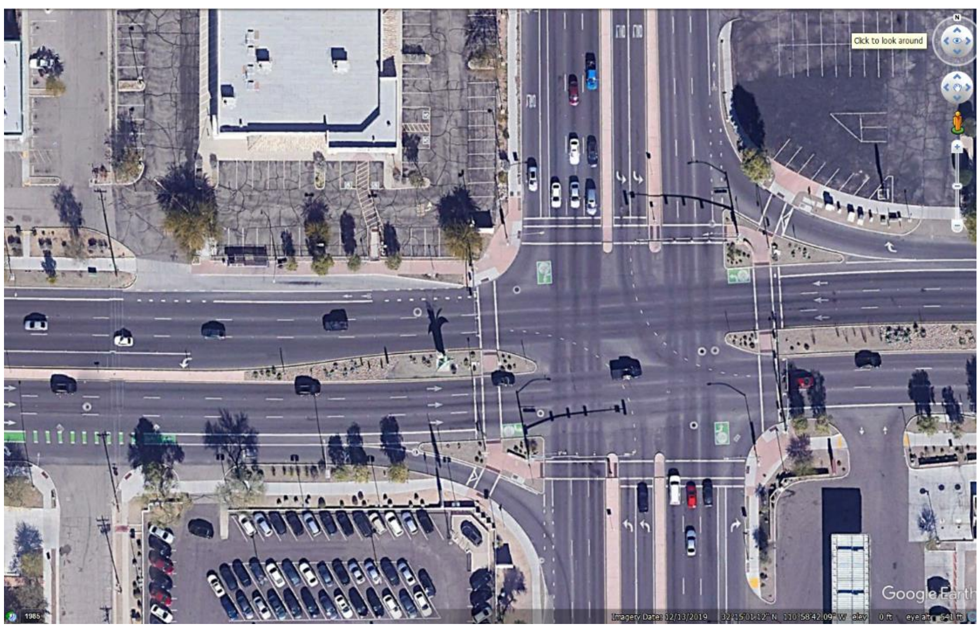
Figure 7. Photo. Grant Road at Stone Avenue intersection, closeup of main intersection, after condition.[32]

Grant Road at Oracle Road North, Arizona

The research team collected posttreatment data at the intersection of Grant Road at Oracle Road North in Tucson, AZ, and used this information for cross-sectional comparisons. Figure 8 shows an overview of the main intersection and neighboring U-turns during the after period and figure 9 shows a closeup of the main intersection in the after period. The key design features for the second phase of the Grant Road improvement plan included: indirect left turns, eight bus pullouts with shelters and benches, bicycle and pedestrian signals at 6th Avenue, 8-ft wide sidewalks, and 7-ft wide bicycle lanes. Similar pedestrian, bicycle, and transit treatments were included along the improvement area of the Grant Road corridor. This characteristic makes Oracle Road North a good option for comparing sites with the MUT configuration.

© 2019 Google® Earth™.
Note: Grant Road runs east–west and Oracle Road North runs north–south, north at top.

Figure 8. Photo. Grant Road at Oracle Road North intersection, overview of main intersection and U-turns (comparison site), after condition in Tucson, AZ.(32)



© 2019 Google® Earth™.
Note: Grant Road runs east–west and Oracle Road North runs north–south, north at top.

Figure 9. Photo. Grant Road at Oracle Road intersection, closeup of main intersection (comparison site), after condition in Tucson, AZ.(32)

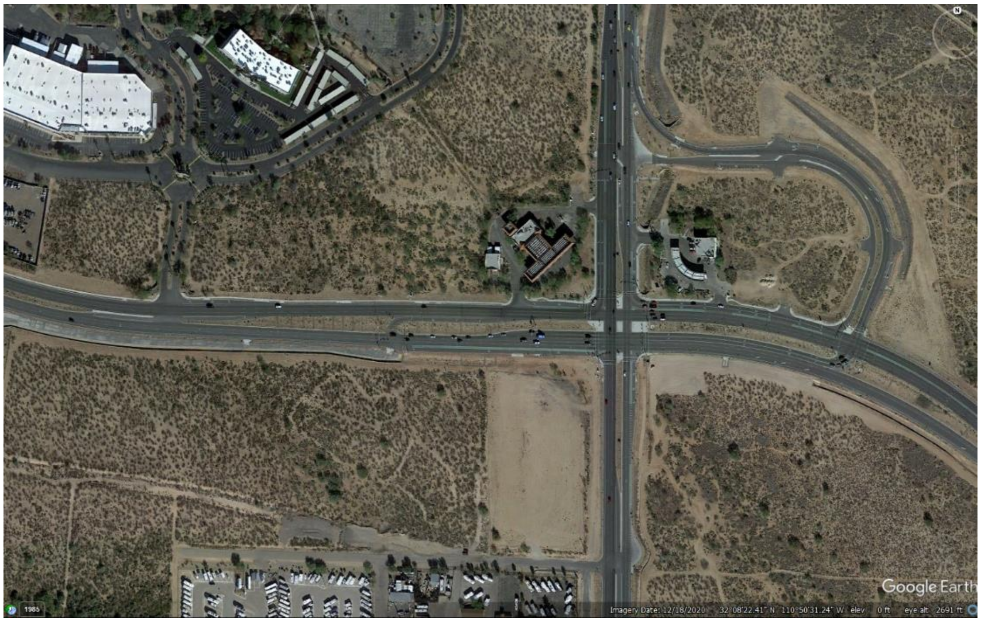
Valencia Road at Kolb Road, Tucson, AZ

The intersection of East Valencia Road at Kolb Road is located within the city limits of Tucson, AZ; however, the Regional Transportation Authority administered the project for the city. Figure 10 shows an aerial view of the Kolb Road at Valencia Road intersection during the before condition. The before condition was a traditional intersection. The after configuration included significant access modifications. Figure 11 shows that the after condition added MUTs and an additional access road that functions like a quadrant. In addition, the lengths of the deceleration lanes were extended to accommodate expected queues and midblock U-turns were added. Figure 12 provides a closeup view of the main intersection.

© 2017 Google® Earth™.

Notes: Valencia Road runs east–west and Kolb Road runs north–south, north at top.

Figure 10. Photo. Valencia Road at Kolb Road intersection, overview of main intersection and U-turns, before condition in Tucson, AZ.[(32)]



© 2020 Google® Earth™.
Note: Valencia Road runs east–west and Kolb Road runs north–south, north at top.

Figure 11. Photo. Valencia Road at Kolb Road intersection, overview of main intersection and U-turns, after condition in Tucson, AZ.[(32)]

© 2020 Google® Earth™.
Note: Valencia Road runs east–west and Kolb Road runs north–south, north at top.

Figure 12. Photo. Valencia Road at Kolb Road intersection, closeup of main intersection and U-turn, after condition in Tucson, AZ.[(32)]

Minnesota RCUT Sites

The five Minnesota sites were all located along the north-south corridor of MN–65. This corridor is north of Minneapolis and primarily functions as a commuting highway. The a.m. peak hour occurs in the southbound (SB) direction, so the p.m. peak hour affects the northbound (NB) direction. Except for the MN–65 at Viking Boulevard NE intersection, the intersections are unsignalized. The research team collected after data at the intersection of MN–65 and 181st Avenue. The team also collected similar data at MN–65 and 209th Avenue NE to explore the after data for this traditional intersection compared to that of the corridor to the south. The team used these data to determine if the lack of an RCUT along a corridor with a series of these unique intersections would introduce any operational challenges for the traditional intersection. The five Minnesota sites (listed in order from south to north) included:

- MN–65 at 157th Avenue NE, Ham Lake, MN—Unsignalized RCUT.
- MN–65 at 181st Avenue NE (Baltimore Street NE), Ham Lake, MN—Unsignalized RCUT.
- MN–65 at 187th Lane NE, East Bethel, MN—Unsignalized RCUT.
- MN–65 at Viking Boulevard NE, East Bethel, MN—Signalized RCUT.
- MN–65 at 209th Avenue NE, East Bethel, MN—Traditional intersection serving as comparison site.

The unsignalized RCUT intersection has stop signs on the minor approaches. Through traffic on the primary highway does not stop at the intersection The cross-street (or minor road) traffic that wants to turn left must turn right onto the major highway and then execute a U-turn. The minor road traffic that wants to go straight must also turn right onto the major highway, execute a U-turn, and then turn right at the main intersection to resume travel on the minor road. The following information briefly summarizes each of these intersections.

MN–65 at 157th Avenue NE, Unsignalized RCUT

The location of the intersection with 157th Avenue NE is entirely within the city limits of Ham Lake, MN, but the construction project was administered by the Minnesota Department of Transportation (MnDOT). The project construction spanned 2018 to 2019. Figure 13 is an aerial view of the intersection before construction. Figure 14 is an aerial view following construction. Figure 15 provides a closeup view of the intersection.

© 2018 Google® Earth™.
Note: MN–65 runs north–south and 157th Avenue NE runs east–west, north at top.

Figure 13. Photo. MN–65 at 157th Avenue intersection, before condition.[32]

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 157th Avenue NE runs east–west, north at right.

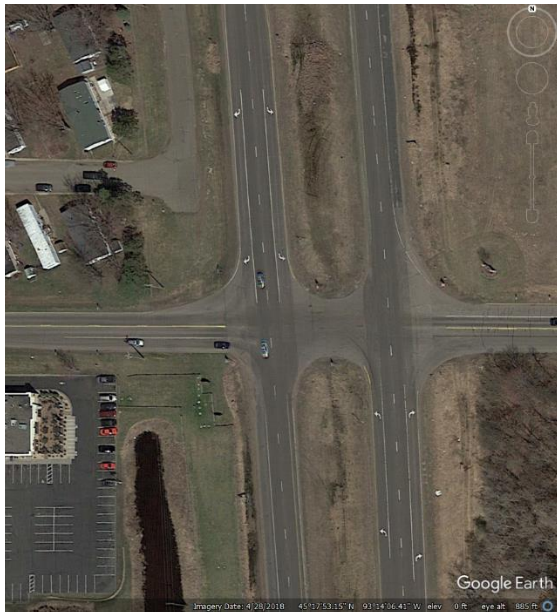
Figure 14. Photo. MN–65 at 157th Avenue intersection, after condition in Ham Lake, MN.[32]

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 157th Avenue NE runs east–west, north at top.

Figure 15. Photo. MN–65 at 157th Avenue intersection, closeup of main intersection, after condition in Ham Lake, MN.(32)

MN–65 at 181st Avenue, NE, Unsignalized RCUT

The RCUT constructed at the intersection of MN–65 and 181st Avenue NE (also known as Baltimore Street NE) is located in Ham Lake, MN. Figure 16 shows an aerial view of the intersection in the before condition, figure 17 shows an overview of the RCUT in the after condition, and figure 18 provides a closeup of the main intersection. MnDOT designed the project, and construction occurred in 2019.



© 2018 Google® Earth™.
Note: MN–65 runs north–south and 181st Avenue NE runs east–west, north at top.

Figure 16. Photo. MN–65 at 181st Avenue intersection, before condition in Ham Lake, MN.(32)

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 181st Avenue NE runs east–west, north at right.

Figure 17. Photo. MN–65 at 181st Avenue intersection, overview of main intersection and U-turns, after condition in Ham Lake, MN.(32)

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 181st Avenue NE runs east–west, north at top.

Figure 18. Photo. MN–65 at 181st Avenue intersection, closeup of main intersection, after condition.[32]

MN–65 at 187th Lane, Unsignalized RCUT

The intersection of MN–65 and 187th Lane NE is another intersection in a series of RCUTs located along MN–65 in Minnesota. The corridor traverses through the cities of Ham Lake, MN, and East Bethel, MN. Figure 19 is an aerial view of the intersection in the before condition, figure 20 is an overview of the RCUT in the after condition, and figure 21 provides a closeup of the main intersection.

© 2018 Google® Earth™.
Note: MN–65 runs north–south and 187th Lane NE runs east–west, north at top.

Figure 19. Photo. MN–65 at 187th Lane intersection, before condition in East Bethel, MN.[32]

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 187th Lane NE runs east–west, north at right.

Figure 20. Photo. MN–65 at 187th Lane intersection, overview of main intersection and U-turns, after condition in East Bethel, MN.(32)

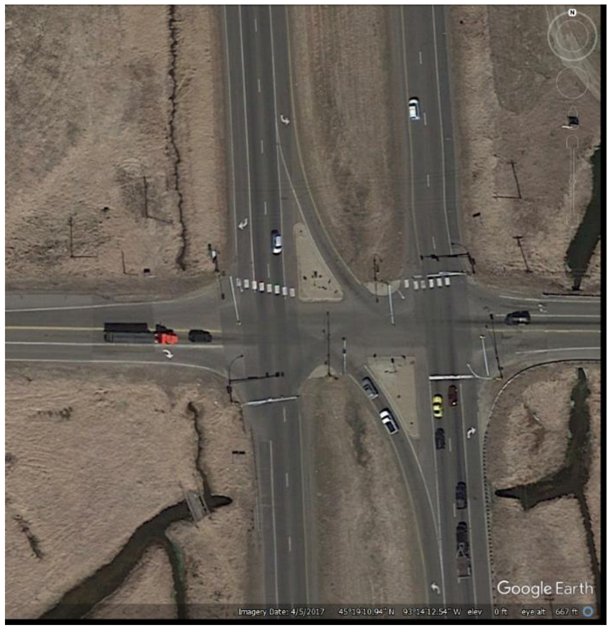
© 2021 Google® Earth™.
Note: MN–65 runs north–south and 187th Lane NE runs east–west, north at top.

Figure 21. Photo. MN–65 at 187th Lane intersection, closeup of main intersection, after condition in East Bethel, MN.(32)

MN–65 at Viking Boulevard NE, Signalized RCUT

The intersection of MN–65 at Viking Boulevard NE is in East Bethel, MN. Figure 22 shows the before period, Figure 23 shows the after period, and figure 24 is a closeup of the main intersection for the after period. MnDOT administered the design, and construction occurred from 2018 to 2019. This intersection was the only signalized RCUT included in the study of the

Minnesota corridor of MN–65. The intersection included unique features designed for future pedestrian and bicycle growth as well as adjacent terrain that will require substantial earthwork enhancements. The presence of the traffic signal will permit shorter queue wait times where needed. The following before and after photos provide additional information about the layout of this intersection.

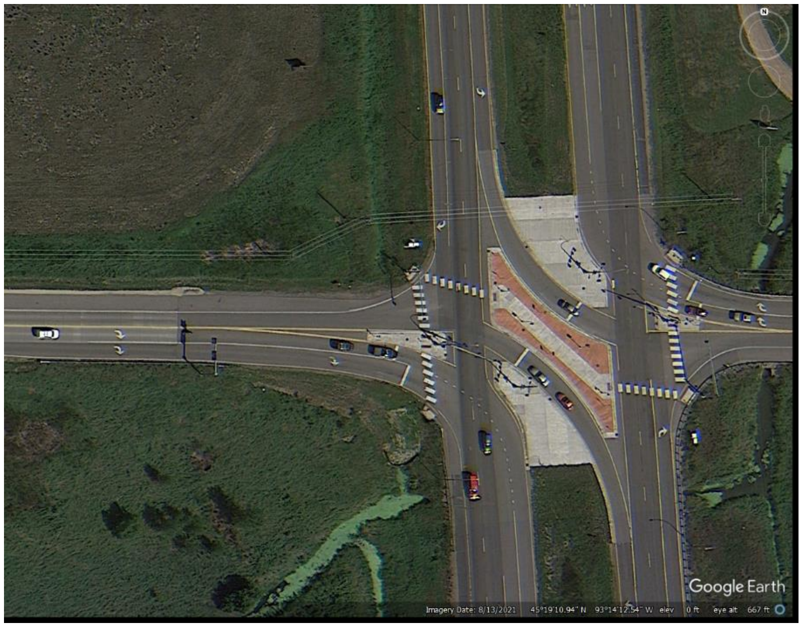


© 2017 Google® Earth™.
Note: MN–65 runs north–south and Viking Avenue NE runs east–west, north at top.

Figure 22. Photo. MN–65 at Viking Avenue intersection, before condition in East Bethel, MN.(32)

© 2021 Google® Earth™.
Note: MN–65 runs north–south and Viking Avenue NE runs east–west, north to right.

Figure 23. Photo. MN–65 at Viking Avenue intersection, overview of main intersection and U-turns, after condition in East Bethel, MN.(32)



© 2021 Google® Earth™.
Note: MN–65 runs north–south and Viking Avenue NE runs east–west, north at top.

Figure 24. Photo. MN–65 at Viking Avenue intersection, closeup of main intersection, after condition in East Bethel, MN.[32]

MN–65 at 209th Avenue NE, Traditional Two-Way, Stop-Controlled Intersection

The intersection of MN–65 and 209th Avenue NE was included in the study as an untreated traditional intersection that is located just north of the RCUT corridor. Figure 25 is an aerial view of the existing untreated intersection. The location of the project is within the city limits of East Bethel, MN. This intersection has stop signs on the 209th Avenue NE approaches. Through traffic on MN–65 does not stop at the intersection. This intersection configuration represents an untreated site for comparison purposes to the previously treated site at 181st Avenue at MN–65.

© 2021 Google® Earth™.
Note: MN–65 runs north–south and 209th Avenue NE runs east–west.

Figure 25. Photo. MN–65 at 209th Avenue NE intersection, untreated condition (two-way, stop-control traditional intersection) in East Bethel, MN.(32)

Displaced Left Interchange in San Antonio, TX

The research team acquired data located at the interchange of Texas State Highway 16 (SH–16), also known as Bandera Road, and West Loop 1604 Access Road (also known as East Charles William Anderson Loop). This study site is in San Antonio, Bexar County, TX. Figure 26 shows an aerial view of the SH–16 (Bandera Road) at West Loop 1604 Access Road interchange. The at-grade intersections to be included in this study are the intersections between SH–16 (Bandera Road) and the West Loop 1604 Access Road intersections on each side of the interchange. West Loop 1604 Access Road is constructed on structures over SH–16 (Bandera Road), and SH–16 (Bandera Road) is constructed at grade. For this application, the Texas turnaround (also referred to as the Texas U-turn) space is converted to accommodate the displaced left traffic.

© 2021 Google® Earth™.

Figure 26. Aerial Photograph. SH–16 at Loop 1604 in San Antonio, TX, after condition.[(32)]

Virginia DLT Intersections

The study included two Virginia intersections:

- Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165), Norfolk, VA.
- Indian River Road at Kempsville Road, Virginia Beach, VA.

Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165), Norfolk, VA, Intersection

The Norfolk, VA, intersection included DLTs on the north and south approaches. The research team acquired after data at the intersection of Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165) in Norfolk, VA, following construction of a DLT on the north and south approaches. The east and west approaches retain a more traditional configuration; before data were collected in 2015. Figure 27 is an aerial view of the intersection in the before condition, figure 28 is an overview of the DLTs and MUTs in the after condition, and figure 29 provides a closeup of the main intersection. The project construction began in August 2016 and was completed in July 2018.

© 2015 Google® Earth™.
Note: Military Highway runs east–west and Northampton Boulevard (U.S. 13 at VA SR–165) runs north–south, north at top.

Figure 27. Photo. Military Highway at Northampton Boulevard intersection, before condition (photo date November 19, 2017) in Norfolk, VA.[32]

© 2019 Google® Earth™.
Note: Military Highway runs east–west and Northampton Boulevard (U.S. 13 at VA SR–165) runs north–south, north at top.

Figure 28. Photo. Military Highway at Northampton Boulevard intersection, overview of main intersection and U-turns, after condition in Norfolk, VA.[32]

© 2019 Google® Earth™.
Note: Military Highway runs east–west and Northampton Boulevard (U.S. 13 at VA SR–165) runs north–south, north at top.

Figure 29. Photo. Military Highway at Northampton Boulevard intersection, closeup of main intersection, after condition in Norfolk, VA.(32)

Virginia Beach, VA, Hybrid Intersection (DLT and MUT)

Figure 30 is an aerial view of the intersection in the before condition, figure 31 is an overview of the DLTs in the after condition, and figure 32 provides a closeup of the newly constructed Kempsville Road at Indian River Road intersection. The facility at this location is owned by the city of Virginia Beach, VA, and the Virginia DOT has worked closely with the city on this effort. The after design is a hybrid intersection with DLTs on the north and south approaches and MUTs on the east and west approaches.

© 2015 Google® Earth™.
Note: Indian River Road runs southeast–northwest and Kempsville Road runs northeast–southwest, north at top.

Figure 30. Photo. Indian River Road at Kempsville Road intersection, before condition (photo date November 10, 2015) in Virginia Beach, VA.(32)

© 2021 Google® Earth™.
Note: Indian River Road runs southeast–northwest and Kempsville Road runs northeast–southwest, north at top.

Figure 31. Photo. Indian River Road at Kempsville Road intersection, overview of main intersection and U-turns, after condition in Virginia Beach, VA.(32)

© 2021 Google® Earth™.
Note: Indian River Road runs southeast–northwest and Kempsville Road runs northeast–southwest, north at top.

Figure 32. Photo. Indian River Road at Kempsville Road intersection, closeup of main intersection, after condition in Virginia Beach, VA.(32)

SUMMARY

This chapter identified the intersections evaluated in this study. Originally the team had 12 before sites, but 3 of the sites experienced a significant delay in construction. Consequently, only nine of the intersections include a before-after pairing. The remaining three sites have data collected for the after period in another location that can be used for cross-sectional comparisons or simulation scenarios. Chapter 4 summarizes the data collection, and chapter 5 summarizes the operational analysis.

CHAPTER 4. STUDY METHODOLOGY

The data collection techniques used in this study began with informational preliminary phone calls to determine the status of potential projects and ultimately identify candidate sites that met the selection criteria. For each field data collection trip, the research team:

- Conducted a web conference with FHWA to review field data collection details before the trip, for example:
    - Methods to be used.
    - Team members who were task leaders.
    - Team members who could lead the data collection task.
    - Equipment and crew allocated.
    - Dates (planned and backup dates) for data collection.
    - Verification that data collection will occur during regular workdays while local schools are in session.
    - Verification that the intersection will not be affected by construction work zone, etc.
- Notified FHWA on identification of the final list of candidate sites. Team members also reached out to police and transportation agencies in the area to notify them that the data collection would be occurring.
- Provided FHWA with the reduced field data for before and after conditions.

Where possible, the team tried to identify at least two sites for a specific type of alternative intersection with an expectation of data collection for 6 to 12 sites (so a range of 3 up to 6 unique alternative intersection configurations). Each site was evaluated two times (before construction and then again 3 mo after construction completion). Data collection occurred on Tuesday through Thursday during periods when schools are in session. These requirements collectively helped to capture typical traffic patterns.

DATA COLLECTION ELEMENTS

The research team performed an onsite investigation of each location and collected data including key geometric, operational, and safety features. This investigation provided baseline information to use to support analysis as the project progresses. The team developed a video record of the overall site conditions before and after construction to include with this investigation.

The research team can use several measures to determine the measures of effectiveness that are of interest. Additional potential variables include saturation flow or traffic volume increases, conflicts, travel time, and (where appropriate) pedestrian walking time. Table 3 and table 4 list the data elements used to gather the required site data and their companion collection methods.

Table 3. Physical site information used for general analysis.

Data Measure	Data Collection Method Used
Intersection geometry (e.g., angle of intersection, distance to nearby intersections)	Aerial photos and site inspection
Cross-sectional geometry (e.g., number, width, configuration of lanes)	Aerial photos or transportation agencies' databases
Horizontal geometry (e.g., left-turn lane length, spacing between movements)	Aerial photos or plan or profile sheets
Traffic control devices (signs, signals markings) including posted speed limit	Aerial photos, Google® Earth™ street view, or site inspection(32)
Roadside development, including pedestrian and bicycle accommodations and driveways	Google Earth street view or site inspection

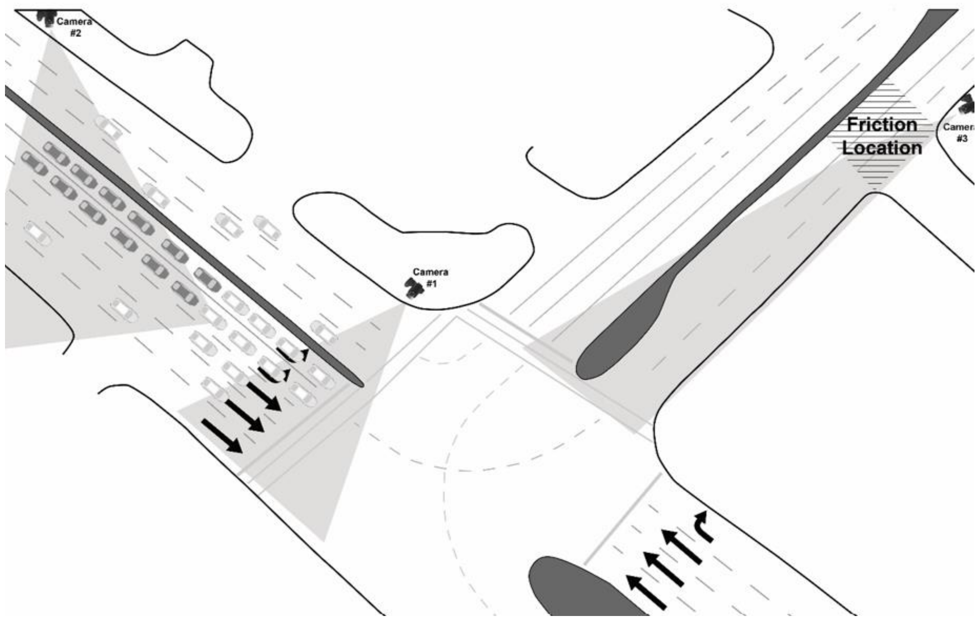
Table 4. Physical site information used for operational analysis.

Data Measure	Data Collection Method Used
Travel time (preselected origin-destination pairs) also useful for evaluating delay	Driving test vehicle (recording start and end time or recording second-by-second position)
Saturation flow rate or traffic volume	Video, onsite visual data collection
Queue length	Video, onsite visual data collection supplemented with video review
U-turning vehicles or other operations that could affect the site measures	Video
Pedestrian path trips through the intersection (at locations with full development and pedestrians present)	Field-walking studies

The research team mobilized within a few weeks of the contract award so that the two Virginia sites could be included in the study before construction beginning. The research team anticipated that video data would be essential for this data collection effort. From this video data, the team planned to determine several measures of interest, such as queue length, traffic volumes, etc. The team also observed the video data to help identify locations where drivers were incorrectly using the facility or appeared to make abrupt maneuvers as though making an unexpected decision. The vehicle type information was also acquired, but ultimately this information was not a significant factor in the study as the study sites either had wide lanes or few trucks present.

Camera Configuration

A typical camera setup is illustrated in figure 33. This camera placement can help provide information on throughput for the through movement. However, to obtain the queue for the through movement, the team had to place an additional camera. The left-turn lane queue could hide the through movement queue unless camera 2 was mounted high enough to record over the left-turn queue. The team used flexible pole-mounted tripods and portable cameras. The tripods were mounted to a pole, tree, or fence located near the intersection—in most cases several feet in the air. The final number of video cameras needed at a site depended on the mounting heights for each camera (e.g., whether two cameras are needed to obtain the left and through queues, or whether one can capture the view), along with the distance from the road. For long queues, multiple cameras were needed.



Source: FHWA.

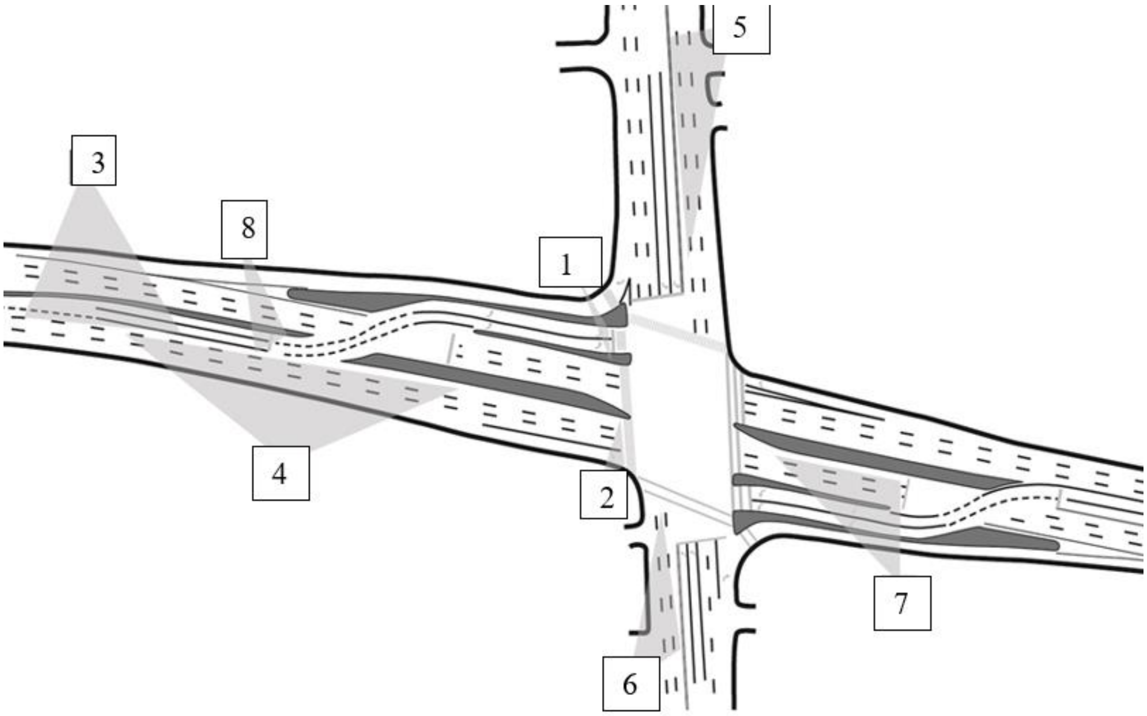
Figure 33. Illustration. Example camera setup for obtaining left-turn delay and saturation flow.

Table 5 lists the data generally collected by each video camera (excluding cameras 5–8). The shaded triangles in figure 33 illustrate the general areas recorded; however, the actual camera coverage was uniquely determined for each specific site.

Table 5. Data recorded by video cameras 1–4 and 9 (example intersection orientation).

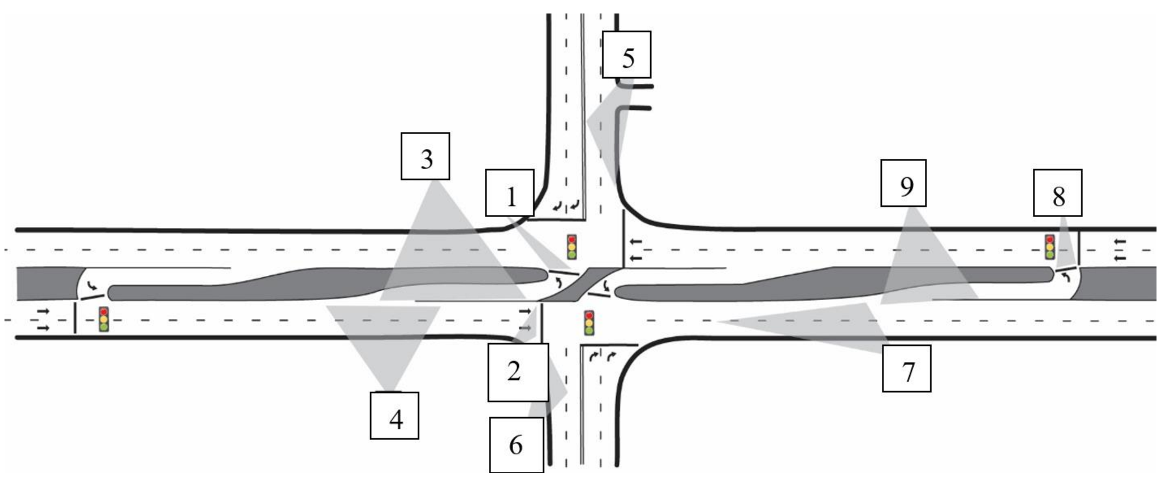
Camera Number	Traditional intersection, DLT, RCUT
1	Left turn: volume (by type), saturation flow rate, conflicts (pedestrian, bicyclist, vehicle), opposing direction volume (by type)
2	Through: volume (by type), saturation flow rate, conflicts (pedestrian, bicyclist, vehicle)
3	Left turn: queue, may need additional cameras for this view depending on length of anticipated queue
4	Through: queue, time-in-queue delay, may need additional cameras for this view depending on length of anticipated queue
9	Not applicable to the traditional or DLT lanes, for restricted crossing, number of vehicles in U-turn lane

Figure 34 shows an example layout for a DLT intersection design, while figure 35 shows an example layout for a RCUT intersection. These figures are examples for a selection of one alternative intersection design. While the video cameras (each camera is represented by a number) collected valuable information, the video cameras were not the only source of data for this project. Team members executed travel time runs during peak and offpeak periods.



Source: FHWA.

Figure 34. Illustration. Example video camera setup for an EB approach at a DLT lane intersection.



Source: FHWA.

Figure 35. Illustration. Example video camera setup for an EB approach at a RCUT intersection.

DATA REDUCTION AND ANALYSIS

Traffic Volumes

To acquire the traffic volume data, the research team mounted cameras at each study site along various approaches of the intersection. Depending on the configuration of each intersection and the height of the cameras, the number of data collection cameras ranged from 8 to 22 cameras. The field volume data were recorded for more than 12 h to cover both the a.m. peak, p.m. peak, and evening peak with a goal of ultimately summarizing the traffic volume for the period extending from 7:00 a.m. to 7:00 p.m. at each site. After obtaining the data, the team counted individual vehicles. The reduced data were recorded based on specific time observed, and eventually aggregated to 15-min intervals.

Vehicle Queues

The research team used the recorded video data to measure the queue length for each lane of various movements at the study intersections. To do so, two types of queues were measured:

- For unsignalized intersections, per-minute queues: The maximum number of vehicles in the queue was counted per lane per movement for each 1-min interval within the a.m. peak and p.m. peak periods.

- For signalized intersections, cycle queues: The maximum queue for each signal cycle was counted for each lane of each movement. The availability of the maximum number of vehicles in the queues for each signal cycle for the 12-h study period provides the opportunity to generate cumulative distribution curves. These curves can be used to identify the percentage of the observations that reflect that length of the queue (in number of vehicles) or less. The curves can be compared between the before period and the after period. The team anticipated that long queues will be less likely in the after period compared to the before period, and that the installation of the alternative intersection design is associated with shorter queues. The advantage of having data for a 12-h period compared to only the peak hour is that this approach captures those situations when long queues exist for more than just a single hour.

Travel Times

To assess the travel time performance of the intersection, the research team conducted field-measured, travel-time studies, with a primary focus on the through and left-turn maneuvers for locations with modified left-turn operations. To collect the travel time data, the research team used the floating car method along with a Global Positioning System unit. One team member was driving a vehicle, and the other team member was marking the predefined start point and end point.

In many cases, the study sites included substantial congestion (particularly during the before condition). When this constrained operational condition occurred, the number of travel time runs were restricted while the data collection team sat in long queues, so the research team used the individual videos to track vehicles through the intersection and develop a travel time using this alternative approach.

Pedestrian Walking Path

One of the research team's objectives was determining how pedestrians are affected by the alternative intersections. To make this comparison, the pedestrian travel times were measured for before and after conditions. The team defined origin and destination points for each approach at the intersections. Two different team members were involved in the pedestrian travel time measurement to consider various walking speeds. Each member measured the distance that was exposed and not exposed to the traffic, as well as the total travel time from an origin to a destination. Each movement was measured multiple times to calculate and present the average pedestrian travel time values. For rural locations, like the Minnesota sites, the walking path was not measured due to the high-speed conditions and lack of appropriate bicycle and pedestrian facilities.

CHAPTER 5. OPERATIONAL ANALYSIS RESULTS

Chapter 5 reviews the operational analysis, based on performance measures, for the individual intersection or alternative roadway elements. To assess the performance of an intersection that incorporates innovative techniques (such as indirect left turns) that is replacing a traditional intersection, understanding the reason that the agency needed to construct the alternative configuration is helpful. For example, if the goal is relieving congestion on the left-turn maneuver, then left-turn travel times should ideally be lower during the after period than during the before period. In some cases, the primary reason for the intersection modification is enhancing safety by altering the trajectories of the vehicles at the intersections. If possible, considering how the design will influence bicycle and pedestrian traffic at each location is also imperative. These examples demonstrate that these intersections are not a one-size-fits-all solution and that the sponsoring transportation agency should assess a variety of candidate performance measures.

ARIZONA SITES

The operational performance for the Arizona sites is summarized in the following sections and in table 6. Three of the intersections are located along the Grant Road corridor. Two of the sites have before-after data (Grant Road at First Avenue and Grant Road at Stone Avenue), and the third intersection only has after data (Grant Road at Oracle Road North). One additional intersection, located at Kolb Road and Valencia Road, is a unique innovative intersection with an indirect left turn as well as a quadrant-type connector.

Table 6. Arizona sites geometric configurations for before and after periods.

Intersection	Before-Period Configuration	After-Period Configuration
Grant Road at First Avenue	Traditional signalized	Signalized with indirect left turn on major corridor via MUT (known as ThruU in Arizona)
Grant Road at Stone Avenue	Traditional signalized	Signalized with indirect left turn on major corridor via MUT (known as ThruU in Arizona)
Grant Road at Oracle Road North	Data not collected because site was added during the after period due to loss of Texas RCUT sites	Signalized with indirect left turn on major corridor via MUT (known as ThruU in Arizona)
Valencia Road at Kolb Road	Traditional signalized	Innovative intersection with indirect left turn and quadrant-type treatment

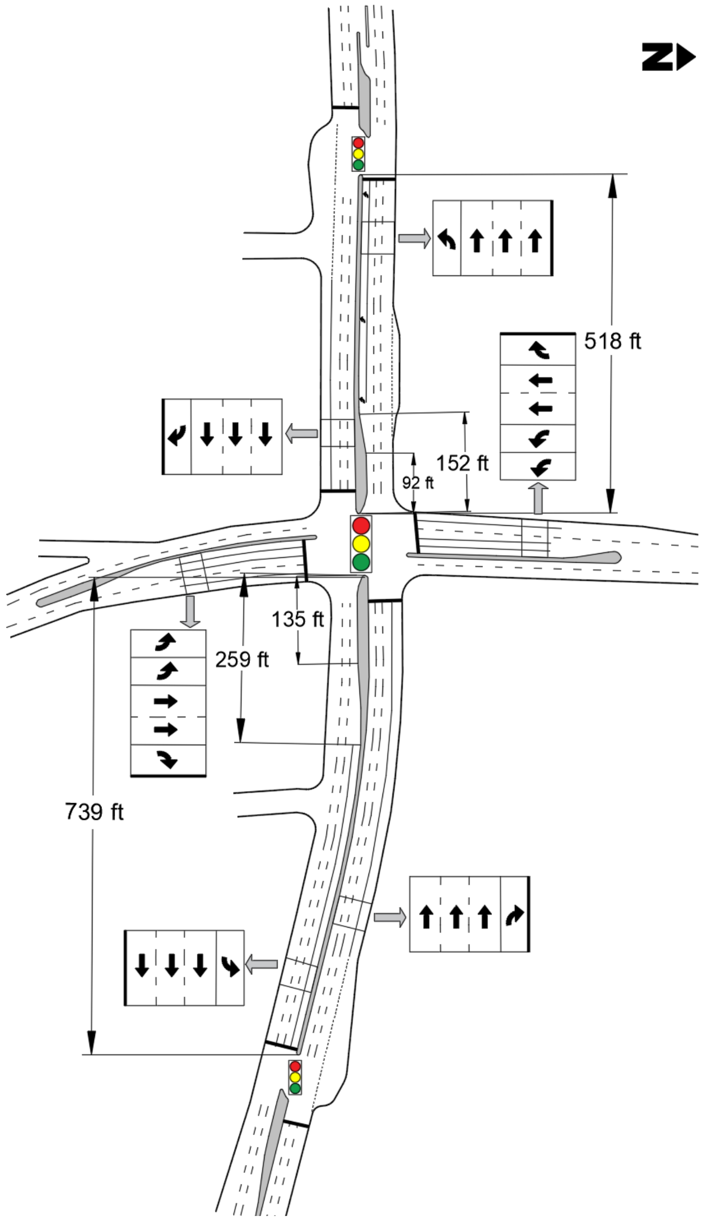
Arizona Signalized Main Intersection with MUTs (also Known as ThruU)

The three intersections located along the Grant Road corridor are in fully developed urban regions where left-turn maneuvers at the signalized intersections on the major road previously introduced significant delay. The Oracle Road North intersection was converted from a traditional signalized intersection to include signalized indirect left turns during an earlier phase of construction that preceded the data collection efforts for this project. In discussions with Tucson, AZ, stakeholders, this type of intersection has been referred to as DLT, an indirect left-turn intersection, a ThruU, or a modified RCUT. For consistency, this report uses the term signalized main intersection with MUTs. These sites differ from the Minnesota RCUTs because these sites use a passive prohibition of the main road left turn, rather than a restricted prohibition of the main road left turn. In other words, the left-turn maneuver from the major to the minor road is prohibited at the core intersection and is enforced using regulatory signage instead of a raised island. Left turns are still permitted from the minor to the major road at the signalized intersections. By contrast, the RCUT configuration physically restricts left turns from minor roads, as previously observed at the Minnesota study locations.

General Site Characteristics

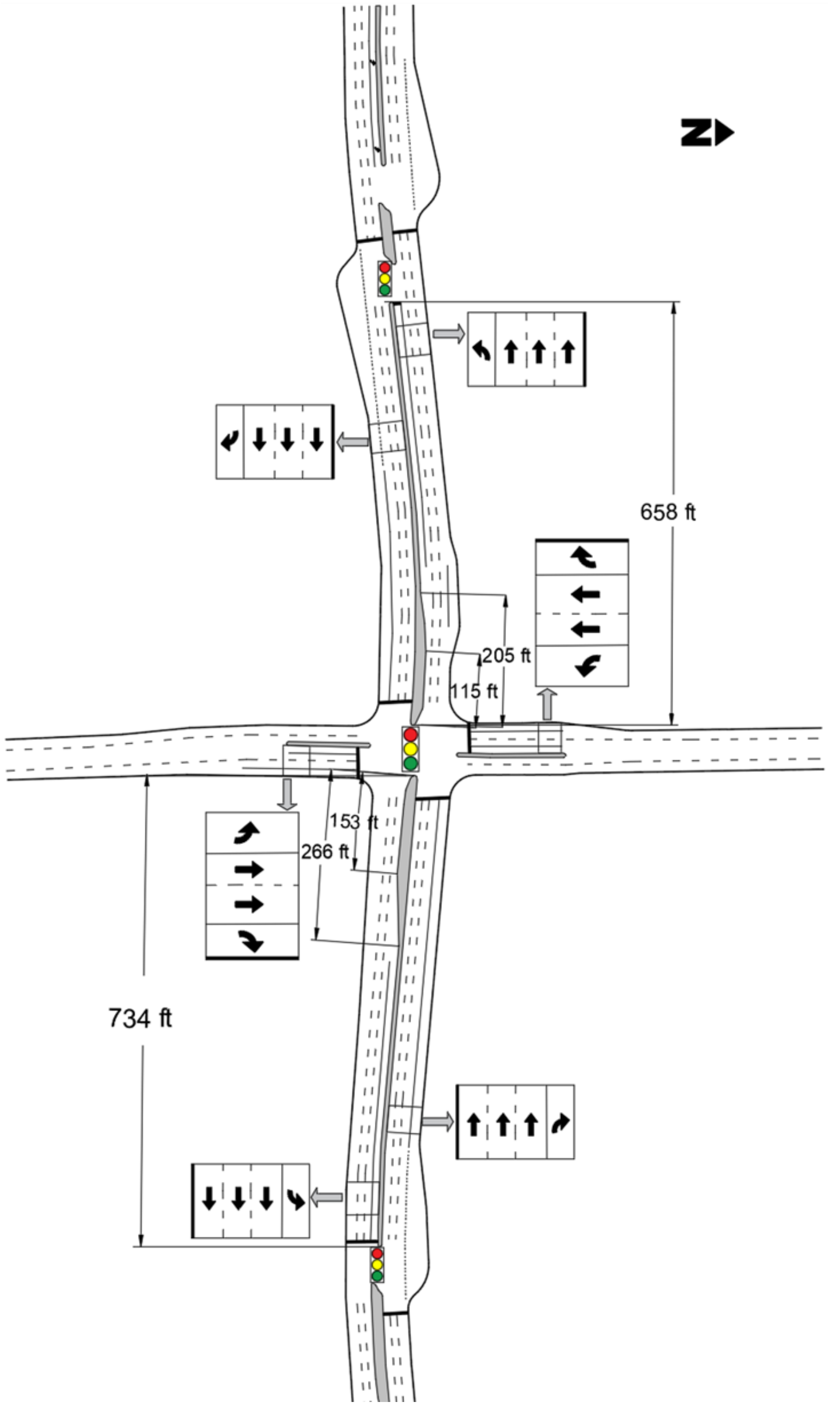
The main approach legs for the Grant Road corridor are the EB and westbound (WB) direction. These intersections are in Tucson, AZ, and are positioned to the east of a major freeway commuting corridor. All three intersections are signalized, and this Grant Road corridor is undergoing a phased reconstruction effort. The MUT locations associated with these three intersections are located upstream and downstream of the Grant Road study intersections. The intersection of Grant Road and Oracle Road North began construction before the data collection efforts for this study, but after data for this intersection have been included for comparison purposes where appropriate.

Figure 36, figure 37, and figure 38 provide a plan view of the corridor designs at Grant Road with First Avenue, Stone Avenue, and Oracle Road North, respectively. The U-turn locations are also characterized with the presence of a loon (additional pavement that is outside of the normal travel lanes and specifically designed to help accommodate U-turn maneuvers for large vehicles), which is common when the median separating the travel directions is relatively narrow. The plan view schematics show the placement of these features for the three Grant Road locations, depicting the distance from the right-turn intersection projected curb to the stop bar located at the associated upstream and downstream U-turns. For example, these dimensions for Grant Road at First Avenue are 739 ft for the WB approach and 518 ft for the EB approach.



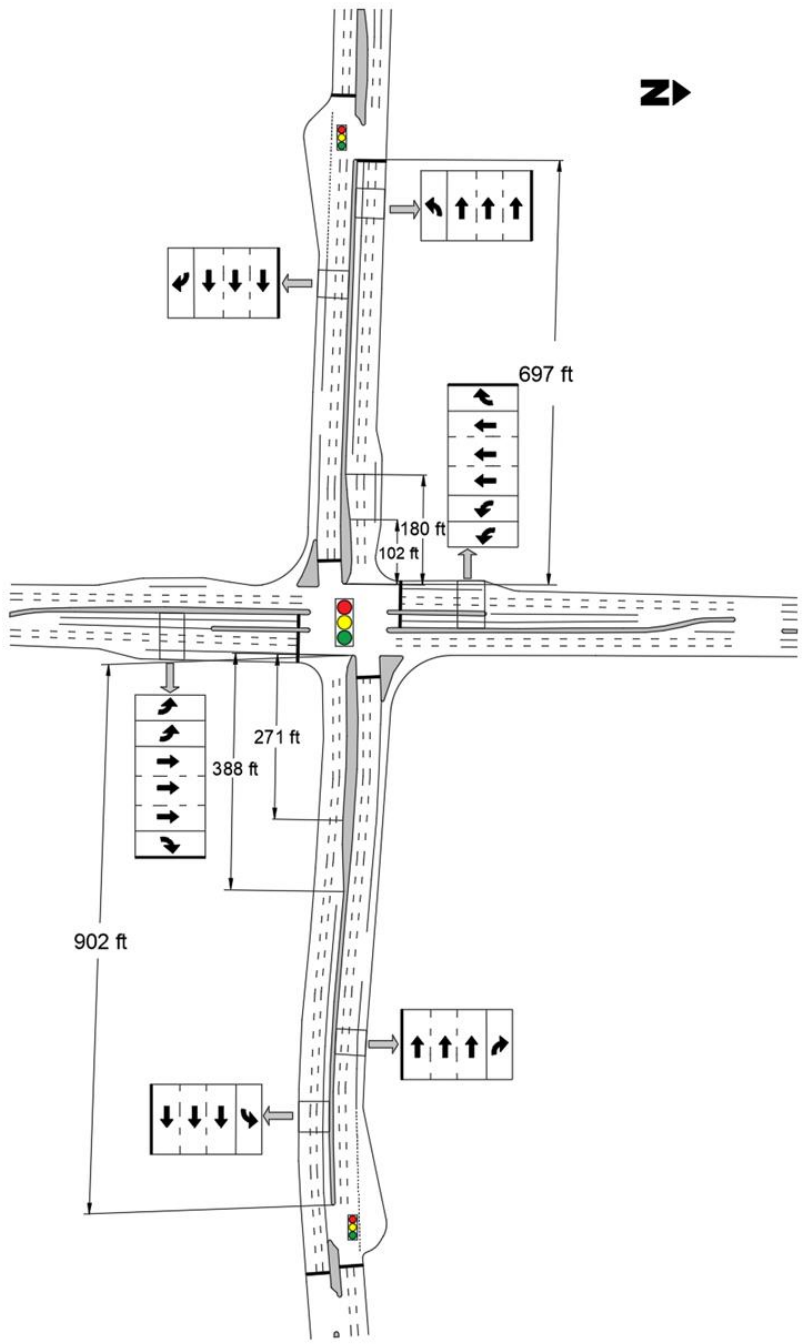
Source: FHWA.

Figure 36. Illustration. Grant Road at First Avenue signalized main intersection with MUTs (also known as ThruU in Tucson, AZ).



Source: FHWA.

Figure 37. Illustration. Grant Road at Stone Avenue signalized main intersection with MUTs (also known as ThruU in Tucson, AZ).



Source: FHWA.

Figure 38. Illustration. Grant Road at Oracle Road North signalized main intersection with MUTs (also known as ThruU in Arizona).

Traffic Volumes

The construction of RLTCI is expected to help reduce left-turn crashes at the core intersection while also streamlining operations for traffic flow. Table 7 depicts the total before and after peak hour volumes observed at the Grant Road corridor during the a.m. and p.m. peak hours. The lane distributions for the after condition were previously depicted in figure 36, figure 37, and figure 38. Table 7 shows that higher volumes occurred during the p.m. peak along this corridor during both the before and after period. The evening peak was higher in the after period compared to the before period. The First Avenue and Stone Avenue intersections with Grant Road for the after traffic volumes were similar for the a.m. peak and consistently higher for the after p.m. peak.

The research team acquired 12 h of continuous traffic volume data at the study sites. The turning movement counts are provided in the appendix of this report.

Table 7. Peak hour volumes at signalized main intersections with MUTs (also known as ThruU) in Tucson, AZ.

Site	Analysis Period	Peak Hour a.m. Time	Peak Hour a.m. Total Volume (vehicles)	Peak Hour p.m. Time	Peak Hour p.m. Total Volume (vehicles)
Grant Road at First Avenue	Before	7:45–8:45 a.m.	4,114	4:45–5:45 p.m.	4,539
Grant Road at First Avenue	After	7:00–8:00 a.m.	3,998	4:00–5:00 p.m.	4,977
Grant Road at Oracle Road North	Before	NA	NA	NA	NA
Grant Road at Oracle Road North	After	7:30–8:30 a.m.	4,017	4:30–5:30 p.m.	5,029
Grant Road at Stone Avenue	Before	7:45–8:45 a.m.	3,527	4:30–5:30 p.m.	4,053
Grant Road at Stone Avenue	After	7:30–8:30 a.m.	3,755	4:00–5:00 p.m.	4,455

NA = data not available because before data were not collected for this comparison site that was identified and included in the study at the time of the after-data collection effort.

Vehicle Queues

Although converting the traditional intersection configuration to RLTCI along the Grant Road corridor is expected to have a positive influence on both operational and safety performance, this treatment does require major road left-turning traffic to go straight at the main intersection, U-turn downstream, and then turn right onto the minor road. The remaining maneuvers can continue to operate in a manner similar to their original design. In addition to assessing the peak traffic volumes, another potential performance measure to consider at locations where left turns (before) and indirect left turns (after) occur is to compare the length of queue before and after the intersection modification. The research team acquired 12 h of queue data at the study sites.

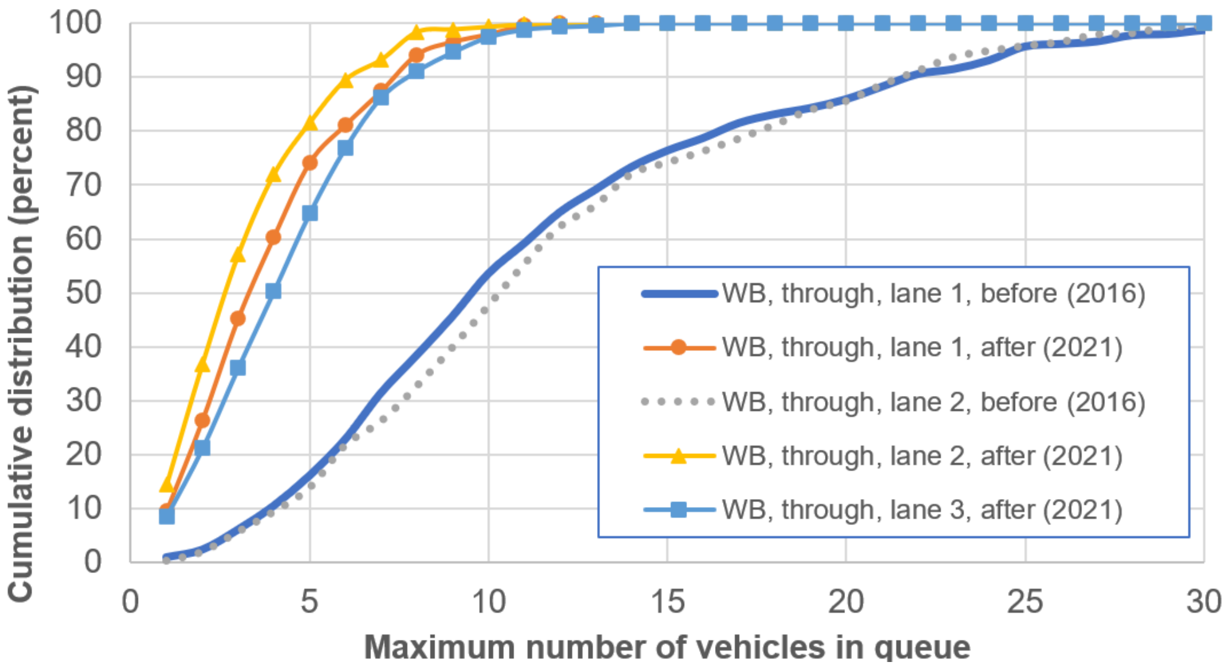
The queue data were reduced using the field video. The team counted the maximum length of queue (number of cars) for each signal cycle for each lane of each movement throughout the 12-h study period. To provide an overview of the typical length of queue, the research team determined the 85th percentile maximum queue by movement, direction, and lane. For example, if the value of 10 was identified, 85 percent of all the signal cycles observed had queues that were 10 vehicles or less. Stated in another manner, 15 percent of the signal cycles had queues that were greater than 10 vehicles. The change in queue lengths from before to after was determined. Table 8 provides the results for Grant Road at First Avenue. The greatest reductions in queue lengths were observed for through movements for the EB and WB approaches. For these approaches, the cumulative distribution was created and provided in figure 39 for WB through and figure 40 for EB through.

Table 8. Comparison of before and after queue lengths for Grant Road at First Avenue in Tucson, AZ.

Movement	Direction	Lane	Number of Vehicles Before (2016)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Left	EB	1	5.70	1.68	71
Left	NB	1	5.81	0.79	86
Left	SB	1	7.61	1.93	75
Left	WB	1	2.92	4.12	−41
Through	EB	1	16.80	7.54	55
Through	EB	2	16.20	5.53	66
Through	EB	3	NP	3.47	Lane added
Through	NB	1	8.75	2.52	71
Through	NB	2	8.92	7.89	12
Through	NB	3	NP	5.98	Lane added
Through	SB	1	9.91	2.41	76
Through	SB	2	9.44	5.90	38
Through	SB	3	NP	5.64	Lane added
Through	WB	1	18.50	5.62	70
Through	WB	2	18.64	4.43	76
Through	WB	3	NP	5.87	Lane added

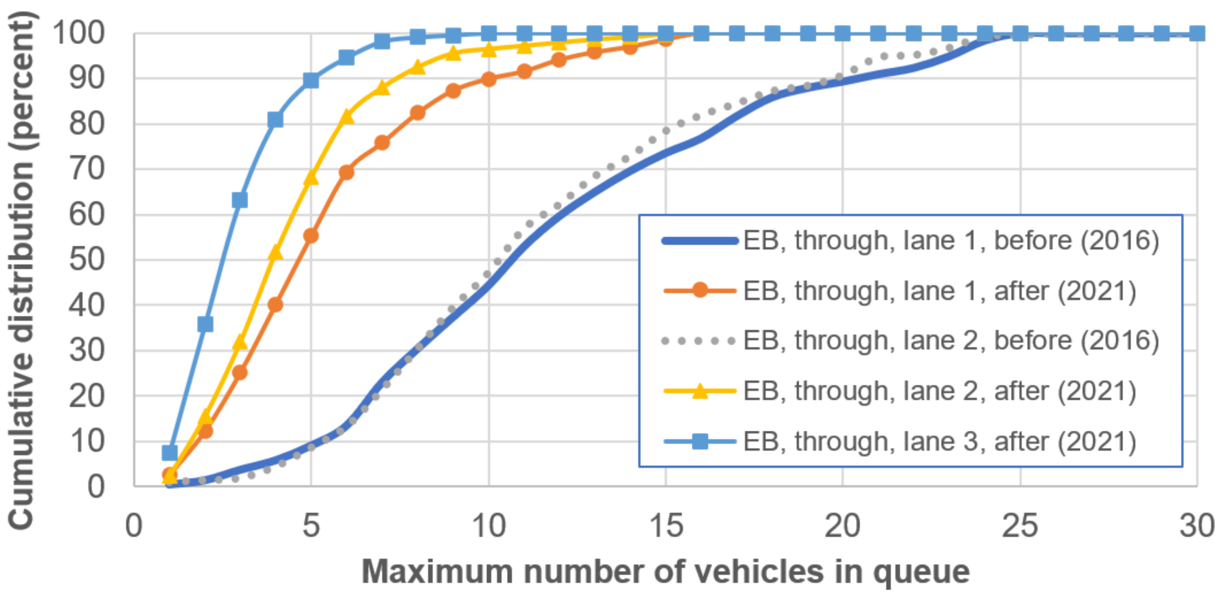
NP = lane is not present in the before period.

Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).



Source: FHWA.

Figure 39. Graph. Distribution of maximum queue length for Grant Road at First Avenue WB through by period and lane.



Source: FHWA.

Figure 40. Graph. Distribution of maximum queue length for Grant Road at First Avenue EB through by period and lane.

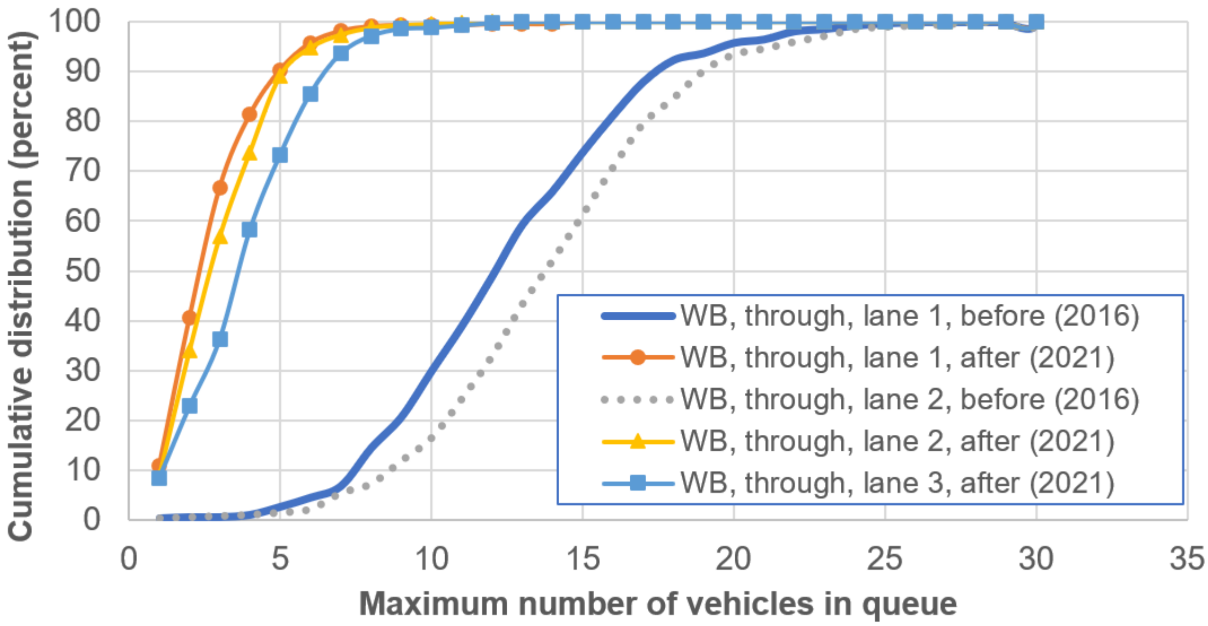
The queue length for the EB left turns in the after period was shorter; however, the queue length for WB left turns was longer. When comparing RLTCI and contrasting for the before and after condition, the measurement points for the queues are different, with the before values measured from the original EB left-turn lanes and the after values measured from the revised U-turn locations. In both cases, the change was about three vehicles, which is much shorter compared to the queues for the before period for the WB and EB through movement.

Table 9 provides the results for Grant Road at Stone Avenue for the queue length representing 85 percent of the signal cycles that had queue of that length or length. Similar to Grant Road at First Avenue, the longest queues in the before period were for EB and WB throughs. The reduction in queue lengths was 63 to 78 percent. For these approaches, the cumulative distribution was created and provided in figure 41 for WB through and figure 42 for EB through.

Table 9. Comparison of before and after queue lengths for Grant Road at Stone Avenue.

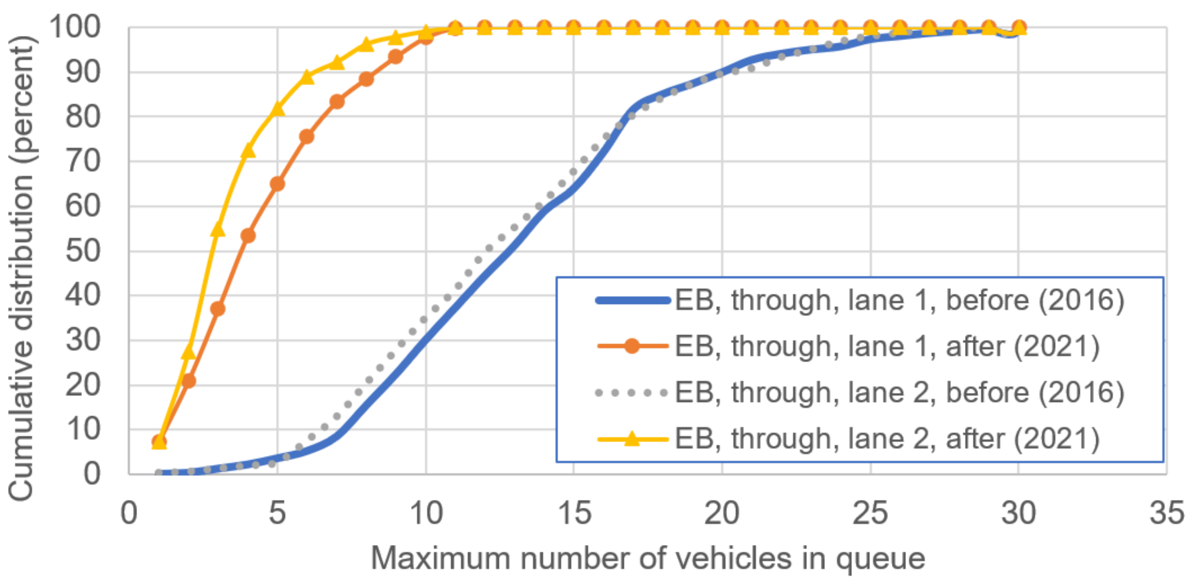
Movement	Direction	Lane	Number of Vehicles Before (2016)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Left	EB	1	5.29	2.45	54
Left	NB	1	2.99	2.30	23
Left	SB	1	4.04	2.85	29
Left	WB	1	4.71	1.83	61
Through	EB	1	16.96	6.33	63
Through	EB	2	17.19	4.44	74
Through	NB	1	7.95	4.10	48
Through	NB	2	8.08	4.29	47
Through	SB	1	8.16	2.90	64
Through	SB	2	8.16	3.16	61
Through	WB	1	15.55	3.41	78
Through	WB	2	17.04	3.73	78
Through	WB	3	NP	4.95	Lane added

Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).



Source: FHWA.

Figure 41. Graph. Distribution of maximum queue length for Grant Road at First Avenue WB through by period and lane.



Source: FHWA.

Figure 42. Graph. Distribution of maximum queue length for Grant Road at First Avenue EB through by period and lane.

The queue lengths for all left-turn approaches were shorter in the after period compared to the before period. Like Grant Road at First Avenue, the change was about three vehicles or less, which is much shorter compared to the queues for the before period for the WB and EB through movement.

Table 10 provides the typical queue lengths for the three intersections along Grant Road. Although there are some differences between the intersections, overall the queue lengths are similar.

Table 10. Comparison of queue lengths for three intersections along Grant Road after the treatment is installed.

Movement	Direction	Lane	Number of Vehicles Grant Road at First Avenue	Number of Vehicles Grant Road at Oracle Road North	Number of Vehicles Grant Road at Stone Avenue
Left	EB	1	1.68	1.64	2.45
Left	NB	1	0.79	1.75	2.30
Left	NB	2	NP	1.74	NP
Left	SB	1	1.93	2.99	2.85
Left	SB	2	NP	5.22	NP
Left	WB	1	4.12	1.70	1.83
Through	EB	1	7.54	8.68	6.33
Through	EB	2	5.53	5.87	4.44
Through	EB	3	3.47	4.57	NP
Through	NB	1	2.52	4.91	4.10
Through	NB	2	7.89	5.04	4.29
Through	NB	3	5.98	2.66	NP
Through	SB	1	2.41	5.74	2.90
Through	SB	2	5.90	6.73	3.16
Through	SB	3	5.64	2.41	NP
Through	WB	1	5.62	5.80	3.41
Through	WB	2	4.43	5.78	3.73
Through	WB	3	5.87	4.76	4.95

Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).

Travel Times

In the treatment within the Grant Road corridor, the left turns from the major street at the intersection were moved to downstream U-turns. Therefore, the travel distance for the EB and WB left turns increased. This treatment can benefit other movements because the green signal time at the subject intersections that previously served the left-turn movement can be reallocated to other movements. As shown in table 11, the travel times for the EB and WB through movements decreased from the before period to the after period. Improvement in travel time was also experienced for most of the movements from the minor road, as shown in table 12. Unfortunately, requiring the left-turn movements from the major road to use the downstream

U-turns resulted in slightly longer travel times, as shown in table 13. Table 14 lists the start and end points by movement for the three intersections studied within the Grant Road corridor.

Table 11. Field-measured travel times along the major road for three intersections along Grant Road, through movements, in Arizona.

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent ***R***
First Avenue	EB through	a.m.	86	52	39
First Avenue	EB through	Off	81	51	37
First Avenue	EB through	p.m.	77	60	23
First Avenue	WB through	a.m.	82	51	38
First Avenue	WB through	Off	79	56	29
First Avenue	WB through	p.m.	135	62	54
Oracle Road North	EB through	a.m.	0	50	NA
Oracle Road North	EB through	Off	0	54	NA
Oracle Road North	EB through	p.m.	0	78	NA
Oracle Road North	WB through	a.m.	0	69	NA
Oracle Road North	WB through	Off	NA	NA	NA
Oracle Road North	WB through	p.m.	0	91	NA
Stone Avenue	EB through	a.m.	118	48	59
Stone Avenue	EB through	Off	83	58	31
Stone Avenue	EB through	p.m.	77	63	17
Stone Avenue	WB through	a.m.	73	61	17
Stone Avenue	WB through	Off	74	58	21
Stone Avenue	WB through	p.m.	150	74	51

B_TT(Ave) = average before travel time for all available runs; A_TT(Ave) = average after travel time for all available runs; Percent *R* = the percent change in travel time (when value is negative, travel time increased).

Table 12. Field-measured travel times along the minor road for three intersections along Grant Road in Tucson, AZ.

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
First Avenue	NB left	a.m.	81	70	13
First Avenue	NB left	Off	83	53	37
First Avenue	NB left	p.m.	118	60	49
First Avenue	NB through	a.m.	62	59	5
First Avenue	NB through	Off	72	69	4
First Avenue	NB through	p.m.	77	72	6
First Avenue	SB left	a.m.	102	54	47
First Avenue	SB left	Off	99	92	7
First Avenue	SB left	p.m.	106	72	32
First Avenue	SB through	a.m.	64	81	−26
First Avenue	SB through	Off	57	54	4
First Avenue	SB through	p.m.	59	58	2
Oracle Road North	NB left	a.m.	NA	66	NA
Oracle Road North	NB left	Off	NA	77	NA
Oracle Road North	NB left	p.m.	NA	59	NA
Oracle Road North	NB through	a.m.	NA	23	NA
Oracle Road North	NB through	Off	NA	58	NA
Oracle Road North	NB through	p.m.	NA	58	NA
Oracle Road North	SB left	a.m.	NA	76	NA
Oracle Road North	SB left	Off	NA	83	NA
Oracle Road North	SB left	p.m.	NA	93	NA
Oracle Road North	SB through	a.m.	NA	27	NA
Oracle Road North	SB through	Off	NA	49	NA
Oracle Road North	SB through	p.m.	NA	58	NA
Stone Avenue	NB left	a.m.	106	57	46

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
Stone Avenue	NB left	Off	98	93	4
Stone Avenue	NB left	p.m.	91	59	35
Stone Avenue	NB through	a.m.	76	48	36
Stone Avenue	NB through	Off	83	55	33
Stone Avenue	NB through	p.m.	76	52	32
Stone Avenue	SB left	a.m.	108	75	30
Stone Avenue	SB left	Off	108	69	36
Stone Avenue	SB left	p.m.	95	79	17
Stone Avenue	SB through	a.m.	40	58	−47
Stone Avenue	SB through	Off	45	61	−35
Stone Avenue	SB through	p.m.	43	69	−60

Table 13. Field-measured travel times along the major road for three intersections along Grant Road, left-turn movements in Tucson, AZ.

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
First Avenue	EB left	a.m.	71	88	−24
First Avenue	EB left	Off	92	112	−22
First Avenue	EB left	p.m.	128	153	−20
First Avenue	WB left	a.m.	67	117	−75
First Avenue	WB left	Off	85	86	−1
First Avenue	WB left	p.m.	98	120	−23
Stone Avenue	EB left	a.m.	63	87	−39
Stone Avenue	EB left	Off	84	129	−54
Stone Avenue	EB left	p.m.	100	104	−4
Stone Avenue	WB left	a.m.	93	89	5
Stone Avenue	WB left	Off	79	92	−17
Stone Avenue	WB left	p.m.	100	96	4

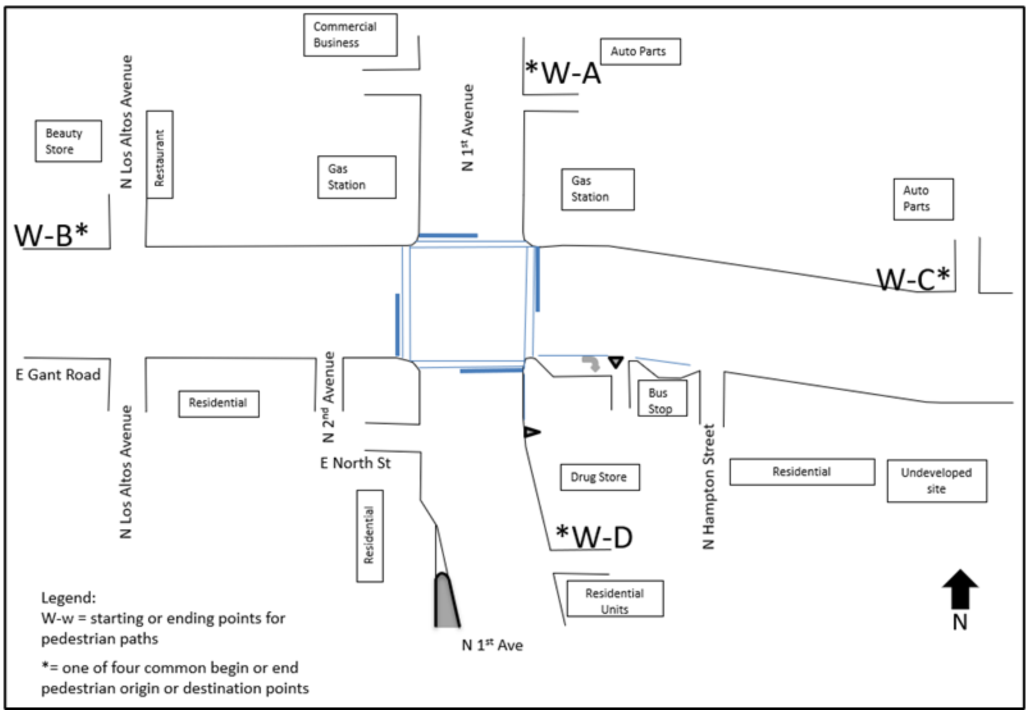
Table 14. Coordinates for start and end field-measured travel times for three intersections along Grant Road in Tucson, AZ.

Site	Movement	Start Point	End Point
First Avenue	EB left	32.250375, −110.963350	32.251271, −110.960948
First Avenue	EB through	32.250375, −110.963350	32.249937, −110.958127
First Avenue	NB left	32.249410, −110.960906	32.250375, −110.963350
First Avenue	NB through	32.249410, −110.960906	32.251271, −110.960948
First Avenue	SB left	32.251271, −110.960948	32.249937, −110.958127
First Avenue	SB through	32.251271, −110.960948	32.249410, −110.960906
First Avenue	WB left	32.249937, −110.958127	32.249410, −110.960906
First Avenue	WB through	32.249937, −110.958127	32.250375, −110.963350
Oracle Road North	EB left	32.250156, −110.982274	32.249232, −110.978037
Oracle Road North	EB through	32.250156, −110.982274	32.250277, −110.973466
Oracle Road North	NB left	32.249232, −110.978037	32.250156, −110.982274
Oracle Road North	NB through	32.249232, −110.978037	32.252135, −110.978091
Oracle Road North	SB left	32.252135, −110.978091	32.250277, −110.973466
Oracle Road North	SB through	32.252135, −110.978091	32.249232, −110.978037
Oracle Road North	WB left	32.250277, −110.973466	32.249232, −110.978037
Oracle Road North	WB through	32.250277, −110.973466	32.250156, −110.982274
Stone Avenue	EB left	32.250261, −110.974442	32.252161, −110.971933
Stone Avenue	EB through	32.250261, −110.974442	32.250347, −110.967656
Stone Avenue	NB left	32.249225, −110.971908	32.250261, −110.974442
Stone Avenue	NB through	32.249225, −110.971908	32.252161, −110.971933
Stone Avenue	SB left	32.252161, −110.971933	32.250347, −110.967656
Stone Avenue	SB through	32.252161, −110.971933	32.249225, −110.971908
Stone Avenue	WB left	32.250347, −110.967656	32.249225, −110.971908
Stone Avenue	WB through	32.250347, −110.967656	32.250261, −110.974442

Pedestrian Walking Paths

When weighing the various features of an alternative intersection, a common concern is how this change impacts pedestrian or bicycle access. For an isolated intersection, the question can be better framed by asking how the improvements influence pedestrian convenience, comfort, and walking time at the intersection. In some cases, the improvements could conceivably provide additional pedestrian refuge, while in other cases, the changes many lengthen the time and quality of access to local businesses by pedestrians. The research team conducted a pedestrian walking study at each Arizona site.

Figure 43 depicts the intersection plan view schematic for the Grant Road at Stone Avenue intersection. The research team conducted multiple walking tests before and after the intersection implementation project. The objective of the study was to measure normal walking time and distance from one local origin at the intersection to a destination. The research team members were asked to walk at a normal pace for them, obey all traffic laws, and select the route that was available at the time of the walk. For instance, a pedestrian might need to reach the opposite corner of an intersection. If the intersection is signalized, the selected path should be the route that has a green signal as the pedestrian approaches the corner. The research team members began and ended at common points. For the schematic shown in figure 43, the research team assigned data collectors (pedestrians) a starting point (shown with an asterisk) and three destination points (designated by a letter A ,B, C, or D). This process resulted in four potential origins or destination pairings: *W-A, *W-B, *W-C, and *W-D (where W = walk) and W-B* represents, as an example, a pedestrian path to and from W (in this case "A") to B, C, and D.



Source: FHWA.

Figure 43. Illustration. Pedestrian crossing points for Grant Road at First Avenue.

There are two primary goals for this study:

- Do not significantly increase the pedestrian walking distance or time.
- Minimize the amount of exposure between the pedestrian and active motor vehicles that share the same space.

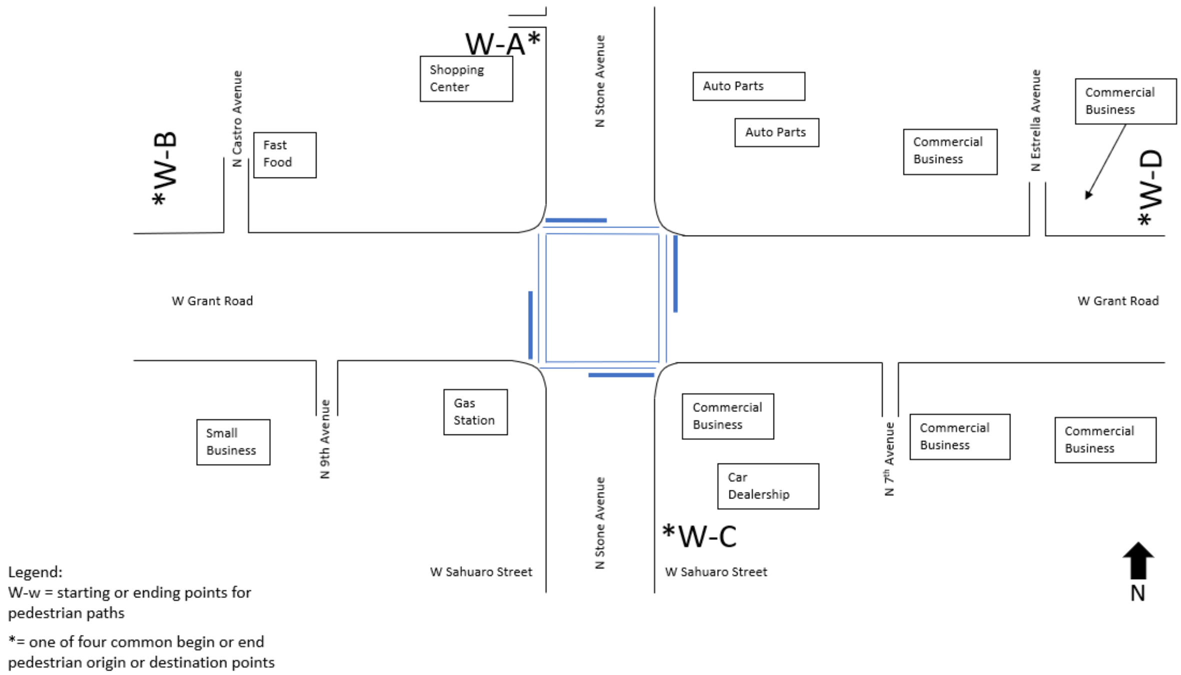
This improvement could be in the form of narrower driveways the pedestrian must cross. When pedestrians are rerouted from the more traditional access routes, pedestrians tend to engage in risky walking paths, rather than walk a substantial distance beyond their normal expectations.

Table 15 shows the before and after pedestrian walking times for each origin-destination pair. Table 15 shows that the improved intersection operations at Grant Road and First Avenue slightly but consistently improved the walking time in the after condition. For each maneuver, the researchers walked a minimum of 4 iterations, and in many cases 8 to 10 pedestrian paths. The distance traversed by the pedestrian also varied slightly from before to after conditions.

Table 15. Pedestrian crossing distances and times for Grant Road at First Avenue.

Origin	Destination	Distance Before Change (ft)	Average Walking Time Before Change (min:s)	Distance After Change (ft)	Average Walking Time After Change (min:s)	Change in Walking Time Travel Time (min:s)
W-A	W-D	747	3:46	735	3:07	−0.38
W-D	W-A	747	3:54	735	3:04	−0:50
W-A	W-B	1,230	5:11	1,200	4:30	−0:41
W-B	W-A	1,230	5:23	1,200	4:45	−0.38
W-A	W-C	1,301	5:38	1,372	4:50	−0:48
W-C	W-A	1,301	5:50	1,372	4:54	−0.56
W-B	W-C	1,911	7:50	2,043	7:24	−0.26
W-C	W-B	1,911	9:07	2,043	7:17	−1:50
W-B	W-D	1,356	7:12	1,415	6:07	−1:05
W-D	W-B	1,356	7:16	1,415	5:38	−1:50
W-C	W-D	1,419	7:28	1,483	5:46	−1:42
W-D	W-C	1,419	7:21	1,483	5:36	−1:45

The research team also conducted a pedestrian walking path analysis at the intersection of Grant Road and Stone Avenue, which is depicted in figure 44 and table 16. A decrease in walking travel time was experienced for the Grant Road at Stone Avenue intersection for all origin-destination pairs.



Source: FHWA.

Figure 44. Illustration. Pedestrian walking path study for Grant Road at Stone Avenue in Tucson, AZ.

Table 16. Pedestrian crossing distances and times for Grant Road at Stone Avenue in Tucson, AZ.

Origin	Destination	Distance Before Change (ft)	Average Walking Time Before Change (min:s)	Distance After Change (ft)	Average Walking Time After Change (min:s)	Change in Walking Time Travel Time (min:s)
W-A	W-B	1,318	6:57	1,523	5:50	−0:58
W-D	W-A	1,318	7:04	1,523	5:35	−1:29
W-A	W-B	1,094	5:22	997	3:58	−1:24
W-B	W-A	1,094	5:54	997	4:01	−1:53
W-A	W-C	787	5:21	752	3:34	−1:47
W-C	W-A	787	5:01	752	3:20	−1:41
W-B	W-C	1,280	7:37	1,186	5:05	−2.32
W-C	W-B	1,280	7:16	1,186	5:08	−2:08
W-B	W-D	1,812	9:34	1,958	7:56	−1:38
W-D	W-B	1,812	9:31	1,958	7:42	−1:49
W-C	W-D	1,343	6:14	1,504	5:51	−0:23
W-D	W-C	1,343	6:14	1,504	5:57	−0:17

Performance Measures Summary

The performance measures of the RLTCI at the signalized intersections within the Grant Road corridor were all positive. Table 17 summarizes the observed performance of the intersection improvements.

Table 17. Performance for Grant Road MUT (RLTCI or ThruU) intersections—Contrasting after to before.

Performance Measure	First Avenue	Stone Avenue
Queues, major road left turn	Reduced for one approach and increased for other approach	Reduced for both approaches
Queues, major and minor road through	Reduced for all approaches	Reduced for all approaches
Travel times	Reduced (improved)	Reduced (improved)
Pedestrian walking paths	Improved	Improved

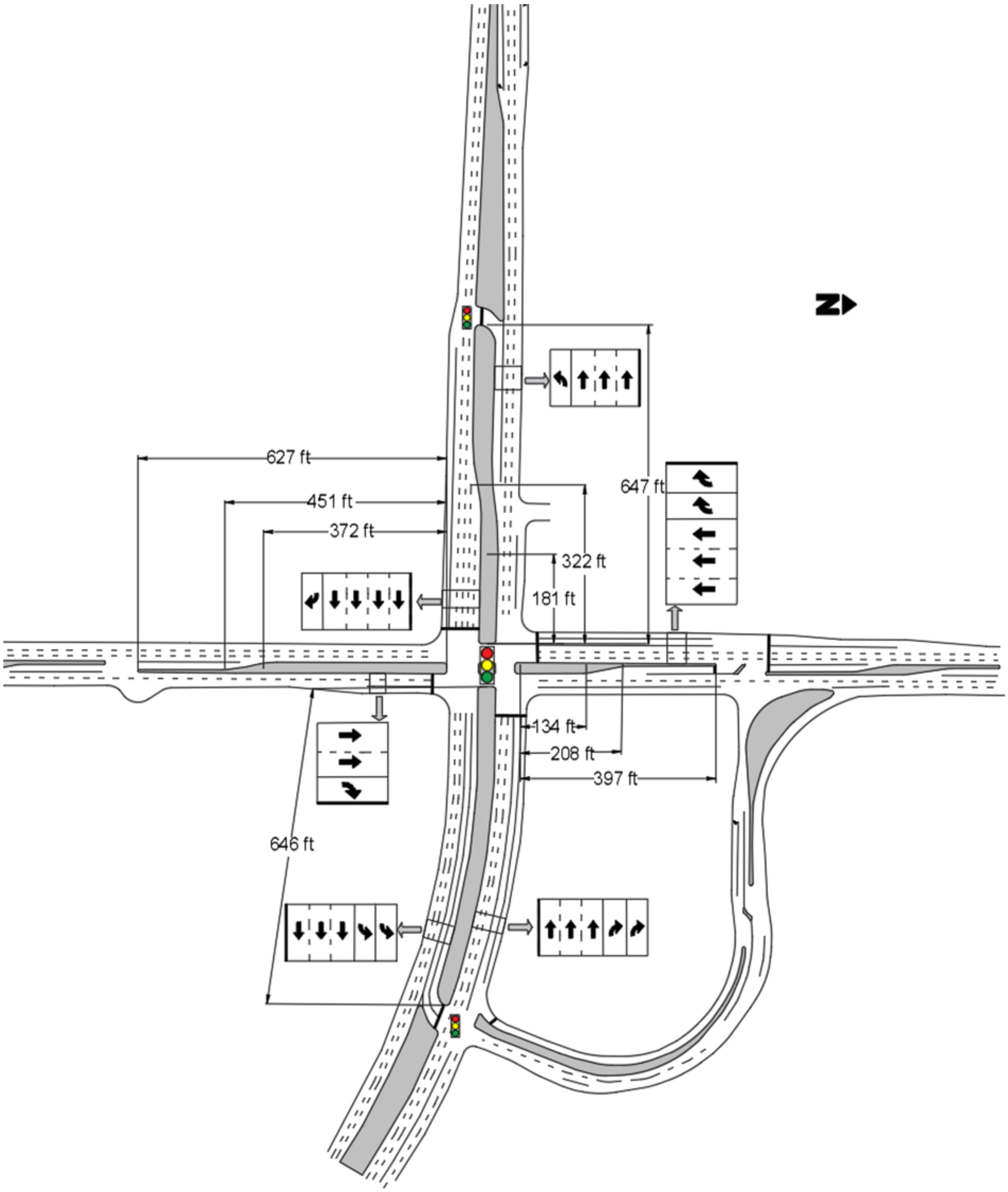
Note: The traffic volumes increased in the after period at the Grant Road at First Avenue intersection by 6 percent and at the Grant Road at Stone Avenue intersection by 9 percent.

Arizona Signalized Hybrid Intersection (Quadrant Road and MUTs), Valencia Road at Kolb Road

As noted earlier in this Chapter and further described in the Chapter 2 section titled East Valencia Road at South Kolb Road in Tucson, AZ, the research team studied one additional intersection in Tucson, AZ. This intersection of Valencia Road at Kolb Road is described in the following section.

General Site Characteristics

Figure 45 shows the intersection of Valencia Road and Kolb Road, which incorporates more than one alternative intersection feature. To better access regional higher volume commuter corridors, the local transportation agencies worked together to provide a quadrant-type maneuver, as well as signalized midblock U-turns on the EB and WB approaches. A south to north U-turn was also constructed on Kolb Road (just south of the core intersection). The agencies also coordinated the placement of traffic signals at the midblock U-turn locations, similar to the indirect left turn previously described for Minnesota and Arizona. Although this site has high peak hour volumes, the land use is currently relatively undeveloped. For that reason, the pedestrian path analysis is not included for this intersection.



Source: FHWA.

Figure 45. Illustration. Hybrid intersection for Valencia Road at Kolb Road in Tucson, AZ.

Traffic Volumes

The traffic volumes at the intersection of Valencia Road and Kolb Road are comparable between a.m. and p.m. peak (table 18).

Table 18. Peak hour volumes for Valencia Road at Kolb Road, Tucson, AZ.

Analysis Period	Peak Hour a.m. Time	Peak Hour a.m. Total Volume (vehicles)	Peak Hour p.m. Time	Peak Hour p.m. Total Volume (vehicles)
Before	7:00–8:00 a.m.	4,309	4:45–5:45 p.m.	4,235
After	7:00–8:00 a.m.	4,207	4:45–5:45 p.m.	4,537

SB and WB Vehicle Queues

Currently, the primary purpose of this intersection is to facilitate commuting traffic. For that reason, the addition of a quadrant type roadway and enhanced access to that facility should help improve operations at this location.

Table 19 provides the results for Valencia Road at Kolb Road for the queue length representing 85 percent of the signal cycles that had queues of that length or longer. The intersections along Grant Road had the longest queues for the through maneuvers. For Valencia Road at Kolb Road, the longest queues were associated with EB left turns, although EB through and SB left turns also have longer lengths compared to other movements.

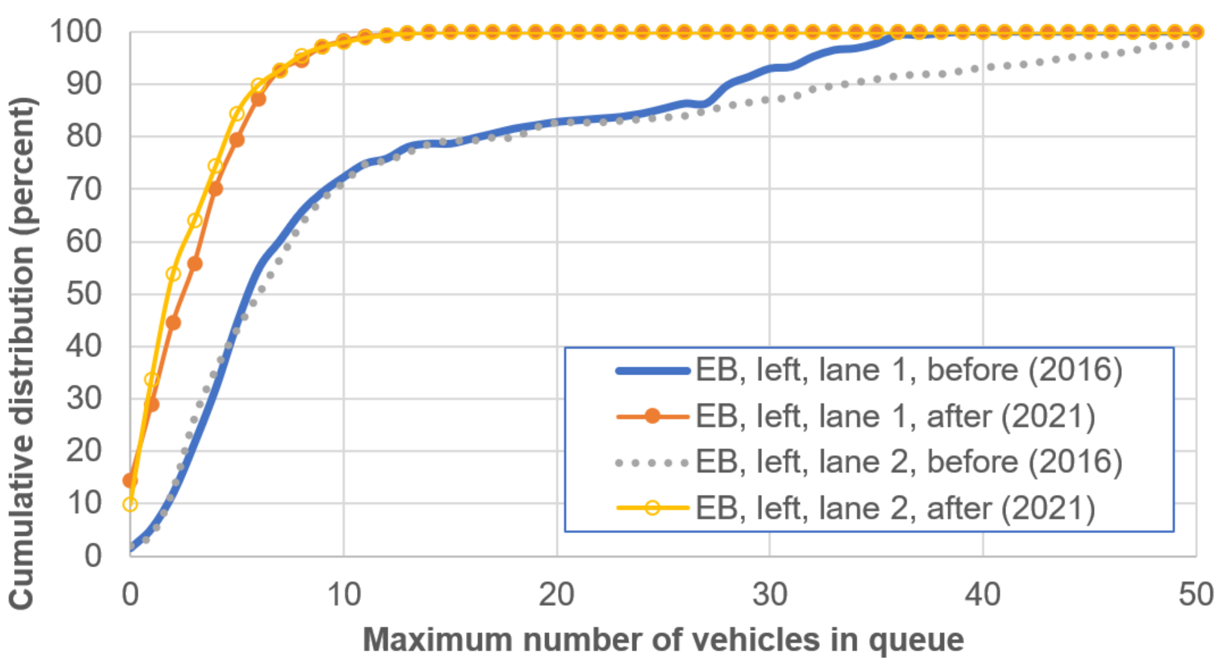
Table 19. Comparison of before and after queue lengths for Valencia Road at Kolb Road in Tucson, AZ.

Movement	Direction	Lane	Number of Vehicles Before (2016)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Left	EB	1	24.63	5.71	77
Left	EB	2	26.97	5.09	81
Left	NB	1	0.85	1.27	−49
Left	SB	1	15.99	2.53	84
Left	SB	2	NP	1.53	Lane added
Left	WB	1	1.83	0.85	53
Through	EB	1	18.90	4.74	75
Through	EB	2	NP	4.57	Lane added
Through	EB	3	NP	2.72	Lane added
Through	EB	4	NP	2.73	Lane added
Through	EB	5	NP	1.03	Lane added
Through	NB	1	9.97	5.98	40

Movement	Direction	Lane	Number of Vehicles Before (2016)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Through	NB	2	NP	4.33	Lane added
Through	SB	1	6.39	7.15	−12
Through	SB	2	5.22	4.20	19
Through	SB	3	NP	2.67	Lane added
Through	WB	1	6.24	3.09	51
Through	WB	2	4.20	2.61	38
Through	WB	3	NP	2.13	Lane added

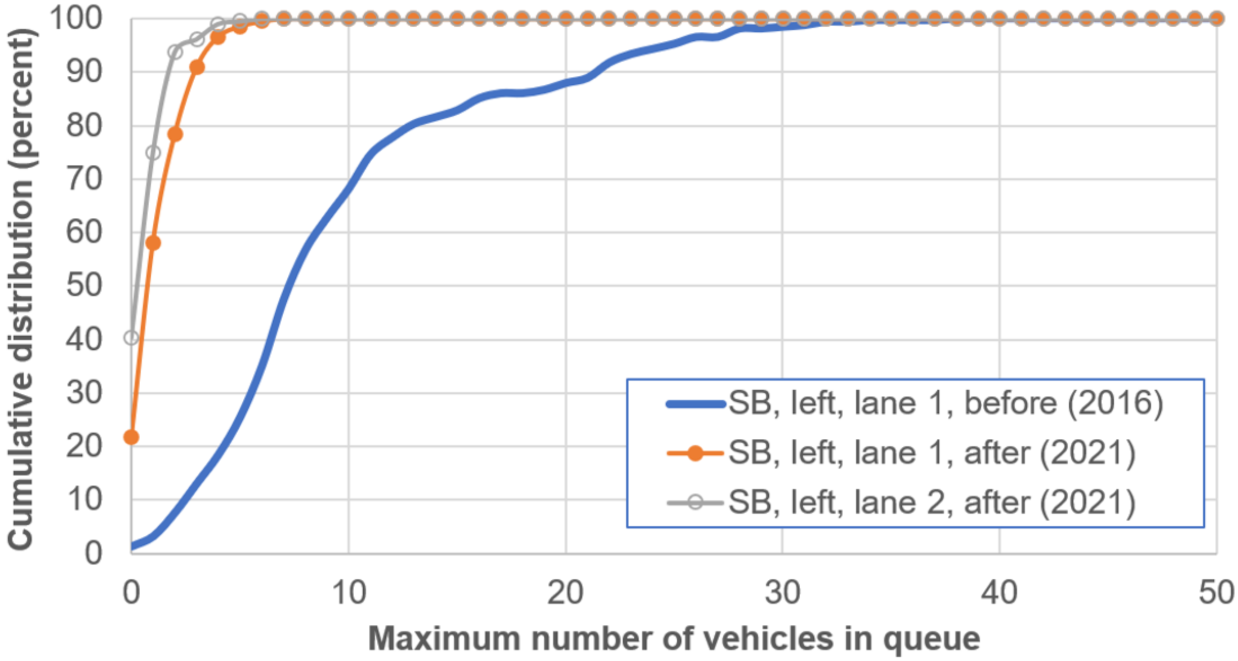
Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).

The installation of the quadrant road created approximately an 80 percent reduction in the left-turn queue going from the west leg to the north leg (i.e., EB left turn). Figure 46 shows the distribution of maximum queue length per signal cycle for both before and after periods for EB left turns. The SB left turn experienced a large reduction in queue lengths, 84 percent (figure 47 or table 19). The EB through and SB left movements also showed large benefits from the installation of the intersection treatments. Figure 48 shows the cumulative distribution for the EB through movement, which had about a 75 percent reduction. Figure 49 is for SB left turns, which showed an 84 percent reduction.



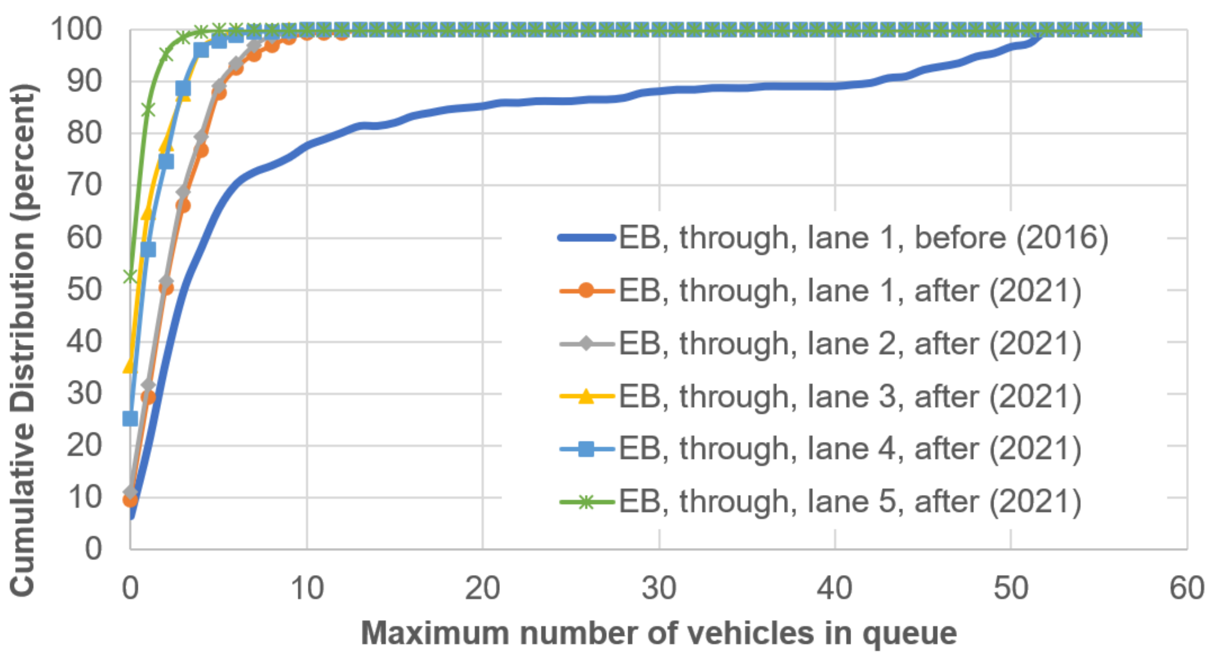
Source: FHWA.

Figure 46. Graph. Distribution of maximum queue length for Valencia Road at Kolb Road EB left by period and lane.



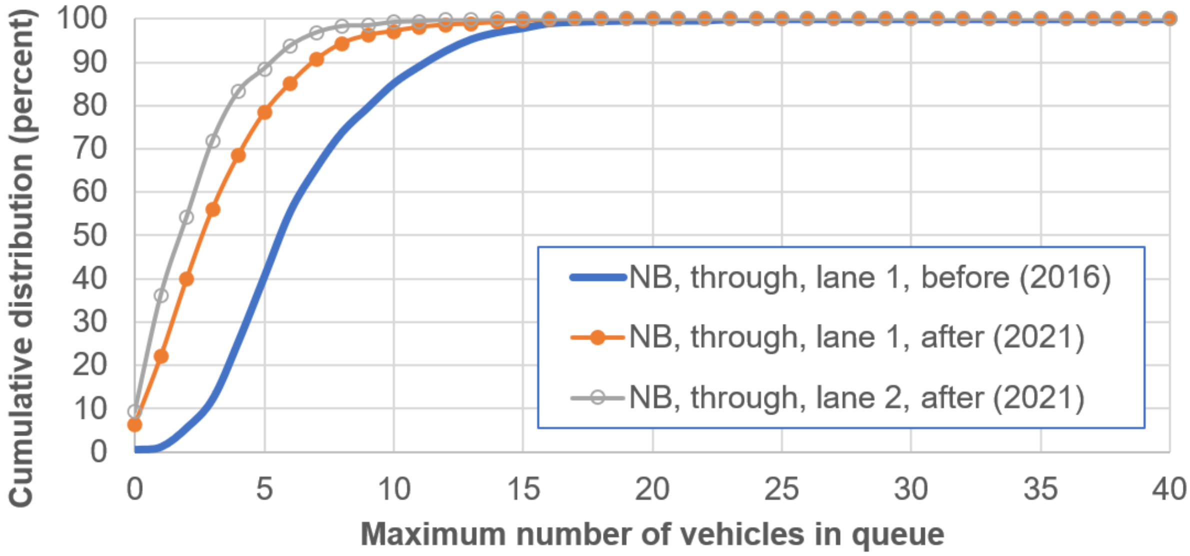
Source: FHWA.

Figure 47. Graph. Distribution of maximum queue length for Valencia Road at Kolb Road SB left by period and lane.



Source: FHWA.

Figure 48. Graph. Distribution of maximum queue length for Valencia Road at Kolb Road EB through by period and lane.



Source: FHWA.

Figure 49. Graph. Distribution of maximum queue length for Valencia Road at Kolb Road NB through by period and lane.

Travel Times

The treatments at Valencia Road and Kolb Road included adding MUTs along some of the legs and adding a quadrant roadway in the northwest corner. The travel distance for the WB left turns increased, but the travel time may decrease because of efficiencies for signal timing at the main intersection. MUTs and quadrant roadways can benefit other movements because the green signal time at the subject intersections that previously served the left-turn movement can be reallocated to other movements. Table 20 shows that the travel times for the NB through and SB left movements increased from the before to the after period during the p.m. period. Improvement in travel time was experienced for the other movements. Table 21 lists the start and end points by movement.

Table 20. Field-measured travel times for Valencia Road at Kolb Road, Tucson, AZ.

Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
EB left	a.m.	125	112	10
EB left	Off	100	161	−61
EB left	p.m.	242	109	55
EB through	a.m.	80	53	34
EB through	Off	60	51	15
EB through	p.m.	268	49	82
NB left	a.m.	148	129	12
NB left	Off	115	84	27
NB left	p.m.	144	78	46
NB through	a.m.	77	47	39

Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
NB through	Off	79	55	31
NB through	p.m.	73	75	−3
SB left	a.m.	81	110	−36
SB left	Off	91	83	9
SB left	p.m.	122	129	−6
SB through	a.m.	58	41	30
SB through	Off	31	69	−121
SB through	p.m.	73	52	28
WB left	a.m.	104	94	10
WB left	Off	78	118	−52
WB left	p.m.	111	94	15
WB through	a.m.	109	42	61
WB through	Off	84	65	22
WB through	p.m.	116	54	53

Table 21. Coordinates for start and end field-measured travel times for Valencia Road at Kolb Road, Tucson, AZ.

Site	Movement	Start Point	End Point
Valencia Road	EB left	32.139469, −110.845311	32.142617, −110.840700
Valencia Road	EB through	32.139469, −110.845311	32.139169, −110.838664
Valencia Road	NB left	32.137361, −110.840725	32.139469, −110.845311
Valencia Road	NB through	32.249225, −110.971908	32.142617, −110.840700
Valencia Road	SB left	32.142617, −110.840700	32.139169, −110.838664
Valencia Road	SB through	32.142617, −110.840700	32.249225, −110.971908
Valencia Road	WB left	32.13916944, −110.8386639	32.13736111, −110.840725
Valencia Road	WB through	32.139169, −110.838664	32.139469, −110.845311

Pedestrian Walking Paths

As previously demonstrated for the other Arizona sites, where practical, the research team walked designated paths to determine if the road intersection implementation project influenced the pedestrian access to businesses at the intersection. The land use at this location is minimally developed and had essentially no development until the improvement project. Consequently, there was no reason to assess the current pedestrian walkability since there were really no pedestrians present at the site.

Performance Measures Summary

Based primarily on the travel time and vehicle maneuver information, the reduced left-turn conflicts at this hybrid location improve the functionality of the corridor. The placement of an

RLTCI creates a midblock facility that may need to be signalized along the Grant Road corridor since the RLTCI performs well. Table 22 reflects the observed performance of the intersection improvements.

Table 22. Performance of the Valencia Road at Kolb Road MUT (RLTCI or ThruU) contrasting after to before, Arizona.

Performance Measure	Valencia Road and Kolb Road Indirect Left and Quasi Quadrant Intersection
Queues, left turn	Reduced for all approaches except NB, which increased slightly
Queues, through	Reduced for all approaches except SB, which increased slightly
Travel times	Consistently improved
Pedestrian walking path	Not assessed for this location

Note: The traffic volumes at the Valencia Road at Knob Road intersection increased by 13 percent in the after period.

MINNESOTA SITES

The operational performance for the Minnesota sites is summarized in the following sections. MN–65 is also sometimes referred to as Trunk Highway 65 or TH–65. Because the intersection of MN–65 and Viking Boulevard NE is signalized, operational performance measures are expected to differ when this intersection is compared to unsignalized RCUT locations. For example, the signalized intersection is expected to have higher volumes and introduced delays due to the presence of the traffic signal. The additional MN–65 study locations include two recently converted RCUTs (at 187th Lane NE and 157th Avenue NE), one preexisting RCUT (at 181st Avenue NE), and one traditional intersection with a four-lane, divided highway and median crossover configuration that is consistent with the before conditions for 187th Lane NE and 157th Avenue NE (the intersection of MN–65 at 209th Avenue NE). Table 23 summarizes each of the five intersections, as well as their configuration before and after construction. The intersections are listed from north to south and the primary corridor occurs along this north-south alignment. 187th Lane NE and 157th Avenue NE are the locations where the geometric configuration was recently converted from the unsignalized intersection with a divided median crossover at the intersection to the unsignalized RCUTs.

Table 23. Geometric configurations for before and after periods.

Road Intersecting MN–65, City	Before Configuration	After Configuration
209th Avenue NE, Ham Lake, MN	Data not collected because site was selected as a comparison site	Traditional two-way stop
Viking Boulevard NE, Ham Lake, MN	Traditional signalized	Signalized RCUT
187th Lane NE, East Bethel, MN	Traditional two-way stop	Unsignalized RCUT
181st Lane NE, East Bethel, MN	Data not collected because site was selected as a comparison site	Unsignalized RCUT
157th Lane NE, East Bethel, MN	Traditional two-way stop	Unsignalized RCUT

Minnesota sites' major road is on the NB and SB approaches, and the paths followed by the left-turning, through, and right-turning vehicles remain the same after the installation of the RCUT. However, the paths for the minor road vehicles going straight or turning left change with the RCUT. The vehicles on the minor road approach must turn right at the main intersection with the major road. The vehicles then use the downstream U-turn intersection to turn around and return to the main intersection. A minor road driver who wants to go straight and stay on the minor road needs to turn right onto the minor road. A minor road driver who wants to turn left needs to go straight at the main intersection.

Minnesota Unsignalized RCUTs

The review of these sites includes some brief information about their location and configuration, followed by reviews of expected operational performance for the locations where the Minnesota unsignalized two-way stop locations were converted to unsignalized RCUTs (MN–65 at 157th Avenue NE and 187th Lane NE). The research team acquired a wide variety of field data elements to use to assess the operational performance of the intersections. The performance measures extracted from this information included:

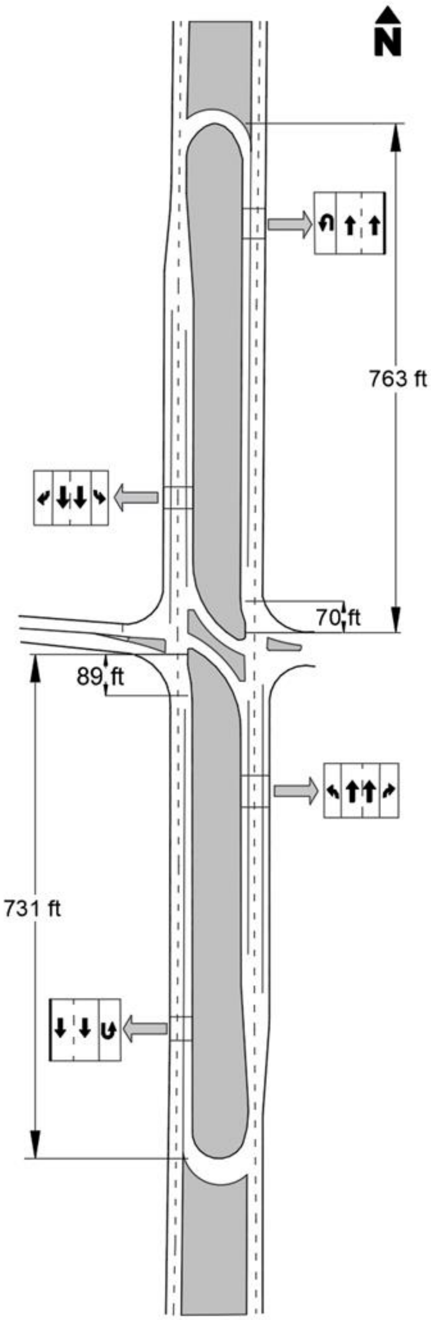
- Examination of observed traffic volumes.
- Measurement and evaluation of left-turn queues.
- Execution of car following travel times.

The inclusion of pedestrian walking paths and measures related to the presence of a traffic signal (such as control delay or length of mainline queues where applicable) were not included as performance measures for the unsignalized RCUT configuration. These MN–65 sites maintained high operating speeds with no stopping on the primary approach. Consequently, pedestrian facilities are not available at these locations. During the field data collection efforts, the data collection team observed that there were few pedestrians crossing at any of these locations.

General Site Characteristics

The main approach legs for each intersection are in the NB and SB direction. The MN–65 study sites are located north of Minneapolis, MN, and accommodate high volumes traveling south toward the city. Viking Boulevard NE is signalized at the core intersection as well as at the upstream and downstream U-turn locations (refer to Viking Boulevard NE analysis in the next section).

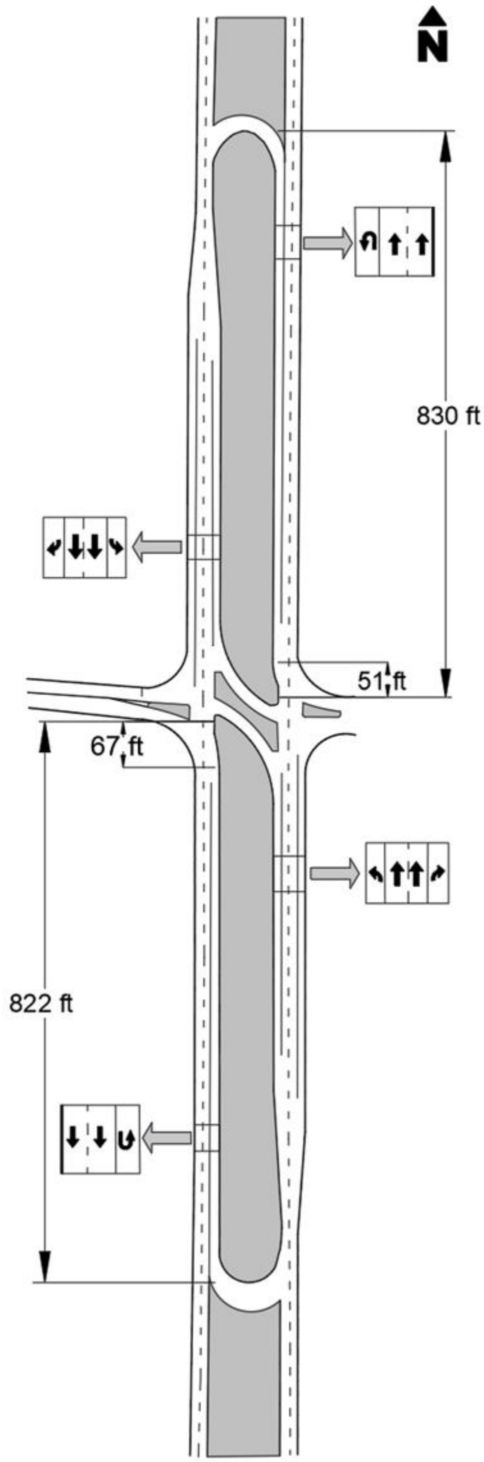
Because alternative intersection configurations are relatively new in the United States, it is not completely clear what dimensions are optimal, but the schematic in figure 50 depicts the distance from the right-turn intersection curb return to the U-turn for the intersection at 157th Avenue NE. This dimension is 731 ft for the NB to SB U-turn and 763 ft for the SB to NB U-turn. A similar dimension is depicted for the 181st Avenue NE and 187th Lane NE locations. The distance between the U-turn and the main intersection must be sufficiently long to enable vehicles exiting the U-turn to change lanes to the right in time to turn right at the intersection (for vehicles that previously would have continued into the median and crossed without any constraints).



Source: FHWA.

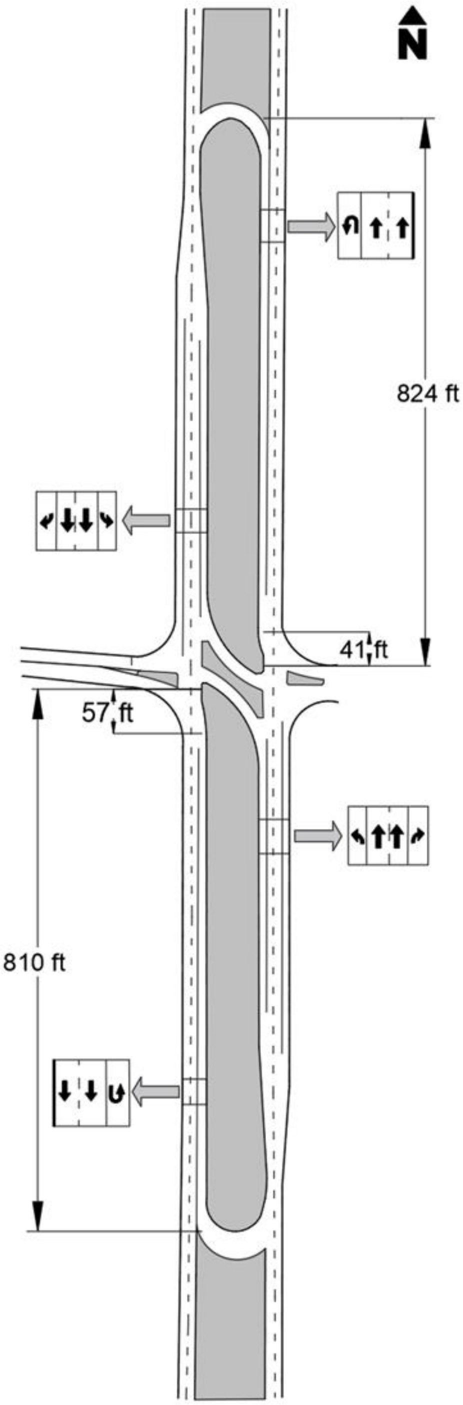
Figure 50. Illustration. RCUT schematic for MN–65 at 157th Avenue NE, Ham Lake, MN.

This U-turn storage lane should have some logical minimum and maximum thresholds, so that a vehicle that turns right from the minor approach will have adequate time to safely change lanes and enter the U-turn deceleration lanes. For the observed sites, the U-turn placement ranged from 662 to 783 ft. The optimal value is not known at this time. Figure 51 and figure 52 depict the layouts for the MN–65 intersections with the unsignalized RCUT intersections at 181st Avenue NE and 187th Lane NE. Table 24 provides additional dimensions associated with the three unsignalized RCUTs.



Source: FHWA.

Figure 51. Illustration. RCUT schematic for MN–65 at 181st Avenue NE, Ham Lake, MN.



Source: FHWA.

Figure 52. Illustration. RCUT schematic for MN–65 at 187th Lane NE, East Bethel, MN.

Table 24. Unsignalized RCUT geometric characteristics for Minnesota sites.

Scenario	Direction 1	Direction 2	Curb Return to U-turn (ft)	Length of Taper (ft)	Posted Speed Limit (mph)
157th Avenue NE	NB	SB	662	39	65
157th Avenue NE	SB	NB	724	35	65
181st Avenue NE	NB	SB	767	34	65
181st Avenue NE	SB	NB	783	50	65
187th Lane NE	NB	SB	767	30	65
187th Lane NE	SB	NB	783	48	65

Traffic Volumes

An agency considers adopting alternative intersection configurations for several reasons. For example, at the Minnesota sites, the left-turning vehicles were prone to develop queues during peak periods. This trend introduces a speed differential that could create safety challenges. Table 25 depicts the peak hour volumes observed at the Minnesota study sites during the a.m. and p.m. peak hour commutes for both the before and after periods. The NB movement provided the peak volume along this corridor during the evening.

Table 25. Peak hour volumes at the unsignalized Minnesota sites.

Site	Period	Peak Hour a.m.	SB Volume a.m. (vehicles)	Peak Hour p.m.	NB Volume p.m. (vehicles)
MN–65 at 157th Avenue NE	Before	7:00–8:00 a.m. May 24, 2018	2,038	4:15–5:15 p.m. May 24, 2018	2,377
MN–65 at 157th Avenue NE	After	7:00–8:00 a.m. May 4, 2021	1,841	4:30–5:30 p.m. May 6, 2021	2,248
MN–65 at 181st Avenue NE	Before	NA	NA	NA	NA
MN–65 at 181st Avenue NE	After	7:00–8:00 a.m. May 11, 2021	2,480	4:15–5:15 p.m. May 11, 2021	2,245
MN–65 at 187th Lane NE	Before	7:00–8:00 a.m. May 22, 2018	1,861	4:15–5:15 p.m. May 22, 2018	2,432
MN–65 at 187th Lane NE	After	7:00–8:00 a.m. May 6, 2021	1,621	5:00–6:00 p.m. May 6, 2021	2,087
MN–65 at 209th Avenue NE	Before	NA	NA	NA	NA
MN–65 at 209th Avenue NE	After	7:00–8:00 a.m. May 12, 2021	1,450	4:15–5:15 p.m. May 12, 2021	1,953

NA = data not available because before data were not collected for this comparison site that was identified and included in the study at the time of the after data collection effort.

For 157th Avenue NE and 187th Lane NE, the after condition has traffic volumes that are similar to the before condition. Therefore, a potential performance measure for the unsignalized Minnesota intersections is the length of queue during these peak hour conditions, in addition to the comparison of the traffic volumes in 2018 and 2021.

Vehicle Queues

Converting the traditional intersection configuration to the RCUT requires minor road through maneuvers to follow an alternative vehicle path. To appreciate the value of installing the RCUT, compare the length of the queues during peak hours along the minor road before and after conditions. In general, for the minor road approaches along the unsignalized RCUTs, the maximum average queue during a peak period was six cars or less. The maximum queues along the minor road during the a.m. and p.m. peak hours were identified for both the before period and the after period as shown in table 26. In general, there was only a one to three car change in the queues on the minor road after the installation of the RCUT. In many cases, the queue length increased by one to three cars, but in a few cases the queue length decreased by one to two cars.

Table 26. Maximum queue (number of vehicles) for a.m. and p.m. peak hours for unsignalized Minnesota intersections.

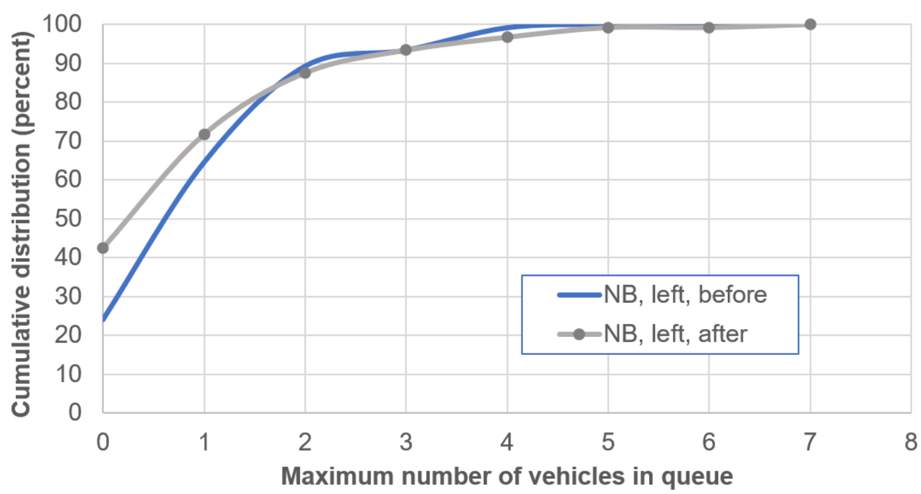
Site	Move	Direction	Before a.m. Max	After a.m. Max	Before p.m. Max	After p.m. Max
MN–65 at 157th Avenue NE	Left	NB	4	7	5	5
MN–65 at 157th Avenue NE	Left	SB	2	1	2	2
MN–65 at 157th Avenue NE	Left	EB	2	1	3	5
MN–65 at 157th Avenue NE	Left	WB	2	1	2	1
MN–65 at 157th Avenue NE	Right	EB	6	5	3	6
MN–65 at 157th Avenue NE	Right	WB	1	1	2	3
MN–65 at 181st Avenue NE (compare, unsignalized RCUT)	Left	EB	NA	0	NA	3
MN–65 at 181st Avenue NE (compare, unsignalized RCUT)	Left	WB	NA	2	NA	0
MN–65 at 181st Avenue NE (compare, unsignalized RCUT)	Right	EB	NA	1	NA	4
MN–65 at 181st Avenue NE (compare, unsignalized RCUT)	Right	WB	NA	4	NA	5
MN–65 at 187th Lane NE	Left	NB	2	0	4	0
MN–65 at 187th Lane NE	Left	SB	2	0	4	1
MN–65 at 187th Lane NE	Left	EB	0	2	2	0
MN–65 at 187th Lane NE	Left	WB	0	1	2	5
MN–65 at 187th Lane NE	Right	EB	6	6	3	2
MN–65 at 187th Lane NE	Right	WB	0	2	2	4

Site	Move	Direction	Before a.m. Max	After a.m. Max	Before p.m. Max	After p.m. Max
MN–65 at 209th Avenue NE (compare, 2w-stop)	Left	EB	NA	1	NA	1
MN–65 at 209th Avenue NE (compare, 2w-stop)	Left	WB	NA	3	NA	3
MN–65 at 209th Avenue NE (compare, 2w-stop)	Right	EB	NA	1	NA	1
MN–65 at 209th Avenue NE (compare, 2w-stop)	Right	WB	NA	1	NA	1

Left = left turn in before period and indirect left turn (U-turn bay) in after period; Max = maximum queue (number of vehicles) within the period on the minor road; 2w-stop = two-way stop; NA = queue length not provided because data were not collected in the before period at this intersection.

Table 26 also provides the maximum queue on the minor road for the comparison sites. The site with two-way stop control had nominal queues, overall. The maximum queue was one vehicle, except for the WB left turns, when the queue was three cars. The other comparison site (MN–65 at 181st Avenue) had maximum queues that were similar to the values for the other two unsignalized RCUTs in Minnesota.

Table 26 also provides the maximum queue for the left-turn lane on the major road (NB and SB) for MN–65 at 157th Avenue NE and 187th Lane NE. The queue lengths were similar between the periods. The maximum queue during a peak period for NB left turns at 157th Avenue NE went from four vehicles to seven vehicles, which initially appears to be a large change; however, that only represented a single occurrence. Figure 53 shows that the cumulative distribution of 1-min maximum queue counts for the two peak hours are similar.



Source: FHWA.

Figure 53. Graph. Distribution of maximum queue length for MN–65 at 157th Avenue NB left turns (major road) for peak hours by period.

Travel Times

Excessive delay can often be observed using travel-time studies. For this effort, the research team conducted field-measured, travel-time studies with a primary focus on the through and left-turn maneuvers for locations with modified left-turn operations. Table 27 provides the start and end coordinates for the travel-time runs by movement. Table 28 lists the travel times for the cross streets. In general, the travel times for the minor road approaches increased due to the additional distance the driver needs to cover while turning right, going to and using the U-turn lane, and then returning to the main intersection to either turn right (for through movements) or continue straight (for left turns). Even with the additional distance that drivers need to cover, the WB left on 157th Avenue NE experienced a decrease in travel time during the peak periods.

Table 27. Coordinates for start and end field-measured travel times for unsignalized RCUTs in Minnesota.

State	Site	Movement	Start Point	End Point
MN	157th Avenue NE	EB left	45.25597, −93.23431	45.257364, −93.23297
MN	157th Avenue NE	EB through	45.25597, −93.23431	45.25589, −93.23152
MN	157th Avenue NE	NB left	45.24800, −93.23361	45.25597, −93.23431
MN	157th Avenue NE	NB through	45.24800, −93.23361	45.26433, −93.23430
MN	157th Avenue NE	SB left	45.26433, −93.23430	45.25589, −93.23152
MN	157th Avenue NE	SB through	45.26433, −93.23430	45.24800, −93.23361
MN	157th Avenue NE	WB left	45.25589, −93.23152	45.25591, −93.23295
MN	157th Avenue NE	WB through	45.25589, −93.23152	45.25597, −93.23431
MN	181st Avenue NE	EB left	45.29802, −93.236251	45.298012, −93.234025
MN	181st Avenue NE	EB through	45.29802, −93.236251	45.297979, −93.232001
MN	181st Avenue NE	NB left	45.291733, −93.234111	45.298020, −93.236251
MN	181st Avenue NE	NB through	45.291733, −93.234111	45.307120, −93.234969
MN	181st Avenue NE	SB left	45.307120, −93.234969	45.297979, −93.232001
MN	181st Avenue NE	SB through	45.307120, −93.234969	45.291733, −93.234111
MN	181st Avenue NE	WB left	45.297979, −93.232001	45.255124, −93.23298
MN	181st Avenue NE	WB through	45.297979, −93.232001	45.29802, −93.236251
MN	187th Lane NE	EB left	45.309437, −93.236097	45.310654, −93.235188
MN	187th Lane NE	EB through	45.309437, −93.236097	45.309427, −93.234575
MN	187th Lane NE	NB left	45.29804, −93.23406	45.30943, −93.23609
MN	187th Lane NE	NB through	45.29804, −93.23406	45.31064, −93.23517
MN	187th Lane NE	SB left	45.31038, −93.23565	45.30943, −93.23465
MN	187th Lane NE	SB through	45.31038, −93.23565	45.29804, −93.23406
MN	187th Lane NE	WB left	45.309427, −93.234575	45.308016, −93.235337
MN	187th Lane NE	WB through	45.309427, −93.234575	45.309437, −93.236097
MN	209th Avenue NE	EB left	45.348783, −93.237401	45.35616, −93.2369
MN	209th Avenue NE	EB through	45.348783, −93.237401	45.348739, −93.236048

State	Site	Movement	Start Point	End Point
MN	209th Avenue NE	NB left	45.33418, −93.2368	45.348783, −93.237401
MN	209th Avenue NE	NB through	45.33418, −93.2368	45.35616, −93.2369
MN	209th Avenue NE	SB left	45.35616, −93.2369	45.348739, −93.236048
MN	209th Avenue NE	SB through	45.35616, −93.2369	45.33418, −93.2368
MN	209th Avenue NE	WB left	45.348739, −93.236048	45.33418, −93.2368
MN	209th Avenue NE	WB through	45.348739, −93.236048	45.348783, −93.237401

Table 28. Field-measured travel times for minor road approaches along MN–65 corridor.

Site	Move	Time	B_TT(Ave)	A_TT(Ave)	Percent ***R***
157th Avenue NE	EB left	a.m.	58	83	−30
157th Avenue NE	EB left	Off	36	105	−66
157th Avenue NE	EB left	p.m.	48	134	−64
157th Avenue NE	EB through	a.m.	35	69	−50
157th Avenue NE	EB through	Off	41	94	−56
157th Avenue NE	EB through	p.m.	74	116	−36
157th Avenue NE	WB left	a.m.	115	89	29
157th Avenue NE	WB left	Off	24	142	−83
157th Avenue NE	WB left	p.m.	133	121	10
157th Avenue NE	WB through	a.m.	70	109	−35
157th Avenue NE	WB through	Off	26	135	−81
157th Avenue NE	WB through	p.m.	64	137	−53
187th Lane NE	EB left	a.m.	65	118	−45
187th Lane NE	EB left	Off	44	72	−38
187th Lane NE	EB left	p.m.	52	123	−58
187th Lane NE	EB through	a.m.	48	78	−38
187th Lane NE	EB through	Off	47	91	−48
187th Lane NE	EB through	p.m.	94	107	−12
187th Lane NE	WB left	a.m.	36	127	−72
187th Lane NE	WB left	Off	41	68	−40
187th Lane NE	WB left	p.m.	70	58	20
187th Lane NE	WB through	a.m.	68	104	−35
187th Lane NE	WB through	Off	36	75	−52
187th Lane NE	WB through	p.m.	75	107	−30

Table 29. Field-measured travel times for minor road approaches along MN–65 corridor.

Site	Move	Time	A_TT(Ave)
181st Avenue NE	EB left	a.m.	64
181st Avenue NE	EB left	Off	81
181st Avenue NE	EB left	p.m.	153
181st Avenue NE	EB through	a.m.	85
181st Avenue NE	EB through	Off	97
181st Avenue NE	EB through	p.m.	202
181st Avenue NE	WB left	a.m.	94
181st Avenue NE	WB left	Off	81
181st Avenue NE	WB left	p.m.	56
181st Avenue NE	WB through	a.m.	80
181st Avenue NE	WB through	Off	106
181st Avenue NE	WB through	p.m.	105

Table 30 provides the travel times by movement along MN–65 (i.e., the major road that does not have active traffic control) for the before-after sites, and table 31 lists the travel times for the comparison sites. The speed column in table 31 shows that the major road was free flowing. For the through movements, the speeds are generally about 60 mph, as expected for an uncontrolled rural highway. Also as expected, the left turns from the major road had similar results in the before and after periods because there was no change in the intersection traffic control.

Table 30. Field-measured travel times for major road approaches along MN–65 corridor.

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
157th Avenue NE	NB left	a.m.	44	57	−30
157th Avenue NE	NB left	Off	37	51	−38
157th Avenue NE	NB left	p.m.	36	61	−68
157th Avenue NE	NB through	a.m.	70	77	−9
157th Avenue NE	NB through	Off	64	69	−7
157th Avenue NE	NB through	p.m.	68	69	−1
157th Avenue NE	SB left	a.m.	51	56	−9
157th Avenue NE	SB left	Off	41	74	−79
157th Avenue NE	SB left	p.m.	41	68	−65
157th Avenue NE	SB through	a.m.	70	80	−14
157th Avenue NE	SB through	Off	68	74	−9
157th Avenue NE	SB through	p.m.	67	72	−7
187th Lane NE	NB left	a.m.	45	67	−48
187th Lane NE	NB left	Off	46	68	−48
187th Lane NE	NB left	p.m.	192	72	63

Site	Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
187th Lane NE	NB through	a.m.	47	51	−7
187th Lane NE	NB through	Off	46	52	−12
187th Lane NE	NB through	p.m.	48	46	4
187th Lane NE	SB left	a.m.	9	26	−183
187th Lane NE	SB left	Off	11	34	−209
187th Lane NE	SB left	p.m.	9	51	−467
187th Lane NE	SB through	a.m.	48	45	6
187th Lane NE	SB through	Off	46	48	−4
187th Lane NE	SB through	p.m.	45	46	−1

Table 31. Field-measured travel times for major road approaches along MN–65 corridor at the two comparison sites.

Site	Direction and Movement	Period	Travel Time After (s)	Speed After (mph)
181st Avenue NE	NB left	a.m.	56.0	35
181st Avenue NE	NB left	p.m.	51.5	38
181st Avenue NE	NB through	a.m.	52.5	73
181st Avenue NE	NB through	p.m.	63.0	61
181st Avenue NE	SB left	a.m.	56.5	51
181st Avenue NE	SB left	p.m.	57.5	46
181st Avenue NE	SB through	a.m.	57.0	67
181st Avenue NE	SB through	p.m.	54.5	70
209th Avenue NE	NB left	a.m.	87.0	48
209th Avenue NE	NB left	p.m.	69.5	55
209th Avenue NE	NB through	a.m.	77.5	71
209th Avenue NE	NB through	p.m.	74.5	73
209th Avenue NE	SB left	a.m.	98.5	20
209th Avenue NE	SB left	p.m.	50.0	39
209th Avenue NE	SB through	a.m.	81.5	68
209th Avenue NE	SB through	p.m.	86.0	64

Performance Measures by Unsignalized Minnesota RCUT Sites

Based on the summary of unsignalized RCUTs for 157th Avenue NE and 187th Lane NE, as well as contrasting this information with the comparison sites at 181st Avenue NE and 209th Avenue NE, the following summary table 32 reflects the observed performance of the intersection improvements.

Table 32. Performance for MN–65 unsignalized RCUTs contrasting after to before.

Performance Measure	187th Lane NE	157th Avenue NE
Queues, minor road	Similar	Reduced
Travel times	Equivalent	Equivalent

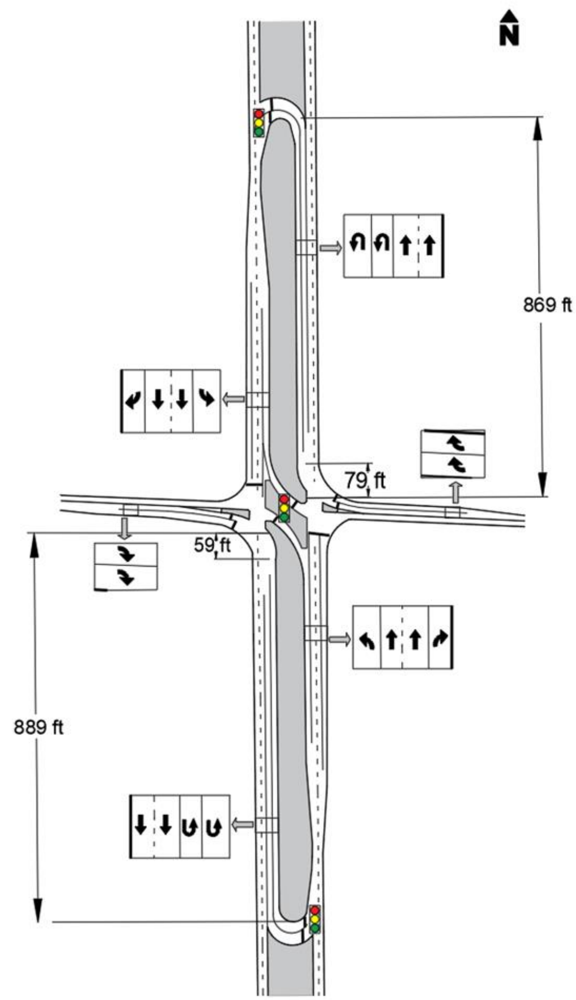
Note: The traffic volumes at the 157th Avenue NE and 187th Lane NE intersection were about 10 percent less in the after period.

Minnesota Signalized RCUT

The MN–65 corridor included one signalized RCUT at its intersection with Viking Boulevard NE. This intersection is at the north end of the improvement corridor. This intersection differs from the unsignalized RCUT intersections to accommodate potential future pedestrian and bicycle activity, because the intersection has higher volumes on the Viking Boulevard NE cross street and a wide pedestrian and bicycle lane constructed within the core intersection island.

General Site Characteristics

As shown in figure 54, the MN–65 intersection at Viking Boulevard NE has a traffic signal located at the core intersection, as well as a signalized SB to NB U-turn and a signalized NB to SB U-turn on MN–65. At the time of the after-data collection, the right-turn maneuvers in the core intersection did not accommodate right-turn-on-red movements, but the signalized U-turns did permit left-turn-on-red movements. As previously indicated in table 23, the traditional signalized intersection was replaced with the signalized RCUT.



Source: FHWA.

Figure 54. Illustration. Signalized RCUT schematic for MN–65 at Viking Boulevard.

Traffic Volumes

As shown in table 33, the traffic volumes following construction increased considerably. This change was unlikely to be due to the road and more likely due to the intersection being in a growing and prospering region. For the a.m. peak period, the traffic volumes increased by 16 percent. Similarly, during the p.m. peak period, the traffic volume increased by 19 percent when compared to the before-data collection. The relationship to this increased traffic volume should be compared to the observed queues and travel time, as shown in the following sections. Therefore, a potential performance measure for this signalized Minnesota intersection is the length of queue during the peak hour conditions, in addition to the comparison of the traffic volumes in 2018 and 2021.

Table 33. Peak hour volumes at Viking Boulevard NE (before and after period).

Intersection Configuration	Hour and Day of a.m. Peak Period	Volume in SB Direction (vehicles)	Hour and Day of p.m. Peak Period	Volume in NB Direction (vehicles)
Traditional signalized before	7:00–8:00 a.m. May 22, 2018	2,874	4:45–5:45 p.m. May 22, 2018	3,726
Signalized RCUT after	7:00–8:00 a.m. May 12, 2021	3,341	4:15–5:15 p.m. May 12, 2021	4,432

Note: The MN–65 cross section at upstream and downstream locations has four travel lanes (two lanes in each direction).

Vehicle Queues

Although the conversion of the traditional intersection configuration to the signalized RCUT at the intersection of MN–65 and Viking Boulevard NE can help optimize the primary (north and south) approaches, this treatment requires the through and left-turn movements on the minor roads to follow an alternative vehicle path. The maximum number of vehicles in a queue during each signal cycle was identified. A cumulative distribution of these maximum number of vehicles per cycle was developed for each movement. To compare the distribution between the before and after periods, the maximum queue length at 85 percent was identified. Table 34 provides the results for MN–65 at Viking Boulevard NE for the queue length representing 85 percent of the signal cycles that had queues of that length or longer. Table 35 also provides the percent reduction in queue length from the before to the after period. In general, there was between 60 and 63 percent reduction in queues on the minor road after the installation of the RCUT at the signalized intersection. The research team also always checked to confirm that the maximum queue occurred in the U-turn lane during the after period. This value was always shorter than the maximum queue on the minor road.

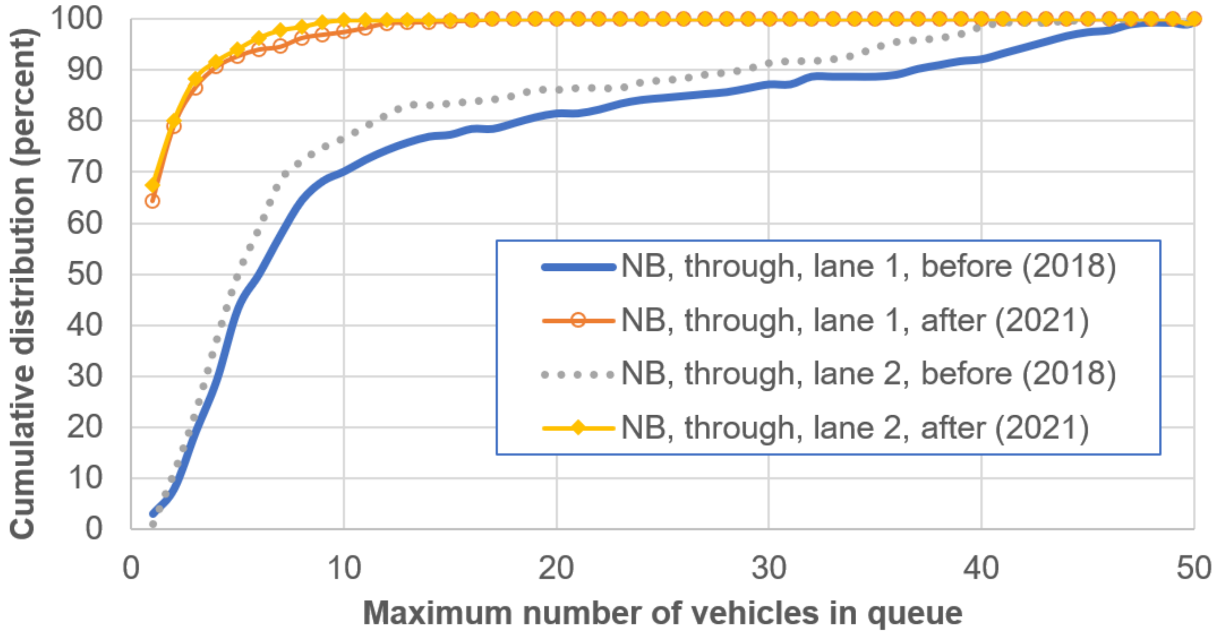
Table 34. Comparison of before and after queue lengths for MN–65 at Viking Boulevard NE.

Road	Direction	Lane	Movement Before	Movement After	Number of Vehicles Before (2018)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Major	NB	1	Left	Left	5.39	2.57	52
Major	SB	1	Left	Left	2.73	0.62	77
Major	NB	1	Through	Through	25.40	1.79	93
Major	NB	2	Through	Through	17.03	1.61	91
Major	SB	1	Through	Through	16.42	0.86	95
Major	SB	2	Through	Through	10.81	1.07	90
Minor	EB	1	Through left	All	10.30	3.19	60*
Minor	EB	2	Right	All	NP	0.98	60
Minor	WB	1	Through left	All	11.03	1.99	63*
Minor	WB	2	Right	All	NP	2.14	63

* Percent reduction between longer queue for the after period and the queue in the before period.
Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated in another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).

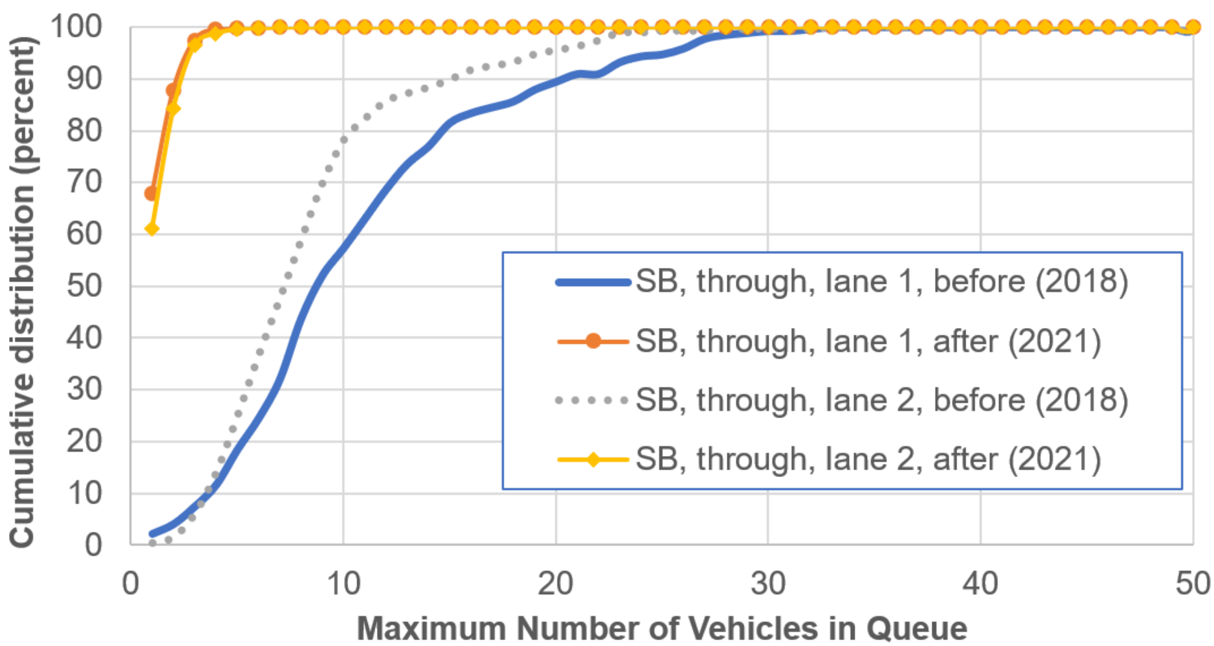
The longest queues at this site in the before period were for the through maneuvers on the NB and SB major road, which saw a significant reduction. Table 34 shows that the major road through lanes had a better than 90 percent reduction in queues. For example, the NB lanes had 85 percent of their signal cycles with 25.4 vehicles or less in the queue in the before period, and only 1.79 vehicles or less in the queue in the after period. The left-turn maneuver from the major road approaches saw large reductions in queue lengths (between 52 and 77 percent).

Figure 55 shows the distribution of maximum queue length per signal cycle for both before and after periods for the major road NB traffic and figure 56 provides similar data for the SB traffic. As these figures illustrate, the reductions in queue lengths throughout a typical day is noticeable.



Source: FHWA.

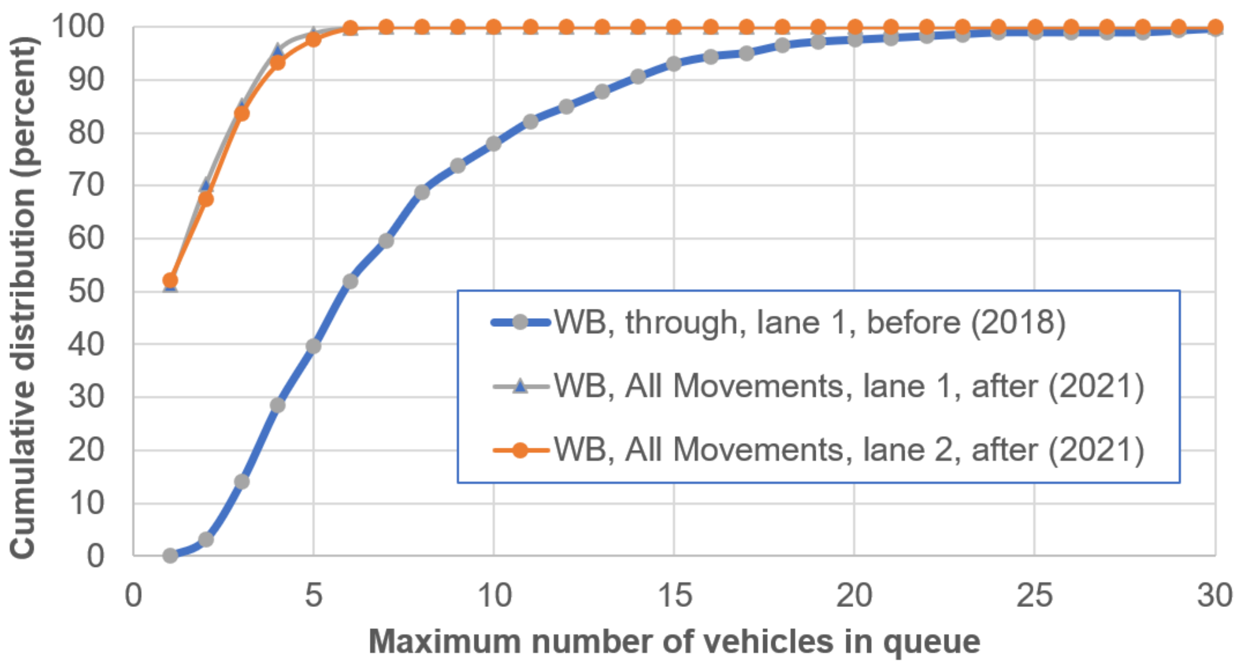
Figure 55. Graph. Distribution of maximum queue length for MN–65 at Viking Boulevard NB through (major road) by period and lane.



Source: FHWA.

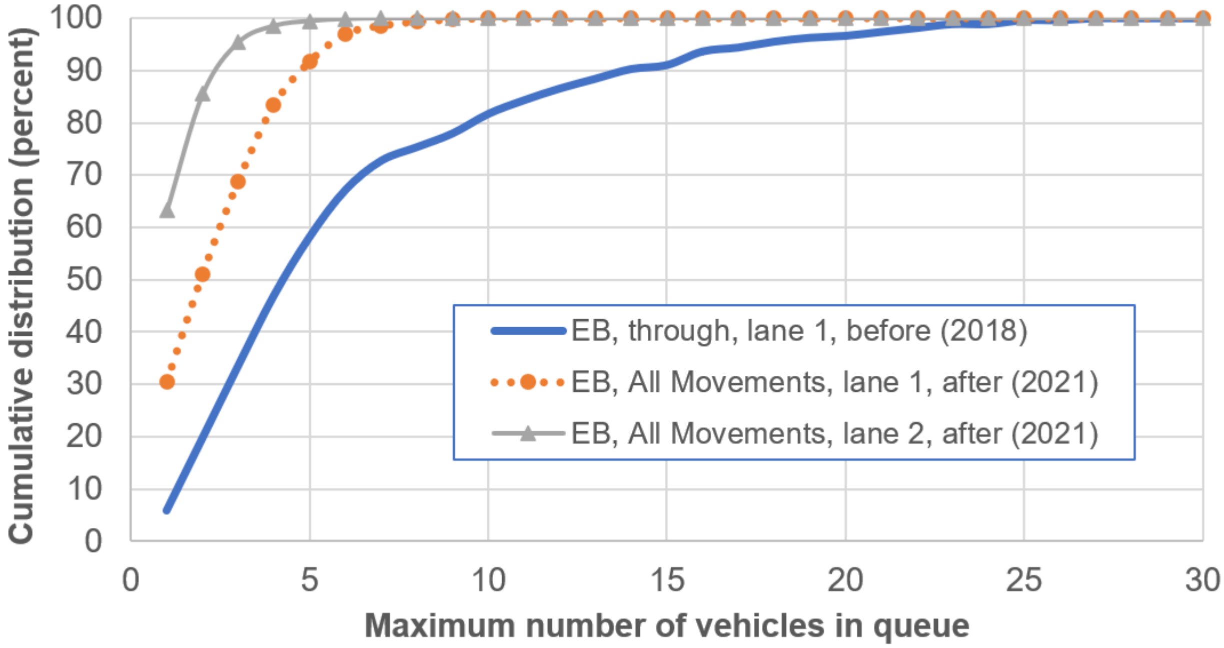
Figure 56. Graph. Distribution of maximum queue length for MN–65 at Viking Boulevard SB through (major road) by period and lane.

The evaluation of queue lengths for the minor road approaches compared the queues measured for the through and left traffic in the before period to the queues measured on the right-turn lanes in the after period. In the after period, all minor road vehicles need to turn right. Figure 57 shows the results for the WB movement and figure 58 shows the EB movement. The shift in queue lengths is also noticeable for the minor road approaches, but not quite as dramatic because the queues in the before period were not as long on the minor road compared to the major road. Each minor road approach in the before period had two lanes: an exclusive right-turn lane and a shared through and left lane. In the after period, these lanes become two exclusive right-turn lanes due to the addition of the raised median redirecting the minor road through and left turns to initially turn right. Table 34 also shows that by reallocating the minor road approach lanes to handle the same movements, the queuing vehicles could be distributed between both lanes, and the overall approach saw reductions in queue lengths of 60 percent.



Source: FHWA.

Figure 57. Graph. Distribution of maximum queue length for MN–65 at Viking Boulevard (minor road) WB by period and lane.



Source: FHWA.

Figure 58. Graph. Distribution of maximum queue length for MN–65 at Viking Boulevard (minor road) EB by period and lane.

Travel Times

One reason agencies convert traditional signalized intersections to signalized RCUT intersections is to preserve the level of service of the major road while enhancing the performance of minor road traffic. Excessive delay can be observed using travel-time studies, and this delay includes user delays, as well as control delays for the traffic signal. The placement of three traffic signals along a corridor that previously functioned with only one traffic signal requires careful traffic signal coordination and timing to optimize facility and incorporate control delay for the before and after conditions.

Table 35 provides the field-measured travel times for the major road approaches for the MN–65 at Viking Boulevard NE signalized RCUT. Table 36 lists the minor road approaches' travel times. Table 37 provides the start and end points for the travel times.

Table 35. Field-measured travel times for major road approaches for MN–65 at Viking Boulevard NE, Minnesota signalized RCUT.

Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
NB left	a.m.	89	49	45
NB left	Off	129	93	28
NB left	p.m.	242	128	47
NB through	a.m.	81	71	13
NB through	Off	125	54	57
NB through	p.m.	163	67	59
SB left	a.m.	83	71	15
SB left	Off	63	77	−22
SB left	p.m.	63	101	−60
SB through	a.m.	71	80	−13
SB through	Off	94	57	39
SB through	p.m.	88	84	5

Table 36. Field measured travel times along the minor road approaches for MN–65 at Viking Boulevard, Minnesota signalized RCUT.

Move	Time	B_TT(Ave)	A_TT(Ave)	Percent *R*
WB left	a.m.	130	66	96
WB left	Off	55	194	−72
WB left	p.m.	84	97	−13
WB through	a.m.	153	82	87
WB through	Off	89	86	4
WB through	p.m.	103	105	−1
EB left	a.m.	111	108	3
EB left	Off	69	119	−41
EB left	p.m.	157	85	86
EB through	a.m.	89	86	3
EB through	Off	106	71	50
EB through	p.m.	115	100	15

Table 37. Coordinates for start and end field-measured travel times for MN–65 at Viking Boulevard, Minnesota signalized RCUT.

Movement	Start Point	End Point
EB left	45.319649, −93.238455	45.322617, −93.236004
EB through	45.319649, −93.238455	45.319713, −93.234440
NB left	45.31257, −93.23536	45.31971, −93.24514
NB through	45.31257, −93.23536	45.3269, −93.23649
SB left	45.3269, −93.23649	45.31964, −93.23278
SB through	45.3269, −93.23649	45.31239, −93.23584
WB left	45.319713, −93.234440	45.317878, −93.236327
WB through	45.319713, −93.234440	45.319649, −93.238455

For NB and SB movements (table 35), the travel time improved for most of the movements and periods of the day. For example, NB left saw about 45 percent reduction in travel time during the a.m. and p.m. peak periods, and a more modest improvement of 28 percent during the offpeak periods. However, the SB left turns are an exception to improved travel time. SB left turns experienced longer travel times, especially during the evening in the after period.

Most minor road approaches and movements also experienced improved travel times with the installation of the signalized RCUT. For example, during the a.m. peak, the WB through had an 87 percent reduction in travel time and the EB through had a 3 percent reduction. The EB left-turn movement saw minimal improvement of 3 percent in travel time during the a.m. peak, but a sizable improvement in travel time of 86 percent during the p.m. peak. WB left turns during the a.m. peak experience a 96 percent reduction in travel time.

Pedestrian Walking Paths

The Viking Boulevard NE intersection includes pedestrian facilities in the format of an angled crosswalk down the middle of the central island. However, at the southeast corner of the intersection is a guardrail that does not easily accommodate pedestrian traffic. This feature appears to be a design for future conditions, because there is currently not a safe way for the pedestrian to navigate this intersection access at this strategic location. The grade beyond the guardrail is extremely steep and feeds into a seasonal waterway. Due to the site limitations and the inability to have data collection personnel safely access the intersection, the research team was not able to assess the effect the alternative intersection has on pedestrian walking paths.

Performance Measures by Signalized Minnesota RCUT Site

Based on the information for the signalized RCUT at the intersection of MN–65 and Viking Boulevard NE, table 38 reflects the observed performance of the intersection improvements. Even though the traffic volume for the intersection increased in the after period by about 17 percent (9 percent for the minor road approaches), the minor road queues and the travel times decreased.

Table 38. Performance for Viking Boulevard NE at MN–65 signalized RCUT comparing after to before.

Performance Measure	Viking Boulevard NE
Queues, minor road all movements	Reduced
Queues, major road left and through	Reduced
Travel times, major road	Reduced for most movements
Travel times, minor road	Reduced for most movements
Pedestrian walkability	Poor—unique for this site

Note: The traffic volume increased by about 17 percent for the overall intersection (9 percent for the minor road approaches) in the after period.

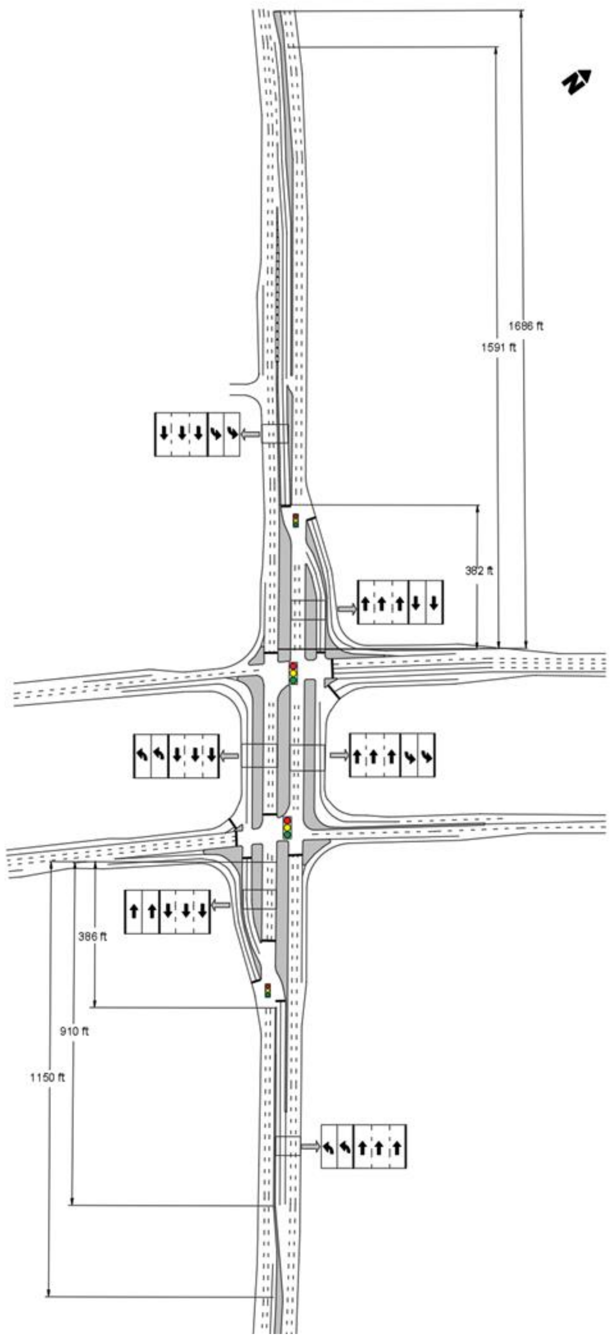
TEXAS INTERCHANGE

Stakeholders considering constructing interchanges such as the diverging diamond often express concern that drivers are switched, so that the driver is in the lane that is commonly the opposing through lane. This idea also creates some concern that one direction of travel is bounded by cars going in the other direction, and that this situation can be confusing, particularly at intersection locations along the corridor. A new alternative interchange configuration called the displaced left diamond addresses this concern. In the displaced left diamond, through traffic remains in their typical lanes, and a DLT is added with the intersection to the left-turn upstream of the signalized intersection.

Texas has a mature frontage road system, and often a Texas turnaround (sometimes called the Texas U-turn) is positioned under a bridge to help facilitate U-turns for the frontage road. The DLT interchange uses this U-turn space as a path for the separate turn lane. Though this research project was focused on intersections and not interchanges, FHWA gave special permission to the team to include this interchange in the project. The following sections summarize some of the observations the team made for this innovative interchange treatment.

General Site Characteristics

The interchange at SH–16 (Bandera Road) and West Loop 1604 Access Road is subject to high traffic volumes, particularly during the peak period. The total volume for a.m. and p.m. peak has slightly reduced in the after period following completion of construction. The site is located near shopping, including a high-volume grocery store at the northeast quadrant. The initial intersection configuration was a traditional diamond interchange with Texas turnarounds present at each end of the bridge. Figure 59 is a schematic of the revised interchange configuration. To enable the DLT movement, the Texas DOT modified the terminal intersection between the frontage road and off-ramp and the cross street.



Source: FHWA.

Figure 59. Illustration. Texas design.

Traffic Volumes

The construction of the DLT interchange can facilitate higher volumes of left-turn traffic. Table 39 shows the traffic volumes are at their highest during the p.m. peak periods. Also during this p.m. period, the peak appears to extend beyond a single peak hour. These volumes represent total interchange volume. The reason why the volumes after construction were so much less than before construction is unclear.

Table 39. San Antonio, TX, before and after periods for a.m. and p.m. peak hours.

SH–16 (Bandera Road) at West Loop 1604 Access Road, San Antonio, TX	Peak Hour a.m. Time	Peak Hour a.m. Total Volume (vehicles)	Peak Hour p.m. Time	Peak Hour p.m. Total Volume (vehicles)
Before volumes (May 2017)	7:15–8:15 a.m.	7,004	4:15–5:15 p.m.	9,773
After volumes (May 2021)	7:15–8:15 a.m.	4,973	4:45–5:45 p.m.	7,042

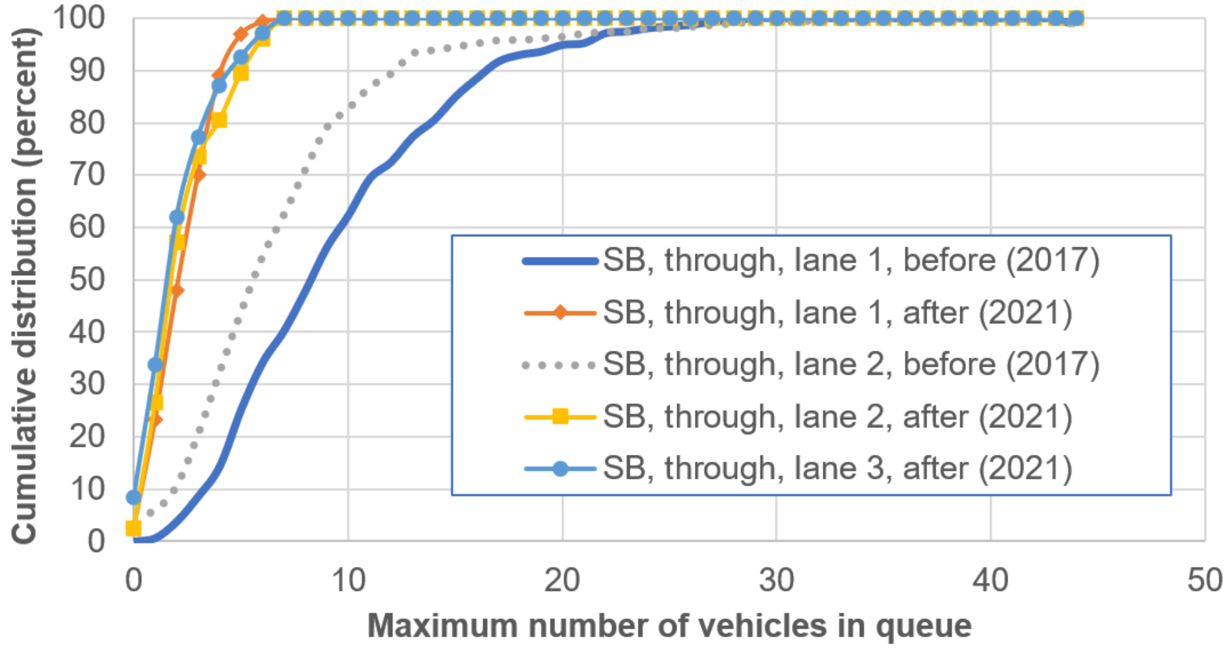
Vehicle Queues

One useful assessment strategy is comparing the number of queued vehicles at a site where extensive delays contribute to minimizing the operations of a facility. Table 40 provides the results for SH–16 (Bandera Road) at West Loop 1604 Access Road for the queue length representing 85 percent of the signal cycles that had queues of that length or longer. For SH–16 (Bandera Road), the longest queues were associated with NB and SB throughs, although the neighboring left turns also have long queue lengths. The installation of the DLTs created between 57 and 85 percent reduction in the through queues, and between 18 and 56 percent reduction in the left-turn queue going from SH–16 (Bandera Road) onto West Loop 1604 Access Road.

Table 40. Comparison of before and after queue lengths for SH–16 (Bandera Road) at West Loop 1604 Access Road, San Antonio, TX.

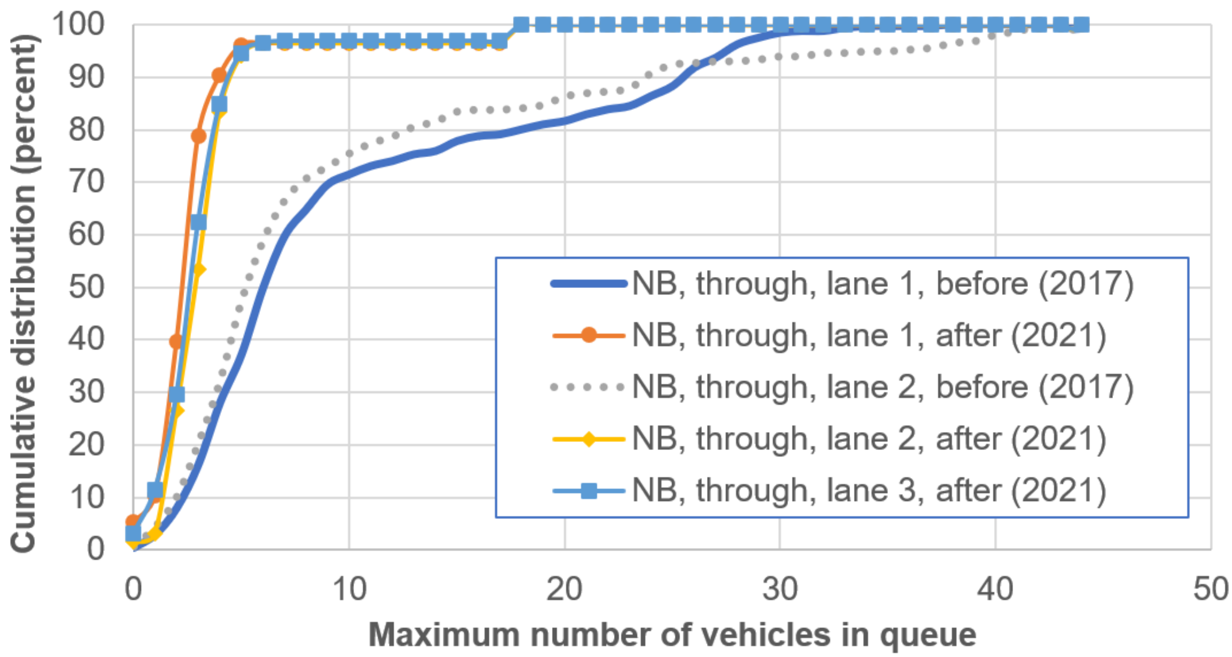
Movement	Direction	Lane	Number of Vehicles Before (2017)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Left	NB	1	12.55	10.34	18
Left	NB	2	11.53	9.46	18
Left	SB	1	15.00	6.59	56
Left	SB	2	12.94	7.63	41
Through	NB	1	23.29	3.53	85
Through	NB	2	19.18	4.14	78
Through	NB	3	NP	4.00	Lane Added
Through	SB	1	15.00	3.78	75
Through	SB	2	10.54	4.50	57
Through	SB	3	NP	3.78	Lane Added

Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated in another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).



Source: FHWA.

Figure 60. Graph. Distribution of maximum queue length for SH–16 (Bandera Road) at West Loop 1604 Access Road SB through by period and lane, Texas.



Source: FHWA.

Figure 61. Graph. Distribution of maximum queue length for SH–16 (Bandera Road) at West Loop 1604 Access Road NB through by period and lane, Texas.

Travel Times

The research team conducted a series of travel time runs to determine changes in travel time along SH–16 (Bandera Road). Table 41 provides the travel times by NB and SB movement and by period. During the a.m. peak period, the travel time increased for the NB left and through and SB left movements on SH–16 (Bandera Road). The SB left movement increased by 77 percent, from an average of 95 to 169 s. However, during the p.m. peak period only the SB left travel time increased (by 10 percent), and all other travel times were reduced (NB left, NB through, and SB through). The p.m. peak period experienced the heaviest traffic flow. Even with the p.m. peak being the heaviest flows, the NB left and through experienced a decrease in travel time of 23 or 8 percent, respectively. The SB through also experienced a reduction in travel time of 41 percent in the p.m. peak hours. Table 42 provides the start and end coordinates for the travel time runs.

Table 41. San Antonio, TX, intersection field-measured travel times.

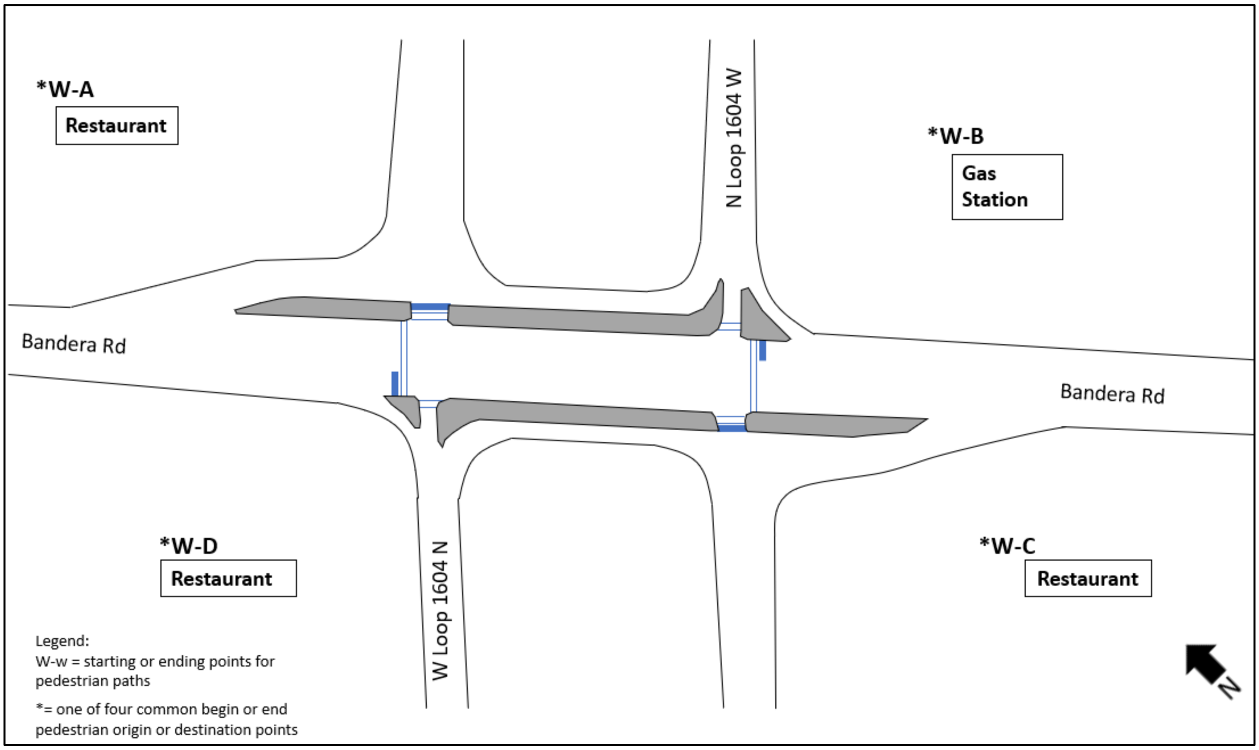
Movement	Time	B_TT(Ave)	A_TT(Ave)	Percent ***R***
NB left	a.m.	114	129	−13
NB left	Off	165	143	14
NB left	p.m.	165	127	23
NB through	a.m.	87	91	−5
NB through	Off	73	94	−28
NB through	p.m.	118	108	8
SB left	a.m.	95	169	−77
SB left	Off	110	222	-101
SB left	p.m.	213	234	−10
SB through	a.m.	101	91	10
SB through	Off	67	95	−42
SB through	p.m.	244	143	41

Table 42. Coordinates for start and end field-measured travel times for Texas intersection.

Movement	Start Point	End Point
NB left SH–16 (Bandera Road)	29.5573, −98.6717	29.5542, −98.6657
NB through SH–16 (Bandera Road)	29.5573, −98.6717	29.5503, −98.6638
SB left SH–16 (Bandera Road)	29.5515, −98.6648	29.5531, −98.6688
SB through SH–16 (Bandera Road)	29.5503, −98.6638	29.5573, −98.6717

Pedestrian Walking Paths

As noted for three of the Arizona sites and both Virginia sites, a pedestrian walking path study can provide important information about quality of service for these users. In many cases, a corridor is congested and movements to improve that location are a priority. To assess how this new interchange configuration may impact pedestrians, the research team conducted walking studies and documented the differences in pedestrian travel time from each origin and destination. Figure 62 and table 43 show the result of this walking study.



Source: FHWA.

Figure 62. Illustration. Pedestrian crossing points for Bandera Road at West Loop 1604 North, San Antonio, TX.

Table 43. Pedestrian crossing distances and times for SH–16 (Bandera Road) at West Loop 1604 Access Road, Texas.

Origin	Destinatio n	Distance Before Change (ft)	Average Walking Time Before Change (min:s)	Distance After Change (ft)	Average Walking Time After Change (min:s)	Change in Walking Travel Time (min:s)
W-A	W-D	1,239	6:41	1,385	9:08	+2:27
W-D	W-A	1,239	5:28	1,428	11:28	+6:00
W-A	W-B	1,166	6:28	1,852, 1,624	11:43	+5:15

Origin	Destinatio n	Distance Before Change (ft)	Average Walking Time Before Change (min:s)	Distance After Change (ft)	Average Walking Time After Change (min:s)	Change in Walking Travel Time (min:s)
W-B	W-A	1,415	8:46	1,852, 1,624	12:59	+4:13
W-A	W-C	1,942, 1,965	8:51	2,529, 2,561	18:08	+9:17
W-C	W-A	1,942, 1,965	8:11	2,529, 2,561	18:26	+10:15
W-B	W-C	1,090	5:45	1,995	13:36	+7:51
W-C	W-B	1,090	6:01	1,995	14:26	+8:25
W-B	W-D	1,259	8:00	1,711	12:06	+4:06
W-D	W-B	1,252	6:54	1,711	12:32	+5:38
W-C	W-D	1,928	7:22	2,210	12:20	+4:58
W-D	W-C	1,928	6:22	2,210	10:27	+4:05

For every origin-destination walking pair at the San Antonio, TX, study site there was a change in walking travel time by as little as 2 min, 27 s to as much as 10 min, 15 s. This finding is unfortunate and suggests that some attention should be given to access for pedestrians at this type of interchange for future designs.

Performance Measures for the Texas Site

Based on the travel time and vehicle queues observed for the San Antonio, TX, site, this interchange does appear to improve vehicle operations. However, the impacts to pedestrian walking paths are not suitable, and remedies should be explored for this treatment at future installations. These performance measures are depicted in table 44.

Table 44. Performance of the Texas site—Comparing after to before.

Performance Measure	San Antonio, TX
Queues, SH–16 (Bandera Road) left turn	Improved (reduced)
Queues, SH–16 (Bandera Road) through	Improved (reduced)
Travel times SH–16 (Bandera Road)	Mix NB left, NB through, and SB through reduced during p.m. peak
Pedestrian walking path	Appears to compromise

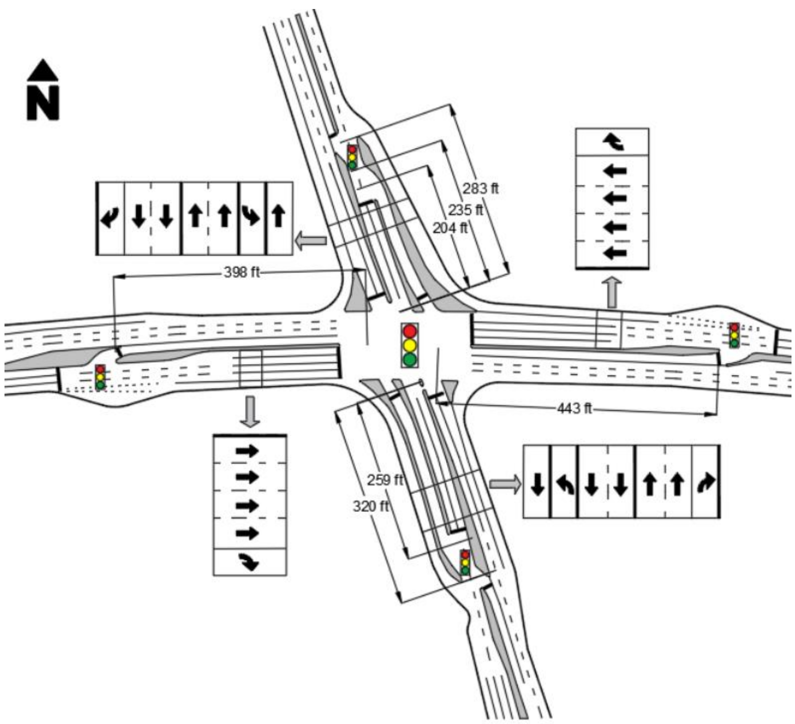
Note: The intersection traffic volume at the Texas site was less in the after period by about 29 percent.

VIRGINIA INTERSECTONS

The research team studied two hybrid innovative intersections located in Virginia. The first site is in Virginia Beach, VA, at the intersection of Kempsville Road and Indian River Road. The second site is in Norfolk, VA, and is located at U.S. 13 and VA SR–165 (also known as Northampton Boulevard).

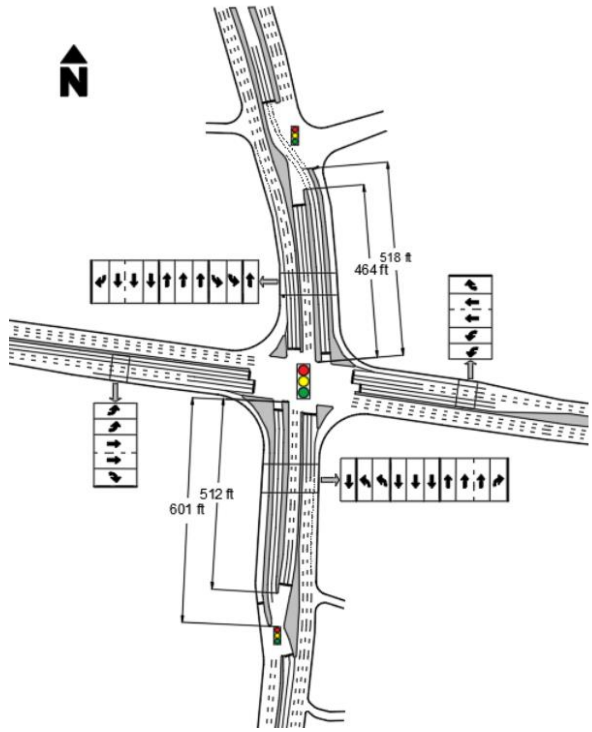
General Site Characteristics

Both study sites are subject to high traffic volumes. The Virginia Beach, VA, site (figure 63) has a heavy morning and late afternoon rush hour. The Norfolk, VA, site (figure 64) is located near major shopping and often experiences a midday queue. These two sites have some creative, innovative treatments. Virginia Beach, VA, converted a traditional intersection that had high NB to WB movements. Their approach for this intersection combined indirect left turns and DLTs that typically shift upstream of the intersection (for the high peak hour condition). The Norfolk, VA, site uses DLTs on the NB and SB approaches, and the research team focused on the performance of these legs.



Source: FHWA.

Figure 63. Illustration. Schematic of Virginia Beach, VA, design.



Source: FHWA.

Figure 64. Illustration. Schematic of Norfolk, VA, intersection.

Traffic Volumes

The construction of RLTCI as well as DLTs (located upstream of the intersection) is expected to help reduce left-turn crashes at the core intersection while also streamlining operations for traffic flow. Table 45 lists the total before and after peak hour volumes observed at the intersection of Kempsville Road and Indian River Road. As table 45 shows, the higher volumes occurred during the p.m. peak along these corridors during the before and after period.

Table 45. Virginia peak hour a.m. and p.m. traffic volumes.

Site	Analysis Period	Peak Hour a.m. Time	Peak Hour a.m. Total Volume (vehicles)	Peak Hour p.m. Time	Peak Hour p.m. Total Volume (vehicles)
Norfolk, VA	Before	7:15–8:15 a.m.	4,851	4:45–5:45 p.m.	5,745
Norfolk, VA	After	7:45–8:45 a.m.	3,951	5:00–6:00 p.m.	5,020
Virginia Beach, VA	Before	7:30–8:30 a.m.	6,825	5:00–6:00 p.m.	7,656
Virginia Beach, VA	After	7:00–8:00 a.m.	7,055	4:45–5:45 p.m.	8,123

Vehicle Queues

The research team acquired 12 h of traffic volume and queue data at the Virginia study sites in each of the before and after periods. Table 46 provides the results for the Norfolk, VA, site for the queue length representing 85 percent of the signal cycles that had queues of that length or longer.

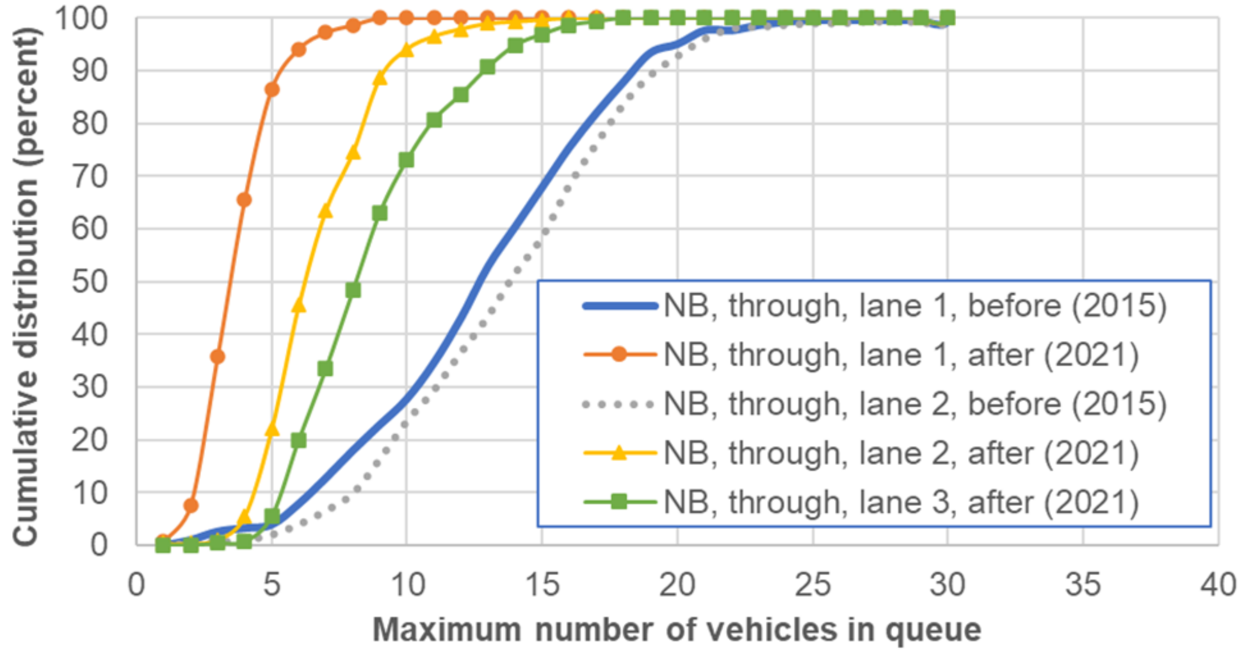
Table 46. Comparison of before and after queue lengths for Norfolk, VA, intersections.

Movement	Direction	Lane	Number of Vehicles Before (2018)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
Left	NB	1	15.76	0.00	100
Left	SB	1	7.51	0.00	100
Left	SB	2	11.28	0.00	100
Through	NB	1	16.51	3.93	76
Through	NB	2	17.24	7.74	55
Through	NB	3	NP	10.89	37*
Through	SB	1	13.57	4.76	65
Through	SB	2	16.92	7.73	54
Through	SB	3	NP	9.98	41*

*Percent reduction between new lane and the longer queue in the before period.

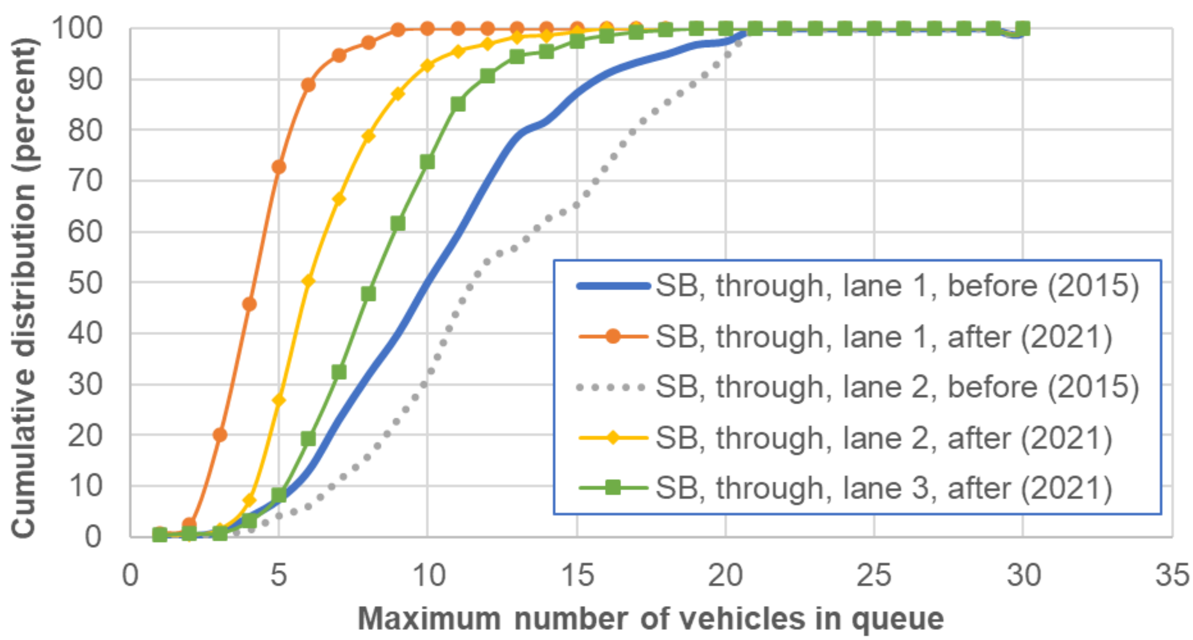
Note: Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated in another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value).

For Norfolk, VA, the through movements experienced a reduction in queues after the installation of the DLTs. In the before period, a large number of queues were 14 to 17 cars, while there were 10 to 11 cars in the after period. Overall, the through movements saw at least a 34 percent reduction in queue lengths. Figure 65 shows the distribution of maximum queue length per signal cycle for both before and after periods for the major road NB traffic. Figure 66 provides similar data for the SB traffic. As these figures illustrate, the reduction in queue lengths for the through movement throughout a typical day is noticeable. The left-turn queues at the Norfolk, VA, intersection were essentially eliminated with the addition of the DLTs. In the before period, the average left-turn queues ranged from 7.5 up to 16 vehicles during peak periods, while left-turn queues were rare in the after period. No left-turn queues were observed during the 290 signal cycles reviewed for this study.



Source: FHWA.

Figure 65. Graph. Distribution of maximum queue length for Norfolk, VA, NB through (major road) by period and lane.



Source: FHWA.

Figure 66. Graph. Distribution of maximum queue length for Norfolk, VA, SB through (major road) by period and lane.

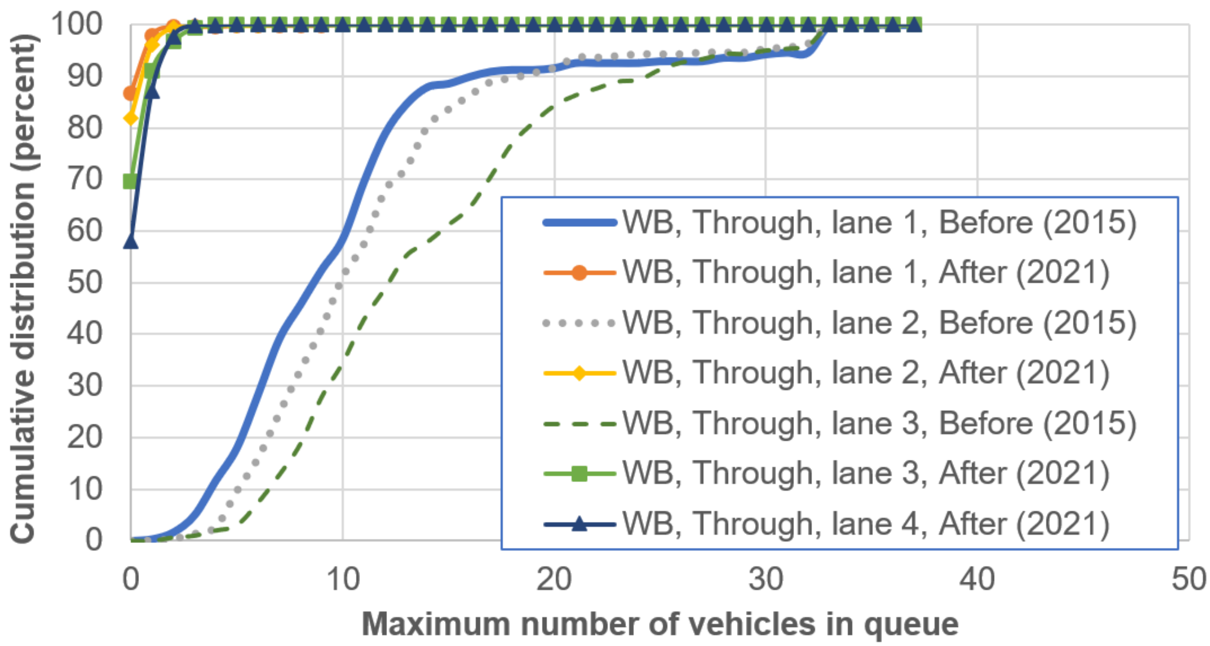
Table 47 provides the results for the Virginia Beach, VA, site for the queue length representing 85 percent of the signal cycles that had queues of that length or longer. At the Virginia Beach, VA, site, a single lane DLT replaced the dual left-turn lanes for the NB and SB approaches. Even reducing from two left-turning lanes to only one left-turning lane, the queues in the lane were smaller by 6 to 40 percent. The through movement on the north and south legs also experienced better operations, with queues that were shorter by 31 to 75 percent.

Table 47. Comparison of before and after queue lengths for Virginia Beach, VA, intersections.

Treatment	Movement	Dir	Lane	Number of Vehicles Before (2018)	Number of Vehicles After (2021)	Reduction in Number of Vehicles (percent)
MUT	Left	EB	1	3.47	Insufficient	Insufficient
DLT	Left	NB	1	7.89	6.36	40*
DLT	Left	NB	2	10.51	6.36	40*
DLT	Left	SB	1	6.68	7.13	6*
DLT	Left	SB	2	7.60	7.13	6*
MUT	Left	WB	1	13.72	6.90	50
MUT	Left	WB	2	14.75	7.81	47
MUT	Through	EB	1	9.87	0.36	96
MUT	Through	EB	2	12.27	0.30	98
MUT	Through	EB	3	9.73	0.50	95
MUT	Through	EB	4	NP	0.76	92†
DLT	Through	NB	1	20.19	11.52	43
DLT	Through	NB	2	17.83	12.39	31
DLT	Through	SB	1	29.89	7.59	75
DLT	Through	SB	2	27.64	6.91	75
MUT	Through	WB	1	13.12	0.30	98
MUT	Through	WB	2	15.52	0.22	99
MUT	Through	WB	3	20.36	0.71	96
MUT	Through	WB	4	NP	0.93	95†

*Percent reduction for the single DLT lane and the longer queue of the dual left-turn lanes in the before period.
†Percent reduction between new lane and the queue present in the before period.
Note: Dir is the direction. Number of Vehicles is the number of vehicles representing the 85th percentile maximum queue (stated another way, 85 percent of the cycles had queues of this length or shorter, or 15 percent of the signal cycles had queues longer than this value). Insufficient signifies that insufficient data were available for this movement for comparison.

The EB and WB approaches had MUTs added. The through movement on these approaches experienced large reductions in queues, 95 percent and greater. At first comparison, the before and after data for the left-turn movement on the EB approach appeared to have longer queues in the after period, compared to the before period. However, equipment challenges had resulted in only 1 h of after data available for that movement. The left-turn movement on the WB approach had shorter queues in the after period (figure 67).



Source: FHWA.

Figure 67. Graph. Distribution of maximum queue length for Virginia Beach, VA, WB through by period and lane.

Travel Times

The field-measured travel times (both before and after implementation of the alternative intersection) for the Virginia Beach, VA, intersection are provided in table 48. Table 49 lists the individual travel times for the Norfolk, VA, intersection. At locations where the queues were extensive and the data collection vehicle could only make a limited number of travel time runs, the research team chose to conduct the travel time run for the movements that appeared to be the primary contributors to the delay. Table 50 provides the start and end coordinates for each movement.

Table 48. Virginia Beach, VA, intersection field-measured travel times.

Movement	Period	B_TT(Ave)	A_TT(Ave)	Percent *R*
EB left	a.m.	81	144	−77
EB left	Off	104	118	−13
EB left	p.m.	NA	108	NA
EB through	a.m.	123	57	54
EB through	Off	38	47	−22
EB through	p.m.	NA	68	NA
NB left	a.m.	200	169	16
NB left	Off	115	83	28
NB left	p.m.	NA	109	NA
NB through	a.m.	127	121	5
NB through	Off	NA	88	NA
NB through	p.m.	NA	49	NA
SB left	a.m.	154	111	28
SB left	Off	131	88	33
SB left	p.m.	NA	126	NA
SB through	a.m.	118	39	67
SB through	Off	NA	76	NA
SB through	p.m.	NA	125	NA
WB left	a.m.	204	97	53
WB left	Off	43	127	−194
WB left	p.m.	NA	142	NA
WB through	a.m.	121	40	67
WB through	Off	104	55	47
WB through	p.m.	NA	58	NA

Table 49. Norfolk, VA, intersection field-measured travel times.

Movement	Time	B_TT(Ave)	A_TT(Ave)	Percent *R*
EB left	a.m.	NA	117	NA
EB left	Off	156	81	48
EB left	p.m.	146	86	41
EB through	a.m.	NA	82	NA
EB through	Off	72	47	35
EB through	p.m.	85	62	28
NB left	a.m.	48	92	−92
NB left	Off	78	91	−16
NB left	p.m.	183	93	49
NB through	a.m.	46	56	−22
NB through	Off	64	93	−45
NB through	p.m.	111	92	18
SB left	a.m.	91	29	68
SB left	Off	179	37	79
SB left	p.m.	85	33	62
SB through	a.m.	70	28	61
SB through	Off	62	56	10
SB through	p.m.	74	28	62
WB left	a.m.	NA	111	NA
WB left	Off	124	111	10
WB left	p.m.	215	117	46
WB through	a.m.	NA	69	NA
WB through	Off	25	72	−186
WB through	p.m.	30	104	−245

Table 50. Coordinates for start and end field-measured travel times for Virginia intersections.

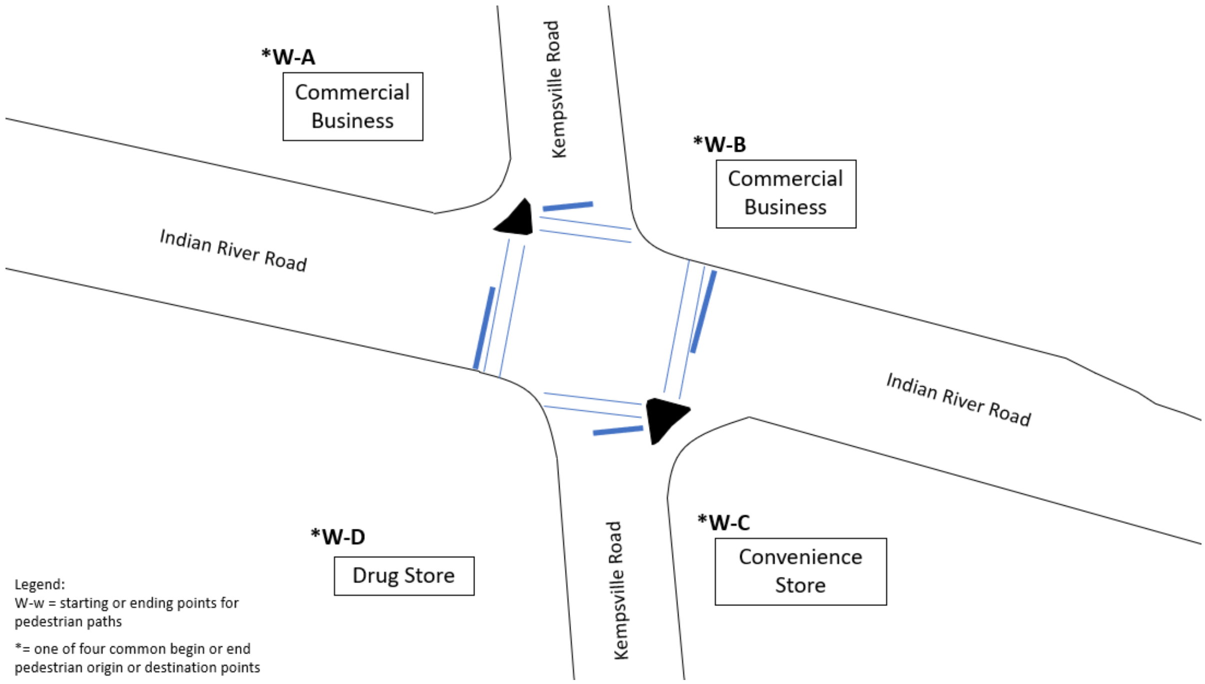
Site	Movement	Start Point	End Point
Norfolk	EB left	36.8750639, −76.21265	36.87645556, −76.2106917
Norfolk	EB through	36.8750639, −76.21265	36.87455556, −76.2091667
Norfolk	NB left	36.8720833, −76.2109472	36.8750639, −76.21265
Norfolk	NB through	36.8720833, −76.21094722	36.87645556, −76.21069167
Norfolk	SB left	36.87645556, −76.21069167	36.87455556, −76.2091667
Norfolk	SB through	36.87645556, −76.21069167	36.8720833, −76.21094722
Norfolk	WB left	36.87455556, −76.2091667	36.8720833, −76.21094722
Norfolk	WB through	36.87455556, −76.2091667	36.8750639, −76.21265
Virginia Beach	EB left	36.798875, −76.1776333	36.8002167, −76.1737417
Virginia Beach	EB through	36.798875, −76.1776333	36.79543889, −76.1721528
Virginia Beach	NB left	36.79553889, −76.1758194	36.798875, −76.1776333
Virginia Beach	NB through	36.795539, −76.1758194	36.800217, −76.1737417
Virginia Beach	SB left	36.8002167, −76.1737417	36.79543889, −76.1721528
Virginia Beach	SB through	36.800217, −76.1737417	36.795539, −76.1758194
Virginia Beach	WB left	36.7954389, −76.17215278	36.7955389, −76.17581944
Virginia Beach	WB through	36.79543889, −76.1721528	36.798875, −76.1776333

Based on an examination of the change in travel times for the Virginia Beach, VA, intersection during either the a.m. or p.m. peak period, the site experienced a reduction in travel times for most movements. The exception is EB left, where delay increased by approximately 1 min. During offpeak periods, EB left, EB through, and WB left experienced an increase in travel time.

At the Norfolk, VA, intersection, some movements had longer travel times and some had shorter travel times. The movements with the longest travel time in the before period all experienced a reduction in travel time in the after period. For example, WB left changed from an average of 215 s in the before period to 117 s in the after period. The NB left and through movements experienced slightly longer travel times in the a.m. and offpeak periods; however, travel time decreased in the p.m. period for these movements.

Pedestrian Walking Paths

As introduced earlier in this chapter, designing the road for all users is important, and so is avoiding designs that can make changes at the expense of pedestrians. A pedestrian's ability to cross the intersection in a convenient, comfortable, and efficient manner should be a priority. Pedestrians are often concerned that alternative intersection configurations might compromise pedestrian access to provide better motor vehicle access. The Virginia Beach, VA, site includes several common developments for commercial land use, including two drug stores and a convenience store. The walkability comparison can be used to assess pedestrian paths. The Virginia reduced left-turn conflict maneuvers demonstrated that assessing the impact that the intersection would have on walking time at this location is possible. The research team conducted a series of before and after walking path evaluations, where the origin and destination were each identified as the entry door of specific businesses on the corridor. Figure 68 depicts the destination points for pedestrians at the Virginia Beach, VA, site. The pedestrians for this analysis were research team members who were instructed to follow the pedestrian paths and follow all laws. If, for example, the pedestrian approached a traffic signal that was about to change, he or she was not to run to catch the green light but rather wait for the next available traffic signal that would work for the assigned destination. The research team members walked every available scenario multiple times.



Source: FHWA.

Figure 68. Illustration. Pedestrian crossing for Kempsville Road at Indian River Road, Virginia Beach, VA.

A wide variety of information can be extracted from this type of study. If the goal is to assess the total distance and the pedestrian exposure distance, an assessment table similar to table 51 (Virginia Beach, VA, site) or table 52 (Norfolk, VA, site) can be useful (see the columns labeled "distance" and "exposure distance"). This exposure distance indicates locations where the pedestrian is exposed to motor vehicles at driveway or intersection crossing locations. A small ratio of exposure distance to distance is a goal. There is no clear guidance on what this ratio should be, so determining an optimal ratio would be a good candidate for future research efforts.

Table 51. Pedestrian crossing distances and times for Kempsville Road at Indian River Road, Virginia Beach, VA.

Origin	Destination	Distance (ft)	Exposure Distance (ft)	Travel Time (number of runs)	Travel Time (min:s)
W-A	W-D	532	235	1	3:11
W-A	W-D	532	235	2	3:12
W-D	W-A	532	235	1	3:44
W-D	W-A	532	235	2	3:10
W-A	W-B	487	116	1	1:57
W-A	W-B	487	116	1	3:26
W-B	W-A	487	116	1	2:36
W-B	W-A	487	116	2	4:11
W-A	W-C	529	306	1	2:51
W-B	W-A	487	116	2	3:50
W-C	W-A	529	306	1	5:53
W-C	W-A	529	306	1	3:13
W-B	W-C	558	265	1	2:55
W-B	W-C	558	265	2	5:37
W-C	W-B	558	265	1	4:24
W-C	W-B	558	265	2	4:40
W-B	W-D	608	339	1	9:06
W-B	W-D	608	339	2	4:20
W-D	W-B	608	339	1	5:07
W-D	W-B	608	339	2	5:22
W-C	W-D	491	247	1	2:48
W-C	W-D	491	247	2	2:20
W-D	W-C	491	247	1	2:56
W-D	W-C	491	247	2	3:24

Table 52. Pedestrian crossing distances and times for Military Highway at Northampton Boulevard, Norfolk, VA.

Origin	Destination	Distance (ft)	Exposure Distance (ft)	Travel Time (number of runs)	Travel Time (min:sec)
W-A	W-D	1172	257	1	6:43
W-A	W-D	1172	257	2	6:35
W-D	W-A	1172	257	1	6:28
W-D	W-A	1172	257	2	7:27
W-A	W-B	1643	678	1	7:01
W-A	W-B	1643	678	1	7:40
W-B	W-A	1643	678	1	6:45
W-B	W-A	1643	678	2	7:16
W-A	W-C	1684	785	1	10:54
W-A	W-C	1684	785	2	9:29
W-C	W-A	1684	785	1	8:13
W-C	W-A	1684	785	1	7:24
W-B	W-C	2178	1137	1	10:01
W-B	W-C	2178	1137	2	10:56
W-C	W-B	2178	1137	1	9:31
W-C	W-B	2178	1137	2	9:19
W-B	W-D	1881	763	1	8:53
W-B	W-D	1881	763	2	9:20
W-D	W-B	1881	763	1	8:52
W-D	W-B	1881	763	2	8:54
W-C	W-D	1652	640	1	6:27
W-C	W-D	1652	640	2	8:14
W-D	W-C	1652	640	1	6:41
W-D	W-C	1652	640	2	7:02

Performance Measures for the Virginia Sites

Based primarily on the travel time and vehicle maneuver information observed for the two Virginia sites, both offer benefits that improve mobility at the study locations. These performance measures are depicted in table 53.

Table 53. Performance of the Norfolk, VA, and Virginia Beach, VA, sites—Contrasting after to before.

Performance Measure	Norfolk, VA, Site	Virginia Beach, VA, Site
Queues, left turn	Improved (reduced)	Improved (reduced)
Travel times	Generally improved	Consistently improved
Pedestrian walking path	Maintained	Maintained

Note: The traffic volumes at the Norfolk, VA, site decreased by about 9 percent. The traffic volume at the Virginia Beach, VA, site increased by about 4 percent.

CHAPTER 6. OBSERVED ROAD USER BEHAVIOR

During the project planning stages, community stakeholders often express concern that alternative intersection configurations may be confusing for prospective road users. This confusion can then contribute to an increase in crashes or a decline in operations. Based on the field data acquired and evaluated for this study, the alternative intersections generally perform in a straightforward manner; however, there are a few notable field observations that indicate potential problems due to driver error or misinterpretation of the intersection configuration. These items are generally summarized as follows:

- Major road vehicle using the dedicated left-turn lane at the RCUT primary intersection to make a U-turn.
- Apparent lane selection confusion by vehicles interacting with city buses.
- Confusion over lane changes or lane selection.
- Access management challenges near crossover intersections.
- Bicycle conflicts with motor vehicles.

The following sections further clarify these issues.

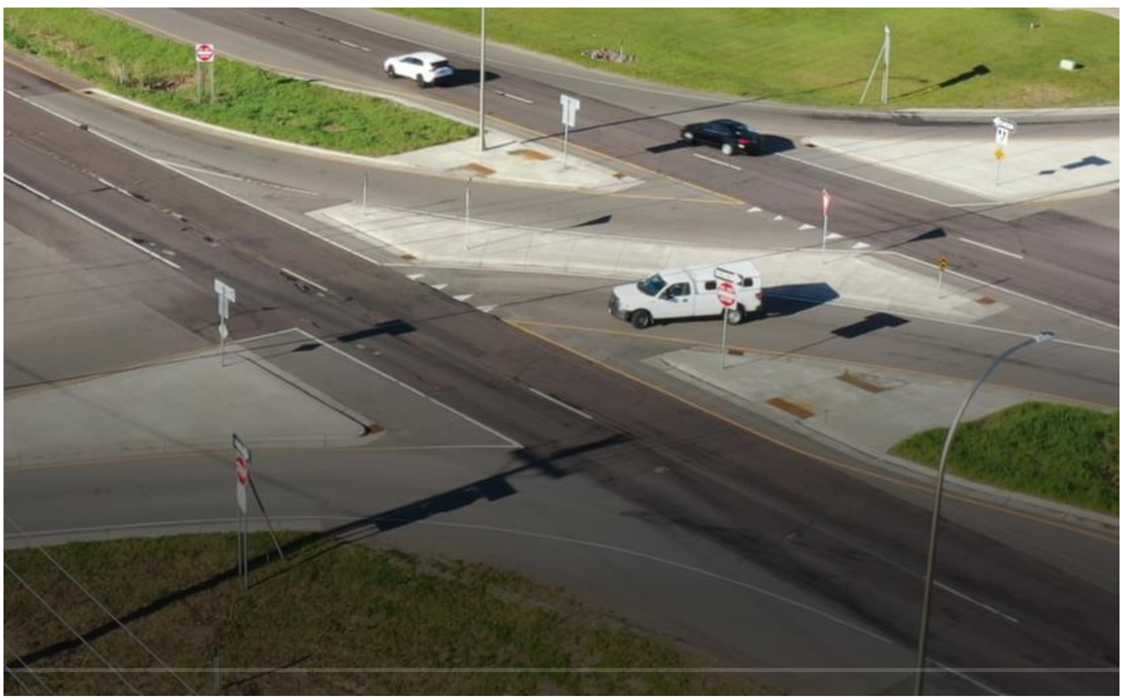
U-TURN VIOLATION FROM DEDICATED LEFT TURN AT RCUT PRIMARY INTERSECTION

The RCUT restricts direct left turns and direct through movements from a minor leg to a major leg. Instead, left-turn maneuvers and through maneuvers are accomplished when a vehicle that originates on the minor leg turns right, makes a U-turn on the major road, and then proceeds straight (to complete the left turn) or turns right (to complete the through movement).

For maneuvers from a major leg to a minor leg, the RCUT provides a channelized direct left turn. To facilitate the minor-to-major right-turn maneuver, U-turns are typically restricted in the channelized direct left turn for the primary intersection location. Figure 69, figure 70, and figure 71 show an example from Minnesota 157th Avenue NE of how this no U-turn restriction at the primary intersection can be violated. This violation introduces potential conflicts with the minor road right turn.

Source: FHWA.

Figure 69. Photo. Van entering RCUT lane (signs prohibiting U-turn are present).



Source: FHWA.

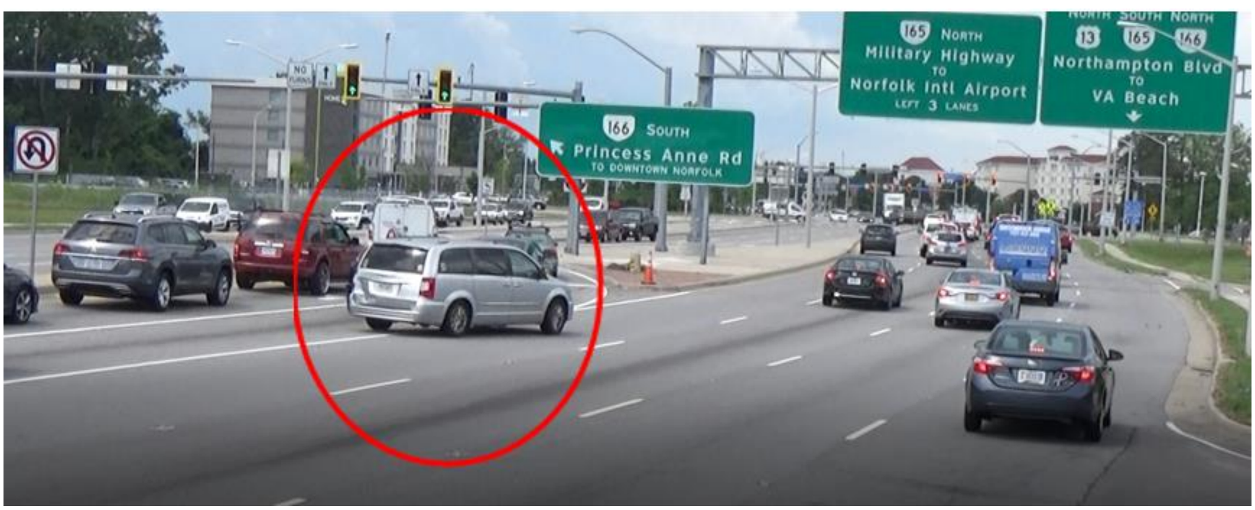
Figure 70. Photo. Van maneuvering through dedicated RCUT lane.

Source: FHWA.

Figure 71. Photo. Van completes illegal U-turn instead of continuing onto minor road.

LANE SELECTION CONFUSION

Alternative intersections, if not clearly marked, can present confusion to a driver. In some cases, this confusion can result in unexpected lane changes or vehicles getting “trapped” in the wrong lane unexpectedly. Figure 72 shows one of the Norfolk, VA, crossover intersections, with a vehicle crossing the solid white stripe and maneuvering out of the turn lane at the last minute.

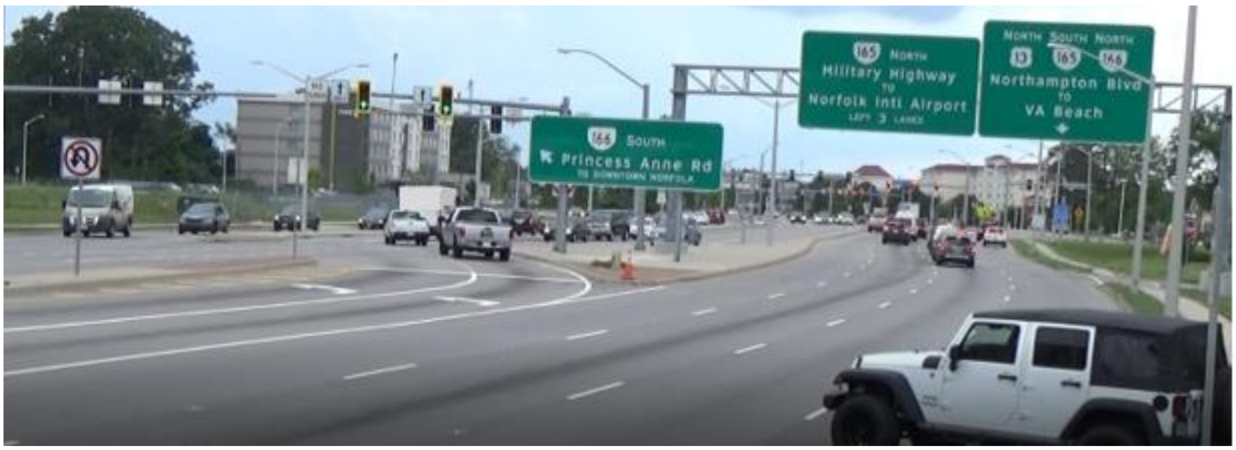


Source: FHWA.

Figure 72. Photo. Example of trapped vehicle crossing solid white stripe and exiting crossover intersection queue.

ACCESS MANAGEMENT NEAR CROSSOVER INTERSECTIONS

Renovation or rehabilitation of an existing road can introduce concerns for adjacent business owners. Often the loss or relocation of a driveway can be perceived as a potential future loss in business. Consequently, the removal of a driveway can be controversial. For alternative intersections, however, the retrofit design may mean that there is little flexibility in driveway placement. If the driveway location is positioned in a way that road users can compromise the alternative intersection effectiveness, then consideration should be given to removing the driveway. Figure 73, figure 74, and figure 75 demonstrate an access management challenge for one of the approaches at the Norfolk, VA, site.

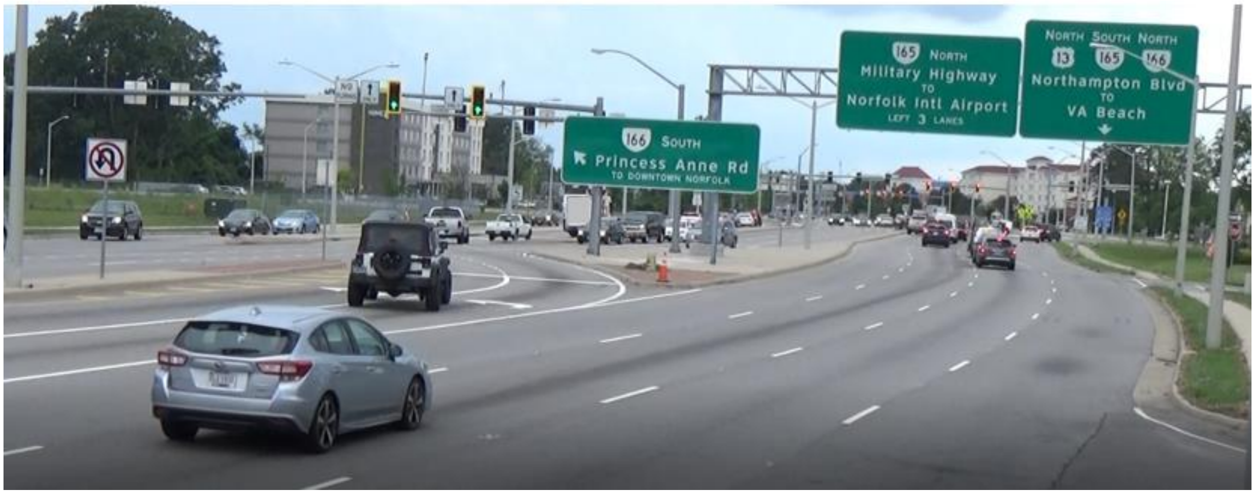


Source: FHWA.

Figure 73. Photo. Vehicle entering road from driveway at an angle that is almost perpendicular to the road.

Source: FHWA.

Figure 74. Photo. Vehicle crossing the road to access the crossover intersection.

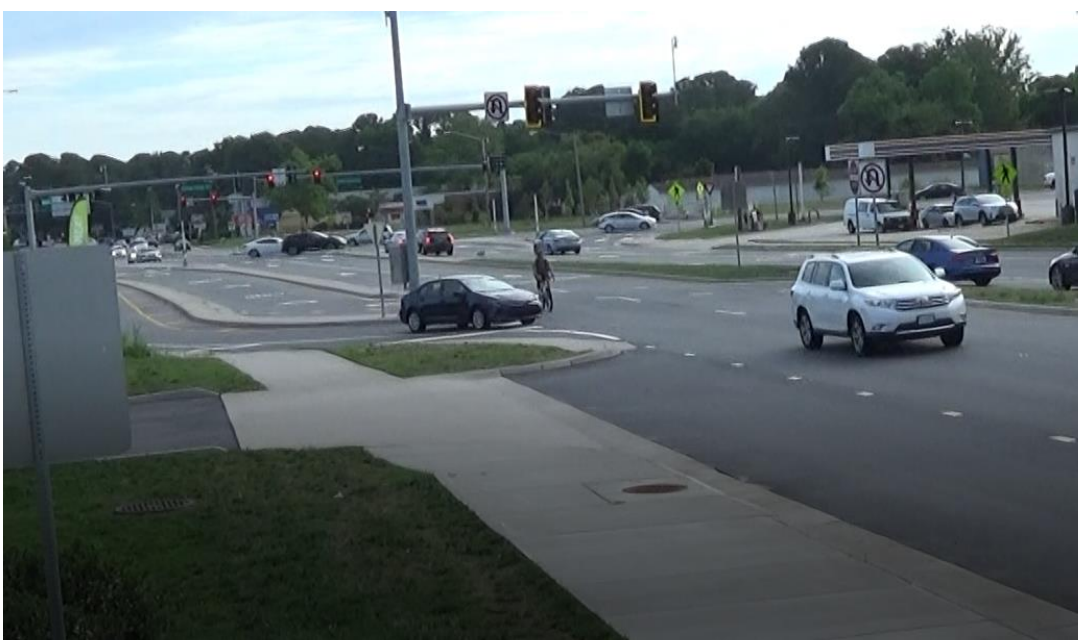


Source: FHWA.

Figure 75. Photo. Vehicle successful (this time) in completing maneuver.

BICYCLE CONFLICTS WITH MOTOR VEHICLES

Many alternative intersections provide dedicated facilities for bicycles to travel through the corridor. In some instances, a location has historically been heavily congested during the peak hour, resulting in a corridor that is less desirable for bicycles. Both drivers and bicyclists need to be acutely aware of each other at these alternative intersection locations. Figure 76 depicts an example of one bicyclist traversing the Virginia Beach, VA, site. At this location, the motor vehicle was executing a right turn on yield as the vehicle exited the crossover intersection. At the same time, a bicyclist with right-of-way was traveling SB, but the automobile continued forward. Ultimately the bicyclist had to take evasive action.



Source: FHWA.

Figure 76. Photo. Conflict between bicycle and motor vehicle.

CHAPTER 7. CONCLUDING FINDINGS AND RECOMMENDATIONS

In recent years, transportation agencies have been implementing alternative intersection configurations using innovative treatments that may help to alleviate critical congestion pinch points along the roadway network. For this study, the research team conducted a before-and-after analysis of several newly constructed innovative intersections. This report summarized the treatment selection process, reviewed the type of data and associated performance measures, added unique assessments such as pedestrian path analysis (if deemed appropriate), and ultimately summarized the site information in a performance measure summary. Initially the research team identified six candidate sites that met the collection criteria. FHWA then extended the study to include six additional sites. Because of the timing of construction for several sites with this project's contract, before and after comparisons were done for nine intersections, and three additional intersections were selected to be used as comparisons.

TYPE OF INNOVATIVE INTERSECTIONS

Each innovative intersection can be expected to include some common, traditional elements as well as alternative treatments. A common operational issue may be that the intersection initially has saturation at left-turn locations. For this challenge, agencies that have medians can alter the left turn by physically restricting the left turn from the major to the minor road through the use of deceleration lanes, acceleration lanes, and a U-turn. For the RCUT configuration, the Minnesota corridor uses this U-turn treatment at unsignalized as well as signalized intersections. For the Arizona sites, this report uses the term MUT; however, Arizona uses the terms RLTCI or ThruU. This treatment requires left-turning vehicles to proceed straight through the intersection and then execute a U-turn followed by a right turn to complete the maneuver. The Arizona treatment differs from the RCUT because the Arizona treatment permits through and left turns from the minor road, a movement not provided by the RCUT. For this reason, the Arizona RLTCI also enforces the U-turn restriction through the use of regulatory signs.

The other common treatment included in the study is the DLT. The DLT intersection physically shifts the left-turn movement into a new location. The Norfolk, VA, and Texas sites included DLT as the primary treatment. The Virginia Beach, VA, site, however, was a true hybrid and included indirect as well as DLT movements.

TYPE OF DATA AND ASSOCIATED PERFORMANCE MEASURES

To fully evaluate these intersection elements, the type of data and associated performance measures vary based on the purpose for the modification. A core data set includes site and traffic characteristics. The research team used traditional field studies to assess performance of the revised intersections. The team collected the following data in the field:

- Vehicle counts for 12 h acquired from camera video and later used for queue and throughput analysis.
- Multiple motor vehicle travel time runs.
- Field data inspection and collection of site characteristics.
- Pedestrian paths and before-and-after walking times.

The pedestrian path assessment applied to locations with pedestrian activity, where the adjacent land was already developed (representing a mature road network), and where research team members could safely traverse the route on foot.

ADDITIONAL DRIVER BEHAVIOR TRENDS

In addition to the computational metrics, the project team observed some common violations or driver errors that occurred at the various sites. These violations and errors included:

- Major road vehicle using the dedicated left-turn lane at the RCUT primary intersection to make a U-turn.
- Apparent lane selection confusion by vehicles interacting with city buses.
- Confusion over lane changes or lane selection.
- Access management challenges near crossover intersections.
- Bicycle conflicts with motor vehicles.

Chapter 6 includes example photographs of these scenarios.

FINDINGS

The key findings by intersection form are as follows:

- The MUTs in Arizona are used so that the major road drivers who want to turn left are required to go straight at the main signalized intersection and then use a U-turn (also signalized) to return to the main intersection, where the drivers would then turn right to complete their left turn. For the two sites included in the before-after analysis, the queues and travel times for the major road through movements were reduced in the after period, even with an increase in volume.

- Another intersection studied in Arizona added a quadrant road along with MUTs to a previously traditional intersection design. Even with volume increasing by 13 percent in the after period, the travel times improved, and queue lengths were reduced for most movements.

- In Minnesota, two unsignalized intersections were converted to unsignalized RCUTs. The queues along the minor road (all movements) and the major road left turns were similar in length between the before and after periods. Travel time along the minor road increased for most of the minor road movements. In other words, the unsignalized RCUTs in Minnesota overall did not show operational benefits within this study.

- The Minnesota signalized RCUT did experience notable improvements in operations. Large reductions in queues were measured for all movements on the minor road and for the major road left and through movements. Travel times on both the major and minor road were reduced for most movements.

- The DLT installed in Texas resulted in shorter queues but had a mix for travel times. In some cases, the travel time was reduced, but in others a longer travel time was present.

- The Norfolk, VA, site had a DLT which improved (reduced) both queues and travel times.

- The Virginia Beach, VA, site is a hybrid intersection with DLTs on the north and south approaches and MUTs on the east and west approaches. Improvements were experienced both in terms of reduced left-turn queues and travel times.

CONCLUSIONS

Overall, the conversion of traditional intersections to innovative intersections does tend to improve operations at locations where there are heavily congested movements, such as a left turn that then takes the driver toward a freeway and heavy commuter traffic (see the Virginia Beach, VA, site for an example). Inspection of the data demonstrates a general reduction in travel time for most legs. The findings also show that queue lengths are shorter, and the queues tend to disperse more quickly.

Overall, there appears to be significant vehicular operational benefits when using the innovative intersection treatments identified for this study, with the exception of the unsignalized RCUT sites. The research efforts for the DLT in San Antonio, TX, the hybrid in Arizona, and the signalized RCUT in Minnesota identified concerns for pedestrian walkability, such as creating a longer walking path or additional conflicts for pedestrians. Therefore, there is a need to further assess how bicycles and pedestrians can be safely serviced at these types of designs. The noted concern about adverse walkability at a few sites is important and emphasizes that pedestrian and bicyclist needs must be considered early in the design process.

The team recommends that a future effort include the final field evaluation for the three College Station, TX, RCUTs that are currently under construction, and for which the before data have already been collected. The following list highlights the recommendations:

- Conduct a study to assess optimal pedestrian paths for DLT intersections.
- Conduct an after analysis for the three College Station, TX, RCUTs currently under development.

APPENDIX. PEAK HOUR TURNING MOVEMENT COUNTS

This appendix summarizes the peak hour count data acquired in the field and assembled for this analysis. Specifically for the Arizona sites, table 54 summarizes the before and after peak hour 15-min counts for Grant Road at Stone Avenue. Table 55 summarizes the peak hour 15-min counts for Grant Road at First Avenue and Grant Road at Oracle Road. Table 56 incudes the peak hour volumes for Valencia Road at Kolb Road. For the Minnesota sites, the peak hour counts for the intersection of MN–65 with 157th Avenue, 181st Avenue, 187th Avenue, 209th Avenue, and Viking Boulevard are shown in table 57 through table 60. Table 61 summarizes the peak hour volume for the San Antonio, TX, site. Finally, table 62 and table 63 summarize the peak hour volume for the Norfolk, VA, and Virginia Beach, VA, sites, respectively.

Table 54. Peak hour 15-min counts for Grant Road at Stone Avenue (Arizona).

Site	Peak Hour	Before/ After	Time	NB -L	NB -T	NB -R	NB -U	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	SB -U	WB -L	WB -T	WB -R	WB -U
Stone	a.m.	Before	7:45	31	88	a	0	59	281	a	0	32	167	a	0	54	213	a	0
Stone	a.m.	Before	8:00	14	87	a	0	45	300	a	0	20	143	a	0	47	231	a	0
Stone	a.m.	Before	8:15	16	86	a	0	34	265	a	0	37	172	a	0	34	208	a	0
Stone	a.m.	Before	8:30	7	87	a	0	31	303	a	0	29	163	a	0	45	198	a	0
Stone	a.m.	After	7:30	15	67	18	0	0	450	28	0	33	86	14	0	0	259	38	1
Stone	a.m.	After	7:45	17	71	8	0	0	373	27	3	27	111	9	1	0	260	32	1
Stone	a.m.	After	8:00	15	69	5	0	0	377	24	0	29	88	13	0	0	231	30	6
Stone	a.m.	After	8:15	12	61	19	0	0	377	19	1	31	96	19	0	0	234	46	3
Stone	p.m.	Before	16:30	27	181	a	0	49	269	a	0	29	132	a	0	32	277	a	0
Stone	p.m.	Before	16:45	14	188	a	0	45	262	a	0	44	140	a	0	35	247	a	0
Stone	p.m.	Before	17:00	36	195	a	0	47	219	a	0	51	167	a	0	22	253	a	0
Stone	p.m.	Before	17:15	23	201	a	0	51	290	a	0	27	189	a	0	39	272	a	0
Stone	p.m.	After	16:00	26	132	11	0	0	336	28	0	48	136	25	0	0	324	62	4
Stone	p.m.	After	16:15	30	115	23	0	0	338	26	2	52	126	22	1	0	289	57	12
Stone	p.m.	After	16:30	18	124	20	0	0	385	25	1	45	117	23	0	0	323	56	9
Stone	p.m.	After	16:45	21	127	17	0	0	312	23	0	37	116	29	1	0	331	66	4

L = left turns; T = through; R = right turns; U = U-turns or indirect left turns for through movements; Stone = Stone Avenue; a = right turn counts included with through count.

Table 55. Peak hour 15-min counts for Grant Road at First Avenue and Grant Road at Oracle Road (Arizona).

Site	Peak Hour	Before/ After	Time	NB -L	NB -T	NB -R	NB -U	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	SB -U	WB -L	WB -T	WB -R	WB -U
First	a.m.	Before	7:45	16	128	a	0	27	308	a	0	77	259	a	0	21	265	a	0
First	a.m.	Before	8:00	36	135	a	0	33	289	a	0	39	194	a	0	18	248	a	0
First	a.m.	Before	8:15	24	159	a	0	33	266	a	0	56	188	a	0	22	230	a	0
First	a.m.	Before	8:30	27	132	a	0	34	289	a	0	71	221	a	0	26	243	a	0
First	a.m.	After	7:45	28	99	15	0	0	391	29	3	39	157	23	1	0	267	61	1
First	a.m.	After	8:00	21	103	9	1	0	336	17	0	34	118	29	1	0	228	54	6
First	a.m.	After	8:15	25	109	18	0	0	350	35	1	57	137	34	0	0	224	37	2
First	a.m.	After	8:30	25	95	13	0	0	392	33	1	30	138	31	0	0	218	42	4
First	p.m.	Before	16:45	39	210	a	0	50	290	a	0	51	182	a	0	26	282	a	0
First	p.m.	Before	17:00	38	217	a	0	44	250	a	0	49	202	a	0	21	283	a	0
First	p.m.	Before	17:15	52	210	a	0	44	297	a	0	45	193	a	0	31	286	a	0
First	p.m.	Before	17:30	59	219	a	0	51	249	a	0	42	198	a	0	34	295	a	0
First	p.m.	After	16:30	63	153	22	0	0	339	40	1	73	188	28	0	0	282	59	9
First	p.m.	After	16:45	63	159	27	1	0	314	38	0	58	140	36	0	0	312	63	4
First	p.m.	After	17:00	51	168	18	0	0	331	34	1	55	141	24	0	0	290	57	2
First	p.m.	After	17:15	62	217	23	0	0	340	48	3	77	189	38	0	0	301	66	2
Oracle	a.m.	Before	7:30	25	74	15	0	0	410	29	4	71	123	23	0	0	245	64	0
Oracle	a.m.	Before	7:45	20	85	14	0	0	335	35	5	55	151	34	3	0	245	58	1
Oracle	a.m.	Before	8:00	22	71	6	2	0	350	25	0	69	122	24	0	0	188	56	0
Oracle	a.m.	Before	8:15	24	96	12	1	0	336	35	2	68	118	30	1	0	179	53	3
Oracle	a.m.	After	16:30	36	223	16	1	0	322	44	4	49	122	27	3	0	270	117	1
Oracle	a.m.	After	16:45	30	236	21	3	0	297	43	1	67	145	30	2	0	242	137	0
Oracle	a.m.	After	17:00	22	205	19	1	1	344	36	1	60	123	25	2	1	298	121	0
Oracle	a.m.	After	17:15	8	211	22	0	0	351	28	4	47	149	53	0	0	282	125	1

First = First Avenue; Oracle = Oracle Road North; a = right turn counts included with through count.

Table 56. Peak hour 15-min counts for Valencia Road at Kolb Road (Arizona).

Site	Peak Hour	Before/ After	Time	NB -L	NB -T	NB -R	NB -U	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	SB -U	WB -L	WB -T	WB -R	WB -U
Valencia	a.m.	Before	7:00	5	124	a	0	113	48	3	0	75	75	224	0	6	143	117	0
Valencia	a.m.	Before	7:15	10	148	a	0	165	53	6	0	67	122	277	0	7	164	134	0
Valencia	a.m.	Before	7:30	8	157	a	0	142	54	1	0	74	112	284	0	5	162	157	0
Valencia	a.m.	Before	7:45	5	161	a	0	123	36	6	0	55	113	277	0	8	147	136	0
Valencia	a.m.	After	7:00	0	126	5	8	0	131	29	4	0	123	182	5	0	169	87	5
Valencia	a.m.	After	7:15	0	149	1	13	0	151	39	4	0	175	196	2	0	189	135	2
Valencia	a.m.	After	7:30	0	205	3	14	0	157	57	8	0	202	254	4	0	189	136	8
Valencia	a.m.	After	7:45	0	168	7	10	0	176	29	8	0	174	180	4	0	159	122	3
Valencia	p.m.	Before	16:45	1	170	a	0	216	125	3	0	125	130	108	0	4	50	88	0
Valencia	p.m.	Before	17:00	2	207	a	0	203	116	5	0	134	137	103	0	9	51	90	0
Valencia	p.m.	Before	17:15	7	173	a	0	202	98	4	0	168	148	117	0	6	44	105	0
Valencia	p.m.	Before	17:30	2	168	a	0	227	122	3	0	146	140	131	0	9	40	98	0
Valencia	p.m.	After	17:00	0	145	5	12	0	343	16	6	0	297	108	10	0	59	83	6
Valencia	p.m.	After	17:15	0	120	6	8	0	371	24	11	0	278	125	10	0	67	78	5
Valencia	p.m.	After	17:30	0	256	7	44	1	295	35	11	0	297	151	7	0	63	88	4
Valencia	p.m.	After	17:45	0	141	7	12	0	266	49	9	0	326	123	6	0	62	79	5

Valencia = Valencia Road; a = right turn counts included with through count.

Table 57. Peak hour 15-min counts for MN–65 at 157th Avenue NE (Minnesota).

Site	Peak Hour	Before /After	Time	NB-L	NB-T	NB-R	EB-L	EB-T	EB-R	EB-U	SB-L	SB-T	SB-R	WB-L	WB-T	WB-R	WB-U
157th	a.m.	Before	7:00	12	131	0	3	1	24	a	0	538	4	0	3	0	a
157th	a.m.	Before	7:15	20	163	3	4	0	27	a	4	533	2	3	0	4	a
157th	a.m.	Before	7:30	20	148	3	9	0	42	a	3	520	9	3	0	1	a
157th	a.m.	Before	7:45	27	165	3	6	2	24	a	4	417	4	2	1	2	a
157th	a.m.	After	7:00	25	150	1	b	b	4	28	4	376	3	b	b	0	1
157th	a.m.	After	7:15	9	163	5	b	b	2	21	4	487	12	b	b	3	1
157th	a.m.	After	7:30	22	183	1	b	b	5	20	3	520	4	b	b	4	3
157th	a.m.	After	7:45	23	171	1	b	b	0	23	2	417	9	b	b	1	2
157th	p.m.	Before	16:15	47	567	1	9	1	23	a	5	236	0	1	0	5	a
157th	p.m.	Before	16:30	28	547	1	19	0	31	a	1	255	3	0	3	4	a
157th	p.m.	Before	16:45	42	559	0	9	0	27	a	0	269	0	0	2	0	a
157th	p.m.	Before	17:00	35	548	2	5	1	22	a	0	262	6	2	2	6	a
157th	p.m.	After	16:30	33	596	2	b	b	34	5	2	267	5	b	b	5	0
157th	p.m.	After	16:45	51	533	2	b	b	20	5	5	223	5	b	b	4	1
157th	p.m.	After	17:00	23	481	0	b	b	37	10	4	280	8	b	b	5	7
157th	p.m.	After	17:15	34	489	4	b	b	26	7	2	268	10	b	b	8	3

157th = 157th Avenue NE; b = all movements from the minor road shown in right-turn column, except those documented in the indirect U-turn.

Table 58. Peak hour 15-min counts for MN–65 at 187th Lane NE (Minnesota).

Site	Peak Hour	Before /After	Time	NB -L	NB -T	NB -R	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	WB -L	WB -T	WB -R	WB -U
187th	a.m.	Before	7:00	4	130	0	0	1	22	a	6	460	18	0	11	0	a
187th	a.m.	Before	7:15	12	163	1	4	0	25	a	3	448	15	2	3	2	a
187th	a.m.	Before	7:30	3	170	1	2	0	17	a	3	473	17	0	0	0	a
187th	a.m.	Before	7:45	11	165	1	3	3	19	a	11	382	25	1	5	6	a
187th	a.m.	After	7:00	7	133	2	b	b	20	0	1	383	10	b	b	0	7
187th	a.m.	After	7:15	9	156	6	b	b	22	1	3	437	13	b	b	2	9
187th	a.m.	After	7:30	10	190	2	b	b	19	4	5	392	20	b	b	2	6
187th	a.m.	After	7:45	10	152	3	b	b	22	3	15	330	12	b	b	0	9
187th	p.m.	Before	16:15	19	599	1	8	2	28	a	5	216	13	7	1	12	a
187th	p.m.	Before	16:30	14	584	1	9	5	25	a	10	236	8	1	2	20	a
187th	p.m.	Before	16:45	17	641	1	6	3	24	a	4	215	12	2	2	8	a
187th	p.m.	Before	17:00	12	543	0	6	1	26	a	9	233	16	1	1	8	a
187th	p.m.	After	17:00	13	523	0	b	b	25	9	12	217	15	b	b	3	15
187th	p.m.	After	17:15	16	515	2	b	b	16	8	8	231	14	b	b	1	17
187th	p.m.	After	17:30	14	499	2	b	b	24	7	7	230	8	b	b	2	12
187th	p.m.	After	17:45	13	489	1	b	b	13	6	4	201	10	b	b	3	7

187th = 187th Lane NE; a = right turn counts included with through count; b = all movements from the minor road shown in right-turn column, except those documented in the indirect U-turn.

Table 59. Peak hour 15-min counts for MN–65 at 181st Avenue NE and MN–65 at 209th Avenue NE (Minnesota) comparison sites.

Site	Peak Hour	Before /After	Time	NB -L	NB -T	NB -R	NB -U	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	SB -U	WB -L	WB -T	WB -R	WB -U
181st	a.m.	After	7:00	1	150	0	0	a	a	11	2	2	848	10	0	a	a	8	0
181st	a.m.	After	7:15	0	150	2	0	a	a	6	5	1	450	5	0	a	a	8	1
181st	a.m.	After	7:30	3	158	6	0	a	a	12	1	0	645	5	0	a	a	0	0
181st	a.m.	After	7:45	5	161	5	0	a	a	3	2	2	511	1	0	a	a	2	5
181st	p.m.	After	16:15	18	523	12	0	a	a	11	4	0	251	3	0	a	a	8	1
181st	p.m.	After	16:30	16	540	12	1	a	a	7	4	2	276	6	1	a	a	8	0
181st	p.m.	After	16:45	16	547	9	1	a	a	13	8	5	256	11	0	a	a	2	1
181st	p.m.	After	17:00	15	522	13	0	a	a	6	5	0	319	13	0	a	a	8	1
209th	a.m.	After	7:00	1	152	4	0	0	1	1	0	0	379	2	0	4	0	11	0
209th	a.m.	After	7:15	2	170	3	0	0	0	0	0	0	410	1	0	3	0	10	0
209th	a.m.	After	7:30	3	141	2	0	0	0	1	0	0	365	2	0	2	0	10	0
209th	a.m.	After	7:45	3	151	6	0	1	0	0	0	7	282	2	0	1	1	3	0
209th	p.m.	After	16:15	6	476	7	0	1	0	0	0	2	236	1	0	1	0	2	0
209th	p.m.	After	16:30	10	475	6	0	3	0	1	0	0	239	3	0	1	0	5	0
209th	p.m.	After	16:45	8	434	4	0	0	0	0	0	0	221	2	0	0	0	2	0
209th	p.m.	After	17:00	13	512	2	0	1	1	1	0	0	223	2	0	1	0	7	0

181st = 181st Avenue; 209th = 209th Avenue NE; a = right turn counts included with through count.

Table 60. Peak hour 15-min counts for MN–65 at Viking Boulevard NE (Minnesota).

Site	Peak Hour	Before /After	Time	NB -L	NB -T	NB -R	EB -L	EB -T	EB -R	EB -U	SB -L	SB -T	SB -R	WB -L	WB -T	WB -R	WB -U
Viking	a.m.	Before	7:00	20	119	15	10	17	52	0	5	388	9	50	21	12	0
Viking	a.m.	Before	7:15	10	136	20	3	14	45	0	5	423	13	63	23	32	0
Viking	a.m.	Before	7:30	22	160	14	14	20	54	0	15	356	12	49	19	13	0
Viking	a.m.	Before	7:45	10	115	24	20	20	41	0	10	320	10	36	6	9	0
Viking	a.m.	After	7:00	20	132	15	a	b	62	26	6	408	31	a	b	10	56
Viking	a.m.	After	7:15	12	168	24	a	b	79	27	9	464	24	a	b	16	62
Viking	a.m.	After	7:30	16	168	33	a	b	98	16	8	567	42	a	b	5	57
Viking	a.m.	After	7:45	14	170	26	a	b	62	17	16	319	14	a	b	7	35
Viking	p.m.	Before	16:45	71	471	42	17	32	27	0	16	235	26	19	22	15	0
Viking	p.m.	Before	17:00	62	429	63	18	20	30	0	16	148	24	30	37	0	0
Viking	p.m.	Before	17:15	70	455	70	28	34	38	0	13	164	18	27	31	0	0
Viking	p.m.	Before	17:30	56	439	49	24	25	45	0	14	193	9	31	24	0	0
Viking	p.m.	After	16:45	62	480	66	a	b	43	27	12	241	29	a	b	17	58
Viking	p.m.	After	17:00	61	473	69	a	b	64	38	9	286	45	a	b	23	63
Viking	p.m.	After	17:15	71	482	85	a	b	76	35	16	262	31	a	b	17	60
Viking	p.m.	After	17:30	61	473	69	a	b	64	38	9	286	45	a	b	23	63

Viking = Viking Boulevard NE; a = right turn counts included with through count; b = all movements from the minor road shown in right-turn column, except those documented in the indirect U-turn.

Table 61. Peak hour 15-min counts for San Antonio (Texas) before.

Site	Peak Hour	Before /After	Time	NB-L	NB-T	NB-R	EB-L	EB-T	EB-R	SB-L	SB-T	SB-R	WB-L	WB-T	WB-R
San Antonio	a.m.	Before	7:15	149	284	174	219	219	114	166	42	79	65	191	36
San Antonio	a.m.	Before	7:30	177	223	145	215	255	111	155	38	57	80	230	50
San Antonio	a.m.	Before	7:45	201	263	111	174	256	112	148	30	93	84	229	89
San Antonio	a.m.	Before	8:00	183	208	121	158	239	77	166	44	79	92	261	112
San Antonio	a.m.	After	7:30	146	179	117	106	27	85	70	185	110	47	119	89
San Antonio	a.m.	After	7:45	101	180	111	118	48	90	106	277	109	40	94	121
San Antonio	a.m.	After	8:00	134	106	67	87	27	76	105	204	93	49	104	81
San Antonio	a.m.	After	8:15	105	135	101	86	31	105	61	213	91	54	94	89
San Antonio	p.m.	Before	16:15	286	131	61	176	351	205	298	105	147	163	443	88
San Antonio	p.m.	Before	16:30	290	122	79	118	383	145	306	117	110	165	426	109
San Antonio	p.m.	Before	16:45	256	99	63	162	422	135	333	144	150	159	424	121
San Antonio	p.m.	Before	17:00	290	103	96	153	330	207	330	156	174	154	379	109
San Antonio	p.m.	After	16:45	114	208	147	100	151	111	170	431	95	103	112	69
San Antonio	p.m.	After	17:00	108	199	159	100	160	102	175	363	128	109	93	80
San Antonio	p.m.	After	17:15	109	169	144	101	138	122	157	434	94	95	77	63
San Antonio	p.m.	After	17:30	108	199	159	100	177	92	175	363	95	109	93	82

San Antonio = Texas SH–16 (Bandera Road) at West Loop 1604 Access Road.

Table 62. Peak hour 15-min counts for Norfolk, VA.

Site	Peak Hour	Before /After	Time	NB-L	NB-T	NB-R	EB-L	EB-T	EB-R	SB-L	SB-T	SB-R	WB-L	WB-T	WB-R
Norfolk	a.m.	Before	7:15	66	193	56	17	157	52	109	242	15	31	150	66
Norfolk	a.m.	Before	7:30	63	195	48	28	156	72	105	244	10	54	171	68
Norfolk	a.m.	Before	7:45	81	206	60	34	155	64	109	238	17	93	183	62
Norfolk	a.m.	Before	8:00	62	173	52	41	146	53	103	215	16	78	186	56
Norfolk	a.m.	After	7:45	39	146	30	106	250	15	42	168	47	21	148	28
Norfolk	a.m.	After	8:00	41	135	19	109	225	17	35	132	49	26	121	39
Norfolk	a.m.	After	8:15	47	126	27	108	215	16	45	166	53	27	133	25
Norfolk	a.m.	After	8:30	43	142	31	79	225	21	52	159	52	20	112	39
Norfolk	p.m.	Before	16:45	59	239	80	22	180	137	112	255	18	103	143	57
Norfolk	p.m.	Before	17:00	81	254	62	19	208	106	131	285	40	78	123	59
Norfolk	p.m.	Before	17:15	62	235	57	16	178	117	152	297	28	100	155	54
Norfolk	p.m.	Before	17:30	67	255	75	18	212	125	112	265	31	76	144	63
Norfolk	p.m.	After	17:00	44	122	53	96	297	19	83	311	87	18	99	48
Norfolk	p.m.	After	17:15	40	95	56	107	319	30	98	277	85	29	115	54
Norfolk	p.m.	After	17:30	59	120	66	92	268	28	65	329	43	30	108	45
Norfolk	p.m.	After	17:45	50	98	65	87	272	22	81	328	17	25	103	37

Norfolk = Military Highway at Northampton Boulevard (U.S. 13 at VA SR–165).

Table 63. Peak hour 15-min counts for Virginia Beach, VA.

Site	Peak Hour	Before /After	Time	NB-L	NB-T	NB-R	EB-L	EB-T	EB-R	SB-L	SB-T	SB-R	WB-L	WB-T	WB-R
VA Beach	a.m.	Before	7:30	74	229	155	48	374	40	33	110	16	68	591	24
VA Beach	a.m.	Before	7:45	80	234	145	28	433	39	38	133	13	99	516	38
VA Beach	a.m.	Before	8:00	73	251	126	35	369	37	36	127	22	102	465	26
VA Beach	a.m.	Before	8:15	73	214	156	24	300	37	26	87	13	99	536	33
VA Beach	a.m.	After	7:00	95	246	83	11	424	89	23	158	11	20	667	26
VA Beach	a.m.	After	7:15	102	265	80	5	302	81	25	159	12	28	493	31
VA Beach	a.m.	After	7:30	98	241	60	4	412	72	36	149	22	47	668	31
VA Beach	a.m.	After	7:45	75	227	62	9	394	61	24	128	12	43	721	23
VA Beach	p.m.	Before	17:00	71	201	171	41	532	103	68	205	12	124	353	43
VA Beach	p.m.	Before	17:15	71	185	195	35	455	100	54	195	9	107	445	56
VA Beach	p.m.	Before	17:30	53	196	167	28	515	96	49	179	16	120	446	49
VA Beach	p.m.	Before	17:45	60	218	186	34	483	95	70	208	13	106	374	64
VA Beach	p.m.	After	16:45	58	207	93	12	648	142	62	229	15	72	479	45
VA Beach	p.m.	After	17:00	58	232	87	9	655	112	77	214	24	75	446	39
VA Beach	p.m.	After	17:15	54	147	75	16	673	126	65	229	5	75	488	42
VA Beach	p.m.	After	17:30	58	232	87	19	655	112	77	214	24	75	446	39

VA Beach = Indian River Road at Kempsville Road.

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The map in figure 1 was modified by the authors to add a circle. The original map is the copyright property of Google® Earth™ and can be accessed at https://www.google.com/earth.[32]

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