ABSTRACT

WARCHOL O’BRIEN, SHANNON ELIZABETH. Impact of Right Turn Treatments at Diverging Diamond Interchanges and Adjacent Intersections. (Under the direction of Dr. Nagui Rouphail).

Diverging Diamond Interchanges, (DDI, or Double Crossover Diamonds) have swept the country as an innovative interchange system since the original installation in Springfield, Missouri in 2009. While the DDI provides many benefits, including reduction in conflict points and critical movements at the intersection, among others, prior research has identified common operational challenges found at some of the first DDIs built around the country. Often, the interchange is bordered on the arterial by a closely spaced adjacent intersection which can lead to queue storage challenges both downstream of the DDI and on the off-ramps.

This work sought to determine the impact five geometric treatments – channelized turn lane with an auxiliary lane, dual right turn lanes, right turn on red (RTOR), dual right turn lanes with RTOR, and a slip lane – applied to the right turn from the off-ramp had on average maximum queue spillback, average delay per vehicle, and average stops per vehicle throughout a DDI corridor. CAP-X was used to develop initial volumes for the base model. To determine the control volume under which the operational challenge of queue spillback was present, the right turn volume from the off-ramp was increased until substantial queues formed on the off-ramp as well as at the outbound through movement at the downstream adjacent intersection. Each treatment was then applied to the control model. The volume of the right turn movement was then incrementally increased to determine the extra capacity provided by each treatment. A calibrated VISSIM model of a DDI surrounded by standard, four critical phase intersections on both sides of the corridor was used for microsimulation.

The results showed some treatments – mainly dual right turn lanes and dual right turn lanes with RTOR – simultaneously increased the capacity of the right turn from the off-ramp as well as the outbound through movement at the crossover. Furthermore, the average maximum queue length at the off-ramp as well as the average delay for vehicles traveling through the corridor decreased; however, there was no improvement in delay for right turning vehicles from the off-ramp, nor was there improvement for either route in terms of number of
stops. The average maximum queue length for the outbound through movement at the downstream adjacent intersection increased under these treatments. The slip lane and RTOR treatments marginally increased the capacity of the westbound to northbound right turn from the off-ramp resulting in a decreased average maximum queue length for the movement and no change in the average maximum queue length at the downstream adjacent intersection. The slip lane and RTOR treatments marginally increased the capacity of the westbound to northbound right turn from the off-ramp resulting in a decreased average maximum queue length for the movement and no change in the average maximum queue length at the downstream adjacent intersection. The auxiliary lane treatment increased the capacity of the westbound to northbound right turn movement from the off-ramp, the northbound through movement at the outbound crossover, and the northbound through movement at the downstream adjacent intersection. In return, both queues of interest significantly decreased. These results are limited by constant vehicle turning percentages at the adjacent intersections. Varying the turning percentages at the adjacent intersection may impact the results by creating or relieving weaving issues between the off-ramp and the downstream adjacent intersection. The research is further limited in that while the base model is calibrated, the model was not recalibrated for each treatment.

This research shows the design of the DDI must be considered in the context of the entire corridor. To provide the full extent of congestion relief of which the DDI is possible, consideration of the geometrics of downstream closely spaced adjacent intersections must be included from the earliest planning stages.
Impact of Right Turn Treatments at Diverging Diamond Interchanges and Adjacent Intersections

by
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DEDICATION

- to Michael -

I never understood why people wrote dedications to their spouses until I saw how many dishes you washed and loads of laundry you switched while I was writing this.
BIOGRAPHY

Shannon Warchol O’Brien was born and raised in Saint Paul, Minnesota. On August 1, 2007, Shannon was six miles away from the collapse of the I-35W Bridge over the Mississippi River. The actions of engineers over the following hours, days, and months sparked her interest in transportation. Shannon graduated from the University of Notre Dame in May of 2013 with a Bachelor of Science in Civil Engineering and – thanks to the unparalleled dedication of the faculty in that department – an appreciation for theory-based education. While pursuing her master’s degree at North Carolina State University, Shannon conducted transportation research at the Institute for Transportation Research and Education. In May 2014, Shannon was awarded the ASCE’s Jack E. Lesich Fellowship. She currently resides in Raleigh, North Carolina with her husband.
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This work would not be possible without the direction and guidance of Dr. Bastian Schroeder and Mr. Chris Cunningham. This thesis is a spur of the much bigger study, “Exploring Corridor Operations in the Vicinity of a Diverging Diamond Interchange”, NCDOT Project No. 2014-13.

Dr. Bastian Schroder spent an untold number of hours on validation and calibration of the VISSIM model from which this work is derived, saving me an incredible amount of work.

Finally, Dr. Billy Williams and Dr. Nagui Rouphail, along with the ITRE Highway Group, brought me to NC State without any knowledge of transportation and provided the education so that I could complete this work fewer than two years later.
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TERMINOLOGY

*Base Model* – Geometrically symmetric, undersaturated DDI corridor with geometry based on the National Avenue and US-60 DDI corridor in Springfield, MO.

*Control Model* – Identical to the base model except for the volume of interest is set to the control volume

*Control Volume* – Volume for the movement of interest at which the operational challenges are present throughout the corridor

*Measures of Effectiveness of Interest* –

- average maximum queue length for the movement of interest
- average maximum queue length for the through movement at the downstream adjacent intersection
- average delay per vehicle for the corridor OD path
- average delay per vehicle for the right turn from the off-ramp through the downstream adjacent intersection OD path
- average number of stops per vehicle for the corridor OD path
- average number of stops per vehicle for the right turn from the off-ramp through the downstream adjacent intersection OD path

*Movement of Interest* – Right turn movement from the off-ramp

*Volume of Interest* – Volume of the right turn movement from the off-ramp
1 INTRODUCTION
Diverging diamond interchanges (DDI) have been implement in the United States since 2009. The DDI is characterized by two two-critical-phase signalized crossovers which direct vehicles to the left side of the road while traveling over (or under) the bridge, and then return vehicles to right side before exiting the interchange area. This research considers the DDI within the context of the surrounding corridor which includes one standard four leg intersection on either side of the arterial. FIGURE 1 identifies particular points of the corridor to be referenced in this work. The labels given to these points are in reference to the vehicle path as noted by the dark arrows.

![DDI corridor with selected terminology](image)

FIGURE 1 DDI corridor with selected terminology.

1.1 OBJECTIVES AND SCOPE
The relative novelty of the DDI, combined with its growing popularity, creates a challenge for researchers. Initial field evaluations of the first DDIs uncovered common operational challenges. Unfortunately, the popularity of the DDI soared before these studies were complete. It is critical to identify and explore potential treatments as soon as possible so that these challenges are not experienced at DDIs across the nation. Geometric treatments are particularly important because of the high cost of
implementation – between $117,000 and $187,500 for the treatments explored in this document – and disruption caused to users during construction relative to implementing and field adjusting signal timing treatments – approximately $6,000 per traffic signal (1). The earlier appropriate geometric treatments can be identified, the more DDIs can be implemented with provisions already in place to mediate queue spillback, thereby saving time, money, and user frustration in completing renovations.

While there are numerous operational challenges associated with DDIs – poor non-peak progression, demand starvation, uneven lane utilization, entrance ramp merge capacity, and ramp metering impacts (2)– queue spillback onto the freeway is also a considerable safety challenge. Driver expectancy on the freeway does not include encountering a stopped queue of vehicles spilled back from the off-ramp into the right exit lane. The queue spillback at the off-ramp cannot be considered in isolation, however (3). It is also known that queue spillback occurs from the downstream adjacent intersection into the outbound crossover. The off-ramp queues cannot be reduced if there is no storage space on the arterial corridor to which those off-ramp vehicles can move.

For those reasons, the focus of this thesis is to:

1. Determine the impact potential right-turn geometric treatments have on queue spillback at the off-ramp as well as queue spillback at the downstream adjacent intersection, and
2. Measure the excess capacity right turn treatments provide to the right turn movement relative to the control, or no-build, scenario.

1.2 PROBLEM STATEMENT AND RESEARCH CONTRIBUTION
The popularity of the DDI has outpaced academia and industry’s abilities to analyze the initial implementations of the design. The operational challenges of DDIs had to be identified before treatments to the challenges could be recommended. Because the oldest DDI in America has only been in operation for six years as of writing, much of the current research has been focused on field evaluations. As more DDIs have been implemented, studies have been conducted to identify common operational challenges. The first nation-wide study on field evaluations of DDIs, FHWA project DTFH61-10-C-
00029, is in the final stages at the time of writing. Therefore, there is limited research on how conventional treatments for queue spillback at the off-ramp impact a DDI corridor. In fact, the first DDI in operation in the United States had a yield controlled right turn from the off-ramp, implying queue spillback on the off-ramp was not expected, otherwise that movement would have been signalized. It was not until designers began to signalize that movement at subsequent DDIs – due to other safety considerations – that the queue spillback on the off-ramp became apparent (4). Given DDIs have been in operation for fewer than six years as the time of writing, and the first study investigating national trends in operational challenges has yet to be published, it is understandable that geometric treatments to operational challenges have not been widely studied.

It was known before the first DDI was installed and evaluated, however, that the signal timing of DDIs would be different than other interchanges. The largest difference is the obvious conflict between the directional through movements. In a DDI, the mainline through movements cannot progress together as shown in the time-space diagram of FIGURE 2. In a corridor with four equally spaced “standard” intersections, mainline movements can receive green time at the same time and therefore perfect progression can be achieved in both directions. In the same corridor where a DDI replaces the center two intersections, perfect progression is no longer possible. This is because when one mainline movement receives green time, the second must receive red time. Therefore, the analyst cannot provide any bandwidth to the non-peak direction if the peak direction is fully progressed.

Because this challenge was identifiable prior to implementation, research began toward the development of new signal timing strategies (5)(6-11). Thus far, these have almost exclusively been focused on coordination between the two crossovers, while efforts on coordination in a corridor context are yet to materialize.
FIGURE 2 Time-space diagram of a corridor with A) standard intersections; B) standard intersections with DDI in the middle.

Following the introduction of the DDI, initial research identified common operational challenges and provided unique signal timing options. There is a gap, however, in the range of geometric options designed to mitigate operational challenges when implementing DDIs with closely spaced intersections. It is important, therefore, to determine which treatments are useful in reducing queue spillback at the off-ramp as well as how they impact the entire corridor, not just the interchange. This research seeks to fill that gap by studying the impact geometric treatments at the off-ramp have on:

- Queue spillback at the off-ramp;
- Queue spillback from the downstream adjacent intersection;
- Total delay for vehicles throughout the corridor; and
- Number of stops for vehicles throughout the corridor.
1.3 THESIS ORGANIZATION

The organization of this thesis is as follows:

- Chapter 2 further explains the DDI and motivation for this work;
- Chapter 3 presents a literature review of prior simulation and field evaluation of DDIs as well as the impact of the treatments of interest at conventional interchanges;
- Chapter 4 presents the methodology of the study including model details, the measures of effectiveness, experimentation process, simulation procedure, treatments applied, and the statistical models used;
- Chapter 5 presents the findings of the effectiveness of varied treatments under the control volume as well as the comparison of each treatment effect under heavier traffic demand; and,
- Chapter 6 synthesizes the major findings and recommendations of the report.
2 BACKGROUND AND MOTIVATION

2.1 DDI BACKGROUND

Diverging Diamond Interchanges (DDI) were first built in France during the 1970s. Chlewicki popularized the design in America with his paper explaining the interchange at the Second Urban Street Symposium in Anaheim, California in July 2003 (12). Since the first American DDI opened in Springfield, Missouri at I-44 and MO-13, more than 30 additional DDIs have opened across the country and one in the United Arab Emirates.

Like a diamond interchange, the DDI features two signals and four access ramps. Right turns from the off-ramp to the arterial and from the arterial to the on-ramp are also similar to the diamond design. It is the design between the signalized intersections that makes a DDI unique. At the signal, inbound arterial traffic switches from the right side of the road to the left, and the outbound arterial traffic switches back to the left side. This switch allows left turns – both from the off-ramp and onto the on-ramp – to not cross opposing traffic streams but instead act similar to left turns from a one way onto a one way. A schematic of the DDI is shown in FIGURE 3.
DDIs are a popular solution because of their cost to efficiency ratio. Unlike some other alternative interchange designs, the DDI can often be built on the same footprint as an existing diamond interchange, and the existing bridge structure can be reused. When compared to a standard diamond interchange, the DDI intersections tend to be located further apart to accommodate the required curve radii. In some cases, this may require additional right of way to be acquired in the areas highlighted in red in FIGURE 4.
Despite the right-of-way acquisitions that may be necessary, the design decreases costs by reusing the existing bridge. The DDI at Interstate 435 and Front Street in Kansas City, Missouri was initially estimated to cost $5.8 million with $4.2 million in construction costs, $1.3 million in right-of-way costs, and the remainder in utility costs. This was $5.5 million less than the tight urban diamond interchange with costs of $6.9 million for construction and $3.9 million for right-of-way. Utilities brought that project to an estimated cost of $11.3 million (13).

Given a set number of lanes, the DDI provides more capacity than the standard diamond interchange, and therefore, widening of the bridge structure is typically not necessary when the DDI is installed (13). Most DDIs in operation in the United States are
designed with one bridge, but the second installation in Springfield Missouri, at National Avenue and US-60, used an existing twin bridge for the opposing traffic flows.

The reverse curves used to direct traffic through the intersection also serves as a traffic calming device. It has been shown that vehicle speeds are naturally reduced as they approach the intersection in order to maneuver through the curves (14).

From an efficiency standpoint, DDIs are most beneficial at interchanges with high left-turn volumes. The two-phase signal control provides through traffic, as well as left turns from the off-ramp, with a green-to-cycle length ratio (g/C) of nearly 0.5 while left and right turns onto the on-ramp can be free-flowing, although they are sometimes signalized for pedestrian safety. Some DDIs have unsignalized right turns from the off-ramp, while others are signalized.

At the most basic level, the DDI can run on a two-phase signalization scheme with one controller. FIGURE 5 shows a simple timing plan when both movements at the off-ramp are unsignalized. The two crossover signals feature a simple alternating pattern between the inbound (movements 2 and 8) and outbound (movements 4 and 6) crossover movements. Ring 1 corresponds to the left signal while Ring 2 controls the right signal. Movements in Ring 1 do not conflict with any movements in Ring 2, so there is no need for a barrier.
FIGURE 5 Two critical phase timing plan (14).

If the off-ramps were signalized, the timing plan could be adjusted to allow the off-ramp movements to run with the non-conflicting crossover. One downside to this timing plan is the excessive loss time caused by the long clearance phase. Vehicles exiting the interchange via movement six must clear both movement eight and movement five before phase 5+8 can commence. This can take upwards of seven seconds depending on the design of the DDI; however, it may only take three of those seven seconds to clear movement eight alone. To reduce the loss time, an overlap can be used as demonstrated in FIGURE 6, which allows movement eight to proceed once cleared. A few seconds later (in this example four seconds later) movement five can proceed. A similar overlap repeats at the conclusion of phase 5+8 and also after phases 1+4 and 2+3 on Ring 1. Because Ring 1 and Ring 2 do not have any conflicting movements, the use of a barrier is
not needed. Therefore, depending on which movement the engineer wishes to progress – options include movement 3 and 6, movement 2 and 6, movement 7 and 4, or movement 8 and 4 – the relative offset of the rings may vary from the figure shown.

FIGURE 6 Two critical phase timing plan with overlaps (14).

FIGURE 6 is just one option for DDI signal timing. Current research has focused strongly on developing signal timing strategies for the DDI. Multiple signal timing plans have been developed using various numbers of rings, barriers, critical phases, overlaps, and controllers (5)(6-11).

Adding to the attractiveness of the DDI is the reduced number of conflict points. DDIs have 14 conflict points as shown in FIGURE 7 as opposed to the 26 conflict points in FIGURE 8 at the conventional diamond interchange.
Initial objections to the interchange system have been seen from citizens often during the design and construction stages of the project. Citizens tend to find the design confusing and consider requiring drivers to move to the left side of the road to be unsafe. Jackson et al. found, however, that once a DDI opens for operation, the public soon becomes accustomed to the system (15). Furthermore, studies have shown that after the interchange becomes operational, drivers believe it leads to a reduction in delay, improvement in flow, and safer conditions than the standard diamond interchange (16).
2.2 OPERATIONAL CHALLENGES

While there are numerous operational challenges associated with DDIs – poor non-peak progression, demand starvation, uneven lane utilization, entrance ramp merge capacity, and ramp metering impacts (2) – queue spillback onto the freeway is also a considerable safety challenge. Driver expectancy on the freeway does not include encountering a stopped queue of vehicles spilled back from the off-ramp into the right lane. The queue spillback at the off-ramp cannot be considered in isolation, however (3). It is also known that queue spillback exists from the downstream adjacent intersection into the outbound crossover. This research aims to address the issue of queue spillback at the downstream adjacent intersection as well as at the off-ramp.

Queue spillback has two causes: (1) a disparity in the volume to capacity ratio for a movement resulting in repeated cycle failure and (2) a lack of sufficient storage capacity for the expected volume. The queue spillback at the off-ramp is caused by the disparate volume to capacity ratio with the spillback on the corridor is a result of both causes.

Queues at the off-ramp have been seen after unsignalized right turning movements at the off-ramp were signalized for safety concerns. In a study of four DDIs, the most frequently observed conflict was the merge at an unsignalized turning movement, particularly at unsignalized right turns from the off-ramp (3).

One cause of conflicts may be due to right-turning drivers looking down the expected lane of oncoming traffic instead of the actual lane of oncoming traffic, as exemplified in FIGURE 9. Seeing no vehicles in the expected lane of oncoming traffic, the driver attempts to merge and a collision occurs.
Others surmise the problem is actually caused by a lack of sight distance due to high median walls blocking the right-turning driver’s view of the oncoming traffic. FIGURE 10 is a collection of look back views from the perspective of a right-turning vehicle from the off-ramp. Pictures (a) and (b) have high median walls obscuring the view of oncoming traffic upstream of the crossover while (c), (d), and (e) have low or no median walls. Picture (f) is from Utah where attempts have been made to extend the distance between the crossover and the right-turn merge location.

FIGURE 9 Look back view from the right turn at the off-ramp (4).
Whatever the cause of the conflicts, the resolution has widely been to signalize the right turn from the off-ramp and prohibit Right Turn on Red (RTOR). It is the signalization, more so than the prohibition of RTOR, which reduces the capacity of the movement. This is the first cause of queue spillback: cycle failures resulting from a disparity in the volume to capacity ratio. FIGURE 11 pictures a queue along the off-ramp at DDI where there are heavy off-ramp volumes and RTOR is not permitted. By signalizing the off-ramp movements to prevent merge conflicts, a new conflict is created when drivers on the freeway unexpectedly encounter stopped vehicles at the gore of the off-ramp.

FIGURE 11 Queue spillback at the off-ramp caused by heavy demand and a RTOR prohibition (17).

While spillback onto the freeway is the result of reduced capacity, spillback from the downstream intersection into the crossover is the result of an increased volume provided by the DDI. At some of the first DDIs constructed, the driving public recognized that the greatest problem lied outside of the DDI at the adjacent intersection (15).

DDIs provide green time to three approaches at most and can serve all movements in as few as two critical phases. A standard, four-leg intersection serves four approaches and has four critical phases. Unless the volumes on the minor approach at the adjacent intersection are significantly low, the throughput of the outbound DDI signal often exceeds the capacity of the downstream standard intersection.
In some corridors across the United States, the spacing between the DDI and the downstream intersection is not great enough to store all the vehicles processed by the DDI. This is the second cause of queue spillback: lack of sufficient storage space.

FIGURE 12 is taken from the left turn lane at the outbound intersection, looking downstream from a DDI. Vehicles have spilled back from the downstream adjacent intersection past the crossover. A queue of right-turning vehicles can also be seen at the off-ramp.

FIGURE 12  Example of Queue Spillback from the Downstream Adjacent Intersection into the Outbound DDI Crossover (17).

2.3  TREATMENT SELECTION

There are numerous potential treatments which can and have been used for the right turn at off-ramps at interchanges. TABLE 1 presents an extended list of treatments as proposed in the Signalized Intersection Information Guide, the advantage of each, and why the treatment was from this research (18).
TABLE 1 Treatments for Right Turns from the Off-Ramp

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Advantage</th>
<th>Reason for Exclusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple Turn Lanes</td>
<td>Additional exclusive right turn lanes allow for a greater capacity</td>
<td>Included</td>
</tr>
<tr>
<td></td>
<td>without increasing the green time</td>
<td></td>
</tr>
<tr>
<td>Channelization</td>
<td>A raised median increases the efficiency of turning movements</td>
<td>Included</td>
</tr>
<tr>
<td>RTOR Allowed</td>
<td>Increases the capacity of the movement without increasing the green time</td>
<td>Included</td>
</tr>
<tr>
<td>Slip Lane</td>
<td>Provides direct access to the adjacent minor road</td>
<td>Included</td>
</tr>
<tr>
<td>Added Lane on Cross Street</td>
<td>Allows for free-flowing right turn onto the cross street</td>
<td>Included</td>
</tr>
<tr>
<td>Dedicated Phase</td>
<td>Provides green time exclusively for turning movements from the off-ramp</td>
<td>Not a geometric treatment</td>
</tr>
<tr>
<td>Pre-empted Phase</td>
<td>Provides green time exclusively for turning movements from the off-ramp on an “as needed” basis</td>
<td>Not a geometric treatment</td>
</tr>
<tr>
<td>Restricting U-Turns</td>
<td>Eliminates movement which conflicts with right turns</td>
<td>No u-turns present in base model volumes</td>
</tr>
<tr>
<td>Exclusive Turn Lane</td>
<td>Can increase saturation flow rate of movement</td>
<td>Base model already includes an exclusive turn lane</td>
</tr>
<tr>
<td>Reducing Movements</td>
<td>Reduces loss time through reduction in critical phases and can increase g/C for remaining phases</td>
<td>Crossover is already reduced to 2 critical phases.</td>
</tr>
<tr>
<td>Yield Control</td>
<td>Can increase capacity of movement given sufficient quantity and length of gaps in conflicting flow</td>
<td>High volume would likely require more gaps than would be available</td>
</tr>
</tbody>
</table>

Given the recent research on DDI signal timing (6-11) and the gap in research on geometric treatments, this research focused only on geometric treatments; therefore, dedicated and pre-empted phases were not tested.

The treatment of restricting conflicting u-turns was not viable given the base model did not have any vehicles which made u-turns at the crossover. Similarly, the exclusive
lane treatment was not tested because the DDI design includes one or more exclusive right turn lanes from the off-ramp.

As a DDI crossover only has two critical phases, no further reduction of movements would result in reducing the number of critical phases to one. To do so would be to alter the interchange to a design other than a diverging diamond.

Yield control treatment was not considered because of the high volume of right turning vehicles (1,000 vph) combined with the conflicting volume (371.7 vphpLn). There would be 370 gaps in an hour with an average headway of 9.7 seconds. Assuming a constant queue of right turning vehicles, a critical headway of 6.2 seconds and a follow up headway of 3.3 seconds (19), an average of one vehicle could merge onto the corridor with each gap giving the movement a volume to capacity ratio of 2.7.

Channelization and the added lane treatment were combined into one strategy. Because the added lane extended through the downstream intersection, it was renamed as an auxiliary lane. This resulted in five remaining geometric treatments to be tested in attempt to reduce the operational challenge of queue spillback at the off-ramp: (1) channelized turn lane (CTL) with auxiliary lane, (2) dual right turn lanes, (3) right turn on red (RTOR) allowed, (4) dual right turn lanes with RTOR allowed, and (5) slip lane.
3 LITERATURE REVIEW
Due to the relative newness of DDIs, the treatments addressed in this report have not yet been studied with specific respect to the impacts at DDIs. Therefore, the literature review focuses on existing research of: (1) DDI simulation, (2) field evaluation of DDIs in operation, and (3) the treatments to be tested in this thesis as applied at diamond interchanges and standard intersections.

3.1 SIMULATION OF DDIS
Chlewicki (12) compared the operations of a DDI and a standard diamond interchange with fixed time signals. Synchro 5, a microsimulation tool, was used to study phasing and geometric differences while microscopic simulation was run using SimTraffic 5. Optimized, fixed time signal phasing was generated by Synchro 5 and used in the simulation. Only one volume combination was used with roughly half of the arterial traffic in both directions consisting of through traffic and the remaining traffic split evenly between left and right turns. The DDI was more efficient in every movement with the exception of the though movement at the outbound signal. Despite this exception, the DDI still had better overall efficiency with roughly one fourth the stop delay, half as many total stops, and one third the total delay as the standard diamond. Chlewicki also found that the overall performance measures were similar when the left turn at the on-ramp was signalized versus when it was unsignalized. The scope of the simulation did not extend to adjacent intersections and the queue lengths were not measured.

Bared et al. (20) conducted a more extensive analysis using VISSIM to compare the capacity of a conventional diamond interchange to that of four and six lane DDIs using five and six traffic flow scenarios, respectively. The total number of vehicles per hour increased over the various flows, however the relative number of vehicles per movement remained fairly consistent. Arterial traffic counted for roughly 60% of the total traffic. The four-lane DDI was directly compared to a standard diamond interchange of unspecified size regarding the measures of total delay per vehicle, stops per vehicle, stop time per vehicle, and maximum queue length. The six-lane DDI was not compared to the standard diamond but was tested for capacity measurements. Left and right turn
movements from the off-ramp were signalized with both the turning movement and concurrent crossover movement receiving the same green time (i.e. no overlap phasing was used). The cycle lengths and signal timings for the DDIs were developed through several trials in VISSIM, while PASSER-3 was used to design and optimize the signal settings for the standard diamond.

The four-lane DDI performed equally as well as the conventional diamond during low and moderate flows. However, in high traffic volumes, the four-lane DDI had an approximate 40% reduction in average total delay per vehicle, stops per vehicle, and stop time per vehicle, and a 28% reduction in maximum queue length. The capacity per lane for all movements was higher for both the four- and six-lane DDIs as compared to the standard diamond. The six-lane DDI was able to double the capacity per lane of left turns from the off-ramp over the standard diamond.

Siromaskul and Speth (21) used VISSIM to compare SPUIs, tight and wide DDIs, tight and wide conventional diamonds, and two types of partial cloverleaves. Tight interchanges had a signal-to-signal spacing of 500 feet, while spacing at wide interchanges was 1,000 feet. The first partial cloverleaf had all ramps on the north side of the interchange, while the second type had ramps on the northwest and south east quadrants. Each of the seven interchanges was designed with the minimum number of lanes and turn bays necessary to meet a level of service “D” designation.

Four volume subsets were considered, which held constant the number of vehicles entering and exiting the interchange area but varied the heavy movement: heavy left turn onto the freeway eastbound to north bound, heavy left turn onto the freeway westbound to southbound, heavy through movement along the arterial, and heavy exchange movement to and from the freeway. In addition to the four movement scenarios, an adjacent intersection was added to both sides of the interchange to feed traffic into the system. Three scenarios were used: the arterial – heavy turning movement feeding into and out of the interchange area, the collector – vehicles mainly reaching and leaving the interchange by traveling straight through the adjacent intersection, and isolated – no adjacent intersections existed. Each of the seven interchange types were tested under the
twelve scenarios using VISSIM’s weighted average delay per vehicle as the key measure of effectiveness. The DDI outperformed the other interchange types at handling heavy turning movements using the fewest number of lanes, and its effectiveness increased when ramp traffic increased but arterial through traffic decreased. The presence of the adjacent intersections increased the average vehicle delay by almost 45% when considering all of the volume scenarios.

Additional analysis was conducted by Hughes et al. in the Alternative Intersections/Interchanges Informational Report (AIIR) (13). Using VISSIM, three different DDI geometries and two different traditional diamond geometries were tested with volumes which increased proportionally. It is unclear what efforts were made toward calibration and validation of the models. A two-lane DDI was tested under two different volume scenarios. In the first, there were 600 left turns from the major road and 300 through movements whereas in the second scenario, these volumes were switched. All other volumes in the two scenarios were identical. The second scenario resulted in a 16% reduction in average vehicle delay through the system. The four other geometries were tested with proportionally increasing volumes.

Overall, the DDIs were able to carry 6,000 veh/h on a six-lane bridge and 3,700 veh/h on a four lane bridge where traditional diamond interchanges required eight and six lanes, respectively. AIIR also suggests the DDI may be the superior option under the following conditions:

- Heavy on-ramp left turns and moderate through volumes
- Heavy, unbalanced through volumes
- On-ramp left-turn demand greater than 300 veh/h/lane
- Off-ramp left turn demand less than 700 veh/h/lane
- Mainline demand in both directions less than 650 veh/h/lane
- An existing bridge with limited width where expansion is not physically possible or is of high cost

Overall, prior research on the simulation of DDIs is lacking in volume and was almost exclusively conducted prior to the installation of the first DDI in the United
States. Chlewicki, Barad et al., and Siromaskul and Speth all published before the first DDI was installed in the United States, so calibration and validation of the simulations were not possible. Schroeder et al. concluded that simulation of the DDI is possible, but careful calibration of the speed and routing decisions are required to accurately model the interchange (14).

Additionally, as is explained below, there was no expectation that the right turn from the off-ramp would cause as many challenges as it has, so there is little discussion of that movement in the papers referenced above. Finally, most of the research was aimed at comparing the DDI to other interchange forms with little emphasis placed on how the DDI interacts with adjacent intersections. Siromaskul and Speth was the only study to consider adjacent intersections.

3.2 FIELD EVALUATION FINDINGS

After the first DDI was built at MO-13 and I-44 in Springfield, Missouri, the Missouri Department of Transportation (MoDOT) (16) provided a post construction evaluation of operations, safety, and public perception. The evaluation reported a slower travel time during low volume periods, presumably due to the traffic calming effect of the DDI. MoDOT also found large improvements in queues and delays for left turning movements to and from the interstate during peak hours. Overall crash rates, which were adjusted for traffic volume, were reduced, including rear-end, left turn, and left turn right angle crashes. The evaluation showed no indication that the DDI created any new types of crashes which were not present at standard signalized intersections. It was noted that the right turn from the off-ramp was originally installed as a yield control movement. Shortly after opening, a traffic signal was installed due to safety concerns, as addressed in Section 2.2 above. Despite public confusion before the installation of DDIs, a survey conducted by MoDOT reported 80% of respondents felt improvements in traffic flow and delay. Also, 87% felt crashes were less likely to occur at a DDI as opposed to a standard diamond intersection, and 91% reported they had a good understanding of how to drive the interchange with the assistance of islands, signing, signals, and pavement markings.
Upon opening four DDIs, the Utah Department of Transportation (UDOT) (22) conducted a similar observational study. There was no mention of challenges with right turn from the off-ramp. Key observations included:

- The DDI works well with high demand from the interstate to the cross street or high through traffic over the interchange accompanied by low exiting volumes.
- Development of left turn auxiliary lanes, either before the inbound DDI signal (for heavy left turn movement) or between the signals (for medium movement), helps to separate left turning traffic from through traffic and improve flow.
- Heavy through traffic demand, coupled with high volumes at the off-ramps, results in congestion and the need for vehicle storage between the intersections. Signal timing can be optimized to prioritize either through traffic or left turn movements from the off-ramp, but it is difficult when heavy demand exists for both movements.
- Coordination of DDIs in a corridor can be challenging as the two phase signals at DDIs prefer a lower cycle length compared to others signals along the corridor.

Vaughan et al. (23) reported interim findings from a multi-year study of seven DDIs located across the United States. Initial findings from data indicated right turn movements from the off-ramp should be signalized or moved as far away from the crossover point as possible to avoid driver confusion which corroborates the Missouri findings. Additionally, pedestrian access, particularly for those with vision impairment, must be considered in the design because of the unique traffic flow. In a corridor context, engineers should consider access management and coordination of DDI signals with adjacent intersections. It was noted that calibration of DDIs in simulation would be of particular importance especially with lane utilization rates. The observance of the DDI operations in the field led to the conclusion that “out of the box” simulation and traffic analysis values may not accurately reflect real-life conditions. It is also noted that careful consideration must be given to adjacent intersections, as the benefits of the DDI can be
eroded by queue spillback from poorly operating adjacent intersections. Unfortunately, the full report from FHWA project DTFH61-10-C-00029 is not yet complete.

3.3 REVIEW OF TREATMENTS
Qi and Li (24) synthesized several studies examining the impact of RTOR on safety. They estimated RTOR is permitted at more than 80% of intersections in the United States. Several prior studies developed guidelines, which fell into one of two categories: (1) criteria under which RTOR shall be prohibited, and (2) criteria under which RTOR may be prohibited. Guidelines of importance to this research include:

- Inadequate sight distance is provided (shall)
- Dual right turn lanes are provided – prohibit RTOR from inside lane due to sight distance blocked by outside vehicle (may)
- RTOR crash history of more than one crash per year (may)
- Inadequate capacity of the receiving lane (may)

A nation-wide survey of traffic engineers confirmed that these criteria were consistent with current engineering practice.

Chandler et al. (18) suggested dual right turn lanes should be provided when the demand for the movement is not able to be met by a single turn lane without prohibitively long green times. They also noted that providing an acceleration lane may be appropriate. Cooner et al. (25) conducted a study of double right turn lanes in Texas. A national survey concluded there was a significant lack of formal guidance on the use of dual right turn lanes. Additionally, a study of five sites with dual right turn lanes found peak hour volumes as high as 1,000 vehicles per hour (vph) (at standard “T” intersections). A safety study of 25 sites concluded a well-designed dual right turn lane resulted in no more frequent nor severe crashes than a single right turn lane.

In an effort to determine the approach capacity of auxiliary lanes compared to continuous through lanes, Bugg et al. (26) found usage of auxiliary was most influenced by through-movement congestion along the approach. In other words, drivers took full advantage of the auxiliary lane if it meant avoiding cycle failure. They suggested the
upstream length of auxiliary lanes should, therefore, be long enough to allow access from the continuous through lane under congested conditions.

3.4 SUMMARY

Overall, prior research on the simulation of DDIs is lacking in volume and was almost exclusively conducted prior to the installation of the first DDI in the United States meaning there was no calibration of the models. The right turn queue spillback at the off-ramp was not identified as an operational challenge until after the first DDI was implemented, so little discussion of that movement exists in the early simulation research. Only one simulation model considered the impact of adjacent intersections, so queue spillback from the downstream adjacent intersection would not have appeared in the other models.

It can be concluded, then, that a calibrated simulation of a DDI with adjacent intersections should be constructed to replicate operational challenges as observed in the field. Furthermore, the treatments to be studied are viable and applicable options for treatment of queues at the off-ramp.
4 METHODOLOGY
This is a microsimulation study of five treatments to determine if, and to what extent, each is capable of reducing the operational challenge of queue spillback common to DDIs around the country. Average maximum queue spillback length, average delay, and average number of stops are used to measure the reductions, and ANOVA is used to determine statistical significance. The control model is based on the DDI at National Avenue and US 60 in Springfield, Missouri. VISSIM and VISTRO are used for signal timing optimization and microsimulation of the models.

4.1 EXPERIMENTATION PROCESS
The first step of the experiment was to build a model which had no operational challenges. This was accomplished by developing a geometric layout and volume combination, as detailed in Section 4.1, which functioned without operational challenges. This model is the base model.

The volume of the WB to NB right turn movement from the off-ramp was increased until the desired operational challenges were observed. This volume is known as the control volume. The model which is identical to the base model but has the control volume at the WB to NB right turn movement from the off-ramp is known as the control model. The measures of effectiveness (MOEs) of the control model are the control MOEs. From the control model, a treatment was applied and once again the volume of the WB to NB right turn movement from the off-ramp (movement of interest) was increased until the average maximum queue lengths at the right turn of the off-ramp and through movement of the downstream adjacent intersection, and delay and number of stops of the WB to NB right turn route from the off-ramp and NBT route through the corridor – hereafter referred to as the MOEs of interest – matched the control MOEs of interest. This was repeated for each treatment. FIGURE 13 details this process in a flow chart.
FIGURE 13 Flow chart of experimentation process.
The volume of the WB to NB right turn movement at the off-ramp was increased in 100 vph increments until the intended challenges appeared as dictated by the MOEs of interest. Volumes from 400 to 1100 vph were tested using the simulation procedure described in Section 4.4 below. The MOEs for each volume were charted and compared. The ideal volume had MOEs of interest with significant deviations in comparison to the undersaturated base model. The deviations needed to be large enough that the impact created by the treatment would be distinct and measurable, but not so significant as to render the model inoperable.

Vehicle counters were placed at every signalized movement in each model to verify the desired flowrate was similar to the actual flow rate (although flowrate is a stochastic, not deterministic, variable in VISSIM, so it was rare for the flowrates to be exactly equal). If the volume become so high that the queue lengths reach the edge of the model, VISSIM would be unable to add additional vehicles until the queues receded. If this happened often enough, the desired flowrate would not be achieved. This would be detected by the vehicle counters. This reduction in flow rate was seen as the desired flowrate for the WB to NB right turn movement from the off-ramp reached 1100 vph.

FIGURE 14, FIGURE 15, and FIGURE 16 show the three MOEs as they vary with the volume increases.
FIGURE 14 Average maximum queue length for base model under an increasing volume of interest.
FIGURE 15 Average delay for base model under an increasing volume of interest.
FIGURE 16 Average number of stops for base model under an increasing volume of interest.
A volume of 1,000 vph was selected for the westbound right turn from the off-ramp. FIGURE 14 shows the average maximum queue length at 1,100 vph extended past the available storage capacity of the off-ramp. As can be seen in FIGURE 15 and FIGURE 16, the volume of 1,000 vph is an inflection point along the graph signaling a larger volume would be problematic. This was consistent with reduced flowrates found at 1,100 vph. Therefore, the WB to NB right turn movement from the off-ramp was selected as the control volume.

Once the appropriate control volume was selected, each treatment was applied to the control model. The MOEs for each of the treatments versus the control were charted and can be found in Section 5.1 below.

Finally, for each treatment model, the volume of interest was again increased in 100 vph increments until the MOEs exceeded those of the control model. Those results can be found in Section 5.2 below.

Overall, 36 models were created. Each model collected data for 3,600 seconds under each replication and was replicated 15 times.

4.2 MODEL DEVELOPMENT

Models were developed in VISTRO for signal timing optimization and VISSIM for microsimulation. The geometries and volume levels of the base model are described below.

4.2.1 Geometry

The base model was a north/south oriented corridor composed of a pair of DDI crossovers with two standard four critical phase signal intersections: one located 1,000 feet north of the north crossover and the second located 1,000 feet south of the south crossover. No intermediary access points were present.

The DDI at National Avenue and US-60 in Springfield, Missouri was used as the base geometry for the model with some modifications to ensure the system was symmetric. This site was used as the base model because a calibrated and validated model was available. Additionally, the existing conditions included one three-lane-by-three-lane
crossover and one three-lane-by-two-lane crossover, so few modifications were needed to update the model to three full lanes throughout the DDI. This three-by-three configuration allowed for heavier traffic volumes and more pronounced operational challenges than a smaller DDI would have.

The base model had three travel lanes in both the north- and southbound directions. Left turns onto the freeway were accessible by use of a single shared through-and-left turn lane. Right turning movements onto the freeway had a single exclusive right turn lane. In the base model, all movements from the freeway used single exclusive turn lanes, one each for right-turn and left-turn maneuvers. The off-ramp was a single lane from the freeway, expanding to two lanes 850 feet upstream of the stop bar. The total length from the freeway to the right turn stop bar was 1,340 feet.

At the adjacent intersections, all left turning movements were coded with exclusive dual turn lanes, while right turning movements were exclusive single lanes. On the mainline, through movements had two exclusive lanes, while one exclusive lane was present on the minor street. The three travel lanes downstream of the DDI transitioned into the two through lanes and a left turn lane. The right turn lane and second left turn lane pockets developed upstream of the stop bar at the downstream adjacent intersection. All right turn lane pockets as well as north- and southbound left turn lane pockets developed 250 feet upstream of the stop bar. East- and westbound left turn lane pockets developed 400 feet upstream. FIGURE 17 shows (a) one of the adjacent intersections, (b) the DDI, and (c) the corridor. All images on the left are shown in VISSIM’s “link-connector” view with links shown in blue and connectors drawn in pink.
FIGURE 17 VISSIM Geometric Model of the DDI Corridor with a) the adjacent intersection (b) the DDI and (c) the entire corridor.
4.2.2 \textit{Volumes}

In designing the volumes, consideration was given to conditions commonly seen at DDIs throughout the United States. At suburban interchanges where DDIs are often used, large retail stores, a combination of fuel and food outlets, or housing developments can be located within the quadrants of the interchange. Heavy turning movements from the minor street are present as the majority of traffic coming from these land uses desires to access the corridor (as opposed to desiring to cross the corridor).

The base model volume scenario was designed to function below capacity. The Capacity Analysis for Planning of Junctions (CAP-X) tool was used to provide a starting point for volume combinations. CAP-X uses critical lane analysis to assist at the planning level in determining the feasibility of eight different intersections, five interchanges, three types of roundabouts and two types of mini-roundabouts (27). The software sums the lane volumes per hour at all merge and crossing points. This sum is known as the critical lane volume sum.

The critical lane volume was assumed to be 1,600 vph with a left turn adjustment factor of 0.95, a right turn adjustment factor of 0.85, and a truck to passenger car equivalence factor of 2. A volume combination was then developed which resulted in a DDI with volume to capacity (v/c) ratios between 0.35 and 0.41 for all six merge or crossing points (WBR with NBT, NBT with SBT at the northern crossover - NX, WBL with SBT, EBL with NBT, NBT with SBT at the southern crossover – SX, and EBR with SBT) as shown in TABLE 2. This allowed for the base model to operate without any operational challenges.
TABLE 2  Volume-to-Capacity Ratio at Crossing and Merge Points of the DDI under the Base Volume Scenario as Estimated by CAP-X

<table>
<thead>
<tr>
<th></th>
<th>CAP-X estimated v/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>WBR/NBT</td>
<td>0.35</td>
</tr>
<tr>
<td>NBT/SBT at NX</td>
<td>0.41</td>
</tr>
<tr>
<td>WBL/NBT</td>
<td>0.41</td>
</tr>
<tr>
<td>EBL/SBT</td>
<td>0.35</td>
</tr>
<tr>
<td>NBT/SBT at SX</td>
<td>0.41</td>
</tr>
<tr>
<td>EBR/SBT</td>
<td>0.41</td>
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</tbody>
</table>

The capacities at these points estimated by CAP-X were verified against capacities at movements using the calibrated VISSIM model. Although they are not directly comparable, it can be expected that if the v/c of the points are near 0.4, the v/c of the movements should be similar, provided sufficient green time exists. To determine the v/c of the movements, saturation flow rates were measured for each movement at the DDI and were combined with the number of lanes, effective green time, and recorded volumes for one simulation with a cycle length of 120 seconds. These values are summarized in TABLE 3. The v/c for all eight movements are between 0.28 and 0.44 thereby validating the assumptions made in the CAP-X modeling.
### TABLE 3 Volume to Capacity Ratios for Movements in the DDI as Measured in VISSIM

<table>
<thead>
<tr>
<th></th>
<th>Saturation Flow s (vph)</th>
<th># of Lanes</th>
<th>Effective Green Time g (s)</th>
<th>Capacity c (vph)</th>
<th>Volume v (vph)</th>
<th>Volume to Capacity v/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>WBR</td>
<td>1,924</td>
<td>1</td>
<td>46</td>
<td>738</td>
<td>274</td>
<td>0.37</td>
</tr>
<tr>
<td>NBT at NX</td>
<td>1,931</td>
<td>3</td>
<td>61</td>
<td>2,945</td>
<td>1,060</td>
<td>0.36</td>
</tr>
<tr>
<td>SBT at NX</td>
<td>1,931</td>
<td>3</td>
<td>45</td>
<td>2,172</td>
<td>955</td>
<td>0.44</td>
</tr>
<tr>
<td>WBL</td>
<td>1,924</td>
<td>1</td>
<td>60</td>
<td>962</td>
<td>271</td>
<td>0.28</td>
</tr>
<tr>
<td>EBR</td>
<td>1,924</td>
<td>1</td>
<td>61</td>
<td>978</td>
<td>273</td>
<td>0.28</td>
</tr>
<tr>
<td>NBT at SX</td>
<td>1,931</td>
<td>3</td>
<td>62</td>
<td>2,993</td>
<td>1,043</td>
<td>0.35</td>
</tr>
<tr>
<td>SBT at SX</td>
<td>1,931</td>
<td>3</td>
<td>46</td>
<td>2,221</td>
<td>951</td>
<td>0.43</td>
</tr>
<tr>
<td>EBL</td>
<td>1,924</td>
<td>1</td>
<td>45</td>
<td>272</td>
<td>277</td>
<td>0.38</td>
</tr>
</tbody>
</table>

At the DDI, the left and right turns from the freeway, as well as onto the freeway had 275 vph. There were 840 vph that drove north through the entire DDI and 560 vph that drove south, representing a 60/40 directional split. The base volumes through the DDIs are shown in FIGURE 18.
To generate the inbound flow upstream of the DDI, it was assumed that the right turn from the minor approach contributed 20% of the traffic and the left turn from the minor approach contributed 20%. The remaining 60% came from the through movement along the major approach upstream of the upstream adjacent intersection. FIGURE 19 shows the origination of vehicles from the northern adjacent intersection heading south into the DDI. These source volume percentages were the same at the southern adjacent intersection heading north into the DDI.
FIGURE 19  Origination of Inbound flow to DDI as shown at the adjacent southern intersection.

For vehicle volumes coming out of the DDI, it was assumed that 15% of the major outbound approach turned left, 15% turned right, and the remaining continued straight through the intersection. At the minor approaches, there was a 48/20/32 split for the north, though, and south movements which was in line with the 60/40 directional split along the mainline. FIGURE 20 summarizes the volume splits by approach for the northern intersection. The volumes splits are identical at the southern adjacent intersection.
The inbound volumes were calculated first according to FIGURE 19. Once the inbound volumes (shown above as the inbound dark movements) were been generated, the green volumes could be set according to FIGURE 20. The three outbound volumes were independent of all other approaches and could be set using the splits in FIGURE 20 (shown above as the outbound dark movement).

4.2.3 Signal Timing

The four intersection corridor was run in coordination on a constant 120 second cycle. It was fixed to reduce variability between the trials. The cycle length was driven by the less efficient adjacent intersection which was not impacted as drastically as the northern DDI crossover. By fixing the cycle length, a more direct comparison of the impact of the treatments at the crossover could be made. The splits, offsets, and left turn leading or lagging were optimized in VISTRO as explained in Section 4.4.1 below. Given that the volume at the crossover was changing, the splits had to be variable to ensure enough green time was provided. The variability in the splits necessitated adjustments to the offsets which, in turn, could be better optimized if leading and lagging sequencing were allowed to vary.
Given the arterial nature of the corridor, characterized by the heavy through movements, the adjacent intersections were semi-actuated with the major through movements coordinated. Minor street through movements and major street left turns had a minimum green time of 15 seconds, while minor street left turns had a minimum green time of 7 seconds. These were all set to meet driver expectancy given the approach (minor or major) and movement (left turn or through) as dictated by the Signal Timing Manual (28).

Non-coordinated movements had one 60 foot detector in each lane located at the stop bar. Under standard semi-actuation, no detector would be present for the coordinated movements, so the coordinated movements should always be served. Therefore, the coordinated movements were set to minimum recall. This aligns with VISSIM’s ring barrier controller design in which the activation of minimum recall for the coordinated movements is necessary to ensure service if no detector is present.

The vehicle extension was set to 2 seconds. Given the speeds along the corridor, two seconds was the suggested extension time from the Signal Timing Manual rounded up to the nearest second. Right turns on red were not allowed, but the right turn was run with the concurrent through movement of the same approach and with the non-conflicting left turn from the adjacent approach. Prohibiting right turn on red ensured the simulation results would be conservative estimates useful in those areas where right turn on red is not permitted. Dual entry was allowed for the minor through movements. This is the most common use of dual entry (18).

The DDI crossovers were pretimed and run on two controllers. FIGURE 21 shows the ring-barrier controller for the northern crossover. The southern crossover is identical. Although some DDIs use overlaps to reduce the loss time, no such overlap phasing for the crossovers was used here. At the time of simulation, VISTRO was not capable of coding overlaps for non-right turn movements. Since these overlaps could therefore not be included in the optimization process, none were used in VISSIM either. This allowed for a pure implementation of the optimized signal timing plan and ensured treatments would be equally impacted.
FIGURE 21  Phasing scheme for northern crossover.

4.3  MEASURES OF EFFECTIVENESS

Three measures of effectiveness (MOE) were selected to monitor the impact of various operational treatments on the challenges described above: (1) average maximum queue lengths, (2) average total delay, and (3) average number of stops. Delay and queue length are both suggested by the Signalized Intersection Information Guide as generally acceptable MOEs (18). The number of stops was also considered as a means of identifying cycle failure. If queue spillback is present and the average number of stops for a route that includes that segment is particularly high, it may indicate a cycle failure is causing the queue spillback. Conversely, if queue spillback is present but there are a reasonable number of stops, it is likely that the storage capacity is not sufficient for the expected volume, but the queue is cleared every cycle.
4.3.1 Maximum Queue Length

The maximum queue length was recorded in the simulation in each cycle. This MOE was chosen as it could directly measure the operational challenge of interest, queue spillback. The queues were recorded at the north and south bound through and left movements for the adjacent intersections, as well as at all signalized movements at the DDI as shown in FIGURE 22.

![Image](https://via.placeholder.com/150)

**FIGURE 22** Locations of queue length collection points along the corridor.

VISSIM has queue counting tools built in which measure the queue from the location of the counter upstream until the location of the next counter. A vehicle is considered in the queue once the speed is 3.1 mph or less, and that vehicle leaves the queued state once the speed reaches 6.2 mph. Additionally, the headway between two vehicles in the queue cannot exceed 65.6 feet.

4.3.2 Average Delay per Vehicle

The Highway Capacity Manual 2010 (19) uses delay to directly determine the level of service of an intersection. Delay over the OD path, as opposed to the intersection delay calculated in the HCM, gives a more holistic understanding of the corridor. A treatment
may reduce delay at one intersection but increase delay at another intersection. Measuring delay over the OD path provides a united view of the system.

Total delay was measured using VISSIM’s vehicle travel time tool. This tool measures the total time elapsed from when a vehicle passes over the start of the travel time section to when it passes over the corresponding end point. In this way, only those vehicles along a specific origin-destination (OD) path, that completed the entire path during the simulation, are considered in the computation. At each computational step, VISSIM compares the vehicle’s actual travel speed to the desired travel speed (as defined by the user). The difference between those speeds are summed over the length of the OD path to provide the total path delay. The delay of all vehicles that complete the entire OD path during simulation is averaged to find the average delay.

Travel time measurements were taken for 10 paths: (1) from south of the southern adjacent intersection to north of the northern adjacent intersection, (2) from south of the southern adjacent intersection proceeding east onto the freeway, (3) from south of the southern adjacent intersection proceeding west on the freeway, (4) from east of the interchange on the freeway to north of the northern adjacent intersection, (5) from west of the interchange on the freeway to north of the northern adjacent intersection, (6) from north of the northern adjacent intersection to south of the southern adjacent intersection, (7) from north of the northern adjacent intersection proceeding east on the freeway, (8) from north of the northern adjacent intersection proceeding west on the freeway, (9) from east of the interchange on the freeway to south of the southern adjacent intersection, and (10) from west of the interchange on the freeway to north of the northern adjacent intersection. These paths as well as their short name codes are shown in FIGURE 23. From here on, the westbound to northbound will be referred to as the right off-ramp movement.
FIGURE 23 OD paths for generation of stops and delays.
4.3.3 Average Number of Stops

The same ten OD paths were used in calculating the number of stops vehicles made over a path. VISSIM’s vehicle travel time tool also records the number of stops a vehicle makes between the route’s start point and the end point. If a vehicle stops in a queue and then proceeds to move within the queue, not exceeding the end queue condition of 6.2 mph or a 65.6 foot headway, the subsequent stop would not be considered in the total stop calculation.

The average number of stops was selected as an MOE because queue spillback and stops are closely related. If a vehicle approaches a queue, it necessarily must stop. If that queue spillback is caused by an insufficient storage space, it would be expected that the vehicle would only stop in the queue one time and then would proceed through the intersection during the next service of the phase. Conversely, if the queue spillback is caused by cycle failure, the vehicle may initially stop for the queue, begin to move as the phase is served, and then have to stop again if the phase is terminated before the vehicle reaches the intersection. Analysis of the number of stops helps identify the cause of queue spillback.

4.4 SIMULATION PROCEDURE

For each unique volume and treatment (or control) combination, the simulation procedure was followed including: developing and optimizing the signal timing in VISTRO, inputting the updated parameters in VISSIM, and running the proper number of trials in VISSIM.

4.4.1 VISSIM

HCM 2010 does not detail a method for analyzing a DDI. Additionally, field testing of geometric treatments is cost and time prohibitive. Conducting a microsimulation of the treatments provides a stochastic, low cost procedure for testing the various geometric treatments. This is preferable to any analytical model which may not account for variations in volume or arrival pattern. VISSIM 7 was selected over other tools because it uses a link-and-node based system. This allows for easy development of the DDI’s
unique crossovers. The base model was built in the VISSIM model according to the geometries and volumes as defined in Section 4.1 above. The model was calibrated and validated by Dr. Bastian Schroeder during prior research which included on-site data collection (14).

Key calibration factors included: (1) OD volumes at the DDI and adjacent intersections, (2) look-back distances from diverge points to control lane positioning, (3) field-measured free-flow speeds through the DDI including geometrically-constrained free-flow speeds at the crossover, and (4) field-implemented signal timing schemes obtained from the field controller settings.

Turn movement volumes were collected in the field at the DDI as well as the adjacent intersections. To generate OD volumes, proportional allocation of the volumes from an upstream origin to all downstream destinations were assumed.

The look-back distance is the distance upstream of the diverge point at which a simulated vehicle initiates necessary lane changes. This variable is of particular importance to DDIs. Research shows vehicles preposition well ahead of the inbound crossover, particularly those vehicles turning left from the arterial onto the freeway (29).

Reduced speed zones were used both in the horizontal curves leading into and out of the crossover as well as on the tangential path connecting the two crossovers. All speed distributions were modeled as normal distributions.

The signal timing scheme, as obtained from the field controller settings, was faithfully replicated within VISSIM’s Ring-Barrier Controller.

In validating the model, three parameters were evaluated: (1) interchange travel times, (2) route travel times, and (3) comparison of average and 95th percentile queue lengths, estimated from maximum queue lengths on a per-cycle basis.

The interchange travel time was defined as the travel time through (1) the two DDI signals, or (2) the left turning route onto the freeway from the inbound crossover to the top of the entrance ramp, or (3) the left turning route off of the freeway from the exit ramp through the outbound crossover.
The route travel times included the travel time through the DDI and adjacent signal(s) for (1) all movements through the entire arterial, (2) left turning movements onto the freeway, starting at the upstream adjacent intersection and terminating at the top of the entrance ramp, or (3) left turning movements from the freeway onto the arterial, starting at the top of the exit ramp and terminating after the downstream adjacent intersection.

The VISSIM model used in this research was acquired after calibration and validation had been completed and was adjusted to reflect the geometry, volumes, and signal timing detailed in 4.1 above. Input volumes and routes reflected the volume specific to the volume/treatment combination being tested and the volume splits described in 4.2.2 above. Because of the stochastic nature of microsimulation, the input volumes and routes were normally distributed variables, not constant. Queue counters were placed immediately upstream of some stop bars as detailed in 4.3.1 above. Travel time routes, as described in 4.3.2 and 4.3.3 originated immediately downstream of the vehicle input point and terminated immediately downstream of the final stop bar encountered during the route.

Signal timing plans were taken from VISTRO and faithfully implemented using VISSIM’s ring barrier controller.

VISSIM’s simulation controller was used to run multiple trials of the same volume/treatment scenario. As explained in Section 4.6, each scenario had 15 trials with each run using a unique random seed. Each trial lasted 4,500 seconds, with the first 900 seconds being a “warm-up” period meant to populate the model. Data was collected over the remaining 3,600 seconds. The simulation resolution was set at 10 time steps per simulation second.

4.4.2 VISTRO

VISTRO was selected as the analytical optimization tool because it is developed by the same company as VISSIM meaning the ring-barrier design and offset references are consistent. By using VISTRO, the optimized signal timing pattern could be faithfully implemented in the microsimulation with no manipulation.
The base model was built in the VISTRO model according to the geometries and volumes as defined in Section 4.1 above. Each subsequent model was adjusted for the proper volume and/or treatment depending on the trial being conducted. A network optimization was then performed using VISTRO’s genetic algorithm. The optimization function was exclusively reduction of delay. Each optimization consisted of a maximum of 100 iterations, but the optimization process would terminate if any 50 consecutive generations were modeled with less than a 1% improvement. The offset and split were optimized to the nearest one second and lead/lag optimization was allowed at the adjacent intersections.

4.5 TREATMENTS

An extensive list of treatments was reduced to five treatments for testing as explained in Section 2.3. Each treatment was applied only to the westbound off-ramp. Because the corridor is symmetric, it would be redundant to apply the treatments to both off-ramps and increase the volumes of both the eastbound and westbound right turns. Therefore, the treatments and volume increases only occurred at the westbound off-ramp.

4.5.1 CTL with Auxiliary Lane

Under the CTL with auxiliary lane treatment, the single right turn lane from the off-ramp is continued onto the arterial resulting in a free flowing turn from the off-ramp onto the arterial. The auxiliary lane is continued through the downstream adjacent intersection as is shown in FIGURE 24.
FIGURE 24 Channelized turn lane with additional lane.

4.5.2 Dual Right Turn Lanes

Under the dual right turn lane treatment, two exclusive right turn lanes are provided on the off-ramp instead of the original single lane as exemplified in FIGURE 25. The single exit lane split 850 feet upstream of the stop bar with one lane becoming an exclusive left turn lane and the other an exclusive right turn lane. The second right turn lane appeared 650 feet upstream of the stop bar.

FIGURE 25 Dual right turn lanes at National Avenue and US-60 in Springfield, MO Source: Google Maps.
4.5.3 **RTOR**

Under the RTOR treatment, vehicles are permitted to turn right from the off-ramp onto the arterial if the signal is red as long as the vehicle has come to a complete stop and an acceptable gap is present in the conflicting traffic. Gap acceptance is based on the user-defined variables controlling driver characteristics which were calibrated and validated. This treatment could be implemented in the field only if the safety concerns that led to RTOR being banned (see Section 2.2) were addressed.

4.5.4 **Dual Right Turn Lanes and RTOR**

Under the dual right turn lane and RTOR treatment, dual turn lanes are provided and RTOR is allowed.

4.5.5 **Slip Lane**

Under the slip lane treatment, 650 feet upstream of the off-ramp stop bar a third lane was developed on the off-ramp to the right of the two existing lanes. This dedicated right turn lane is designed for vehicles making a right turn from the off-ramp proceeding to the downstream intersection, and again making a right turn. Instead of joining the arterial, the slip lane developed on the off-ramp directly leads to the minor street downstream as shown in FIGURE 26.
FIGURE 26 Example of a slip lane at National Avenue and US-60 in Springfield, MO.

4.6 SIMULATION REPPLICATION REQUIREMENT

All comparisons of statistical significance were tested under an analysis of variance (ANOVA) with 95% confidence levels reported. This test reports if at least one treatment is significantly different from the other treatments or the control. If the ANOVA signaled a difference, the Tukey Honestly Significant Difference (HSD) test was used to determine which treatment(s) was different from the control.

To ensure the results of various treatments could be compared for statistical significance at an alpha level of 0.05 and a beta level of 0.2, the number of required trials were estimated before the experiment began. Beta, the probability of rejecting the treatment as an improvement when it is an improvement, is allowed to be larger than alpha, the probability of stating a treatment is an improvement when it is not an improvement. It is more reasonable to conclude a treatment may not work and be proven wrong then to expect it to work, spend money implementing it, and find out it does not. Furthermore, the power had to be kept rather low to avoid needing an excessive number of time-intensive simulations.
Ten simulations were run to find an expected value of standard deviation for each MOE. All are measures of means, the required sample size, n, is given by Equation 1,

\[ n = \sigma^2 \frac{(z_{\alpha} - z_{\beta})^2}{(\mu_1 - \mu_0)^2} \]  

(1)

where \( \sigma \) is the standard deviation, \( z_{\alpha} \) and \( z_{\beta} \) are the z scores for the one-sided alpha and beta values, respectively, and \( \mu_1 - \mu_0 \) is the minimum desired difference in means between two treatments which would be considered statistically significant, assuming the standard deviation estimate is consistent with the results. The other assumptions for this calculation requires samples to be normal and that each sample be random and independent of other samples. It was assumed that the samples were normally distributed. TABLE 4 shows the inputs and outputs of Equation 1.

**TABLE 4 Number of Samples Necessary by MOE**

<table>
<thead>
<tr>
<th></th>
<th>Standard Deviation</th>
<th>Difference in Means</th>
<th>n</th>
<th>Simulation Replications Required</th>
<th>Resulting Power</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Queue Length</td>
<td>50 feet</td>
<td>20 feet</td>
<td>37.51</td>
<td>2</td>
<td>&gt;0.999</td>
</tr>
<tr>
<td></td>
<td>Delay</td>
<td>3.95 seconds</td>
<td>14.98</td>
<td>15</td>
<td>0.805</td>
</tr>
<tr>
<td></td>
<td>Stops</td>
<td>0.115 stops</td>
<td>1.27</td>
<td>2</td>
<td>&gt;0.999</td>
</tr>
</tbody>
</table>

Since maximum queue length is measured in every cycle, every simulation provided 30 trials. The minimum sample size could therefore be achieved in two simulations. Delay was the controlling sample size requirement, so each treatment or control was simulated 15 times for 3,600 seconds with an additional 900 second warm-up period. The queue therefore had a sample size of 450 (since there are 30 – 120 second cycles per simulation), while stops and delay had sample sizes of 15. Given these known number of replications, the actual power of the analysis could be solved for. Those results are also included in TABLE 4. The power is significantly higher than 0.8 for the stops and queue length because the actual number of replications completed – 15 and 450, respectively – are much greater than the required number of replications – 1.27 and 37.51, respectively.
5  RESULTS
The first research question tested was if, for a given volume, the five treatments of interest reduced or increased the operational challenge of queue spillback from the downstream adjacent intersection and also from the off-ramp. The second research question sought to determine at what volume each treatment yielded queue spillback of the same magnitude as the control. Extended charts, including the delays and number of stops for every OD path measured, can be found in Appendices A and B.

5.1  COMPARISON OF TREATMENTS AT CONTROL VOLUME
Each treatment, as well as the control, was tested with 1,000 vph turning right from the westbound freeway off-ramp. It was expected that the auxiliary lane would best reduce queues at both the downstream intersection and the off-ramp because of the additional capacity it provided to the right turn from the off-ramp as well as the northbound through movement at the downstream adjacent intersection. Vehicles at the off-ramp had a free flowing movement, while downstream the capacity was increased by 50% with the addition of a third through lane. The dual turn lanes allowed for a doubling of capacity at the off-ramp but it was expected that the treatment would only serve to move vehicles from the queue at the off-ramp to the queue on the corridor. The RTOR treatment was not expected to have a significant impact because the number of vehicles which would have a gap large enough to make a RTOR would be limited. The slip lane, similar to the auxiliary lane, was expected to help reduce queues on the off-ramp and the corridor. Fifteen percent of the vehicles from the off-ramp would use the slip lane which could lead to modest queue reductions. The reductions expected on the corridor were less as the vehicles using the slip lane made up only 47% of all vehicles turning right at the downstream adjacent intersection and just 7% of all vehicles approaching the downstream adjacent intersection headed outbound.
5.1.1 *Queue Spillback*

The average maximum queue length was measured at ten locations along the corridor. FIGURE 27 is a scale drawing of the DDI corridor with the queue spillback metrics from the simulations superimposed.
FIGURE 27 Average maximum queue spillback for various treatments at 1,000 vph right turn volume from the off-ramp.
Each treatment resulted in a statistically significant reduction in queue length at the off-ramp at the northbound crossover (NX) as shown in TABLE 5. The sample size (n), mean (x), standard deviation (s) of the queue length in feet, as well as the p value (p), and the lower (LCL) and upper (UCL) 95% confidence limits of the difference between the control and the treatment are provided for both the westbound right turn from the off-ramp at the north crossover (WBR at NX) and the northbound through movement at the northern intersection (NBT at NI).

**TABLE 5  Statistical Analysis of Queue Spillback for Varying Treatments at the Control Volume**

<table>
<thead>
<tr>
<th>MOE</th>
<th>Mvmt</th>
<th>Treatment</th>
<th>n</th>
<th>x</th>
<th>s</th>
<th>Treatment minus Control</th>
<th>p</th>
<th>LCL</th>
<th>UCL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Maximum Queue Length</td>
<td>WBR at NX</td>
<td>Control</td>
<td>450</td>
<td>961.20</td>
<td>156.91</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Auxiliary</td>
<td>450</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
<td>-981.77</td>
<td>-940.63</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual</td>
<td>450</td>
<td>439.34</td>
<td>93.96</td>
<td>0.000</td>
<td>-542.43</td>
<td>-501.28</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual &amp; RTOR</td>
<td>450</td>
<td>249.22</td>
<td>66.18</td>
<td>0.000</td>
<td>-732.55</td>
<td>-691.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>RTOR</td>
<td>450</td>
<td>773.06</td>
<td>136.03</td>
<td>0.000</td>
<td>-208.71</td>
<td>-167.57</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slip</td>
<td>450</td>
<td>642.62</td>
<td>118.26</td>
<td>0.000</td>
<td>-339.15</td>
<td>-298.01</td>
<td></td>
</tr>
<tr>
<td>Average Maximum Queue Length</td>
<td>NBT at NI</td>
<td>Control</td>
<td>450</td>
<td>758.53</td>
<td>58.65</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Auxiliary</td>
<td>450</td>
<td>296.47</td>
<td>24.08</td>
<td>0.000</td>
<td>-472.74</td>
<td>-451.38</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual</td>
<td>450</td>
<td>784.29</td>
<td>78.83</td>
<td>0.000</td>
<td>15.08</td>
<td>36.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual &amp; RTOR</td>
<td>450</td>
<td>814.08</td>
<td>57.32</td>
<td>0.000</td>
<td>44.87</td>
<td>66.23</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>RTOR</td>
<td>450</td>
<td>736.53</td>
<td>48.67</td>
<td>0.000</td>
<td>-32.68</td>
<td>-11.32</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slip</td>
<td>450</td>
<td>727.68</td>
<td>55.44</td>
<td>0.000</td>
<td>-41.53</td>
<td>-20.17</td>
<td></td>
</tr>
</tbody>
</table>

While all five treatments were successful in significantly reducing the queue at the off-ramp, the dual turn lanes and the dual turn lanes with RTOR resulted in statistically significant increases in queue lengths at the downstream intersection.

It is not surprising that the queues on the southern half of the corridor remained largely unchanged as the treatments were applied to the northern half. No queue existed for the right turn from the off-ramp under the auxiliary lane scenario because it was converted to a free flowing movement. Because this movement no longer required any green time, and because it was the critical movement in the phase (with the non-critical
movement being the southbound through), more green time was given to the northern through movement and westbound left turn at the northern crossover. This, in turn, reduced the queue lengths for the auxiliary lane treatment at both of these locations. Similar patterns can be seen with the four other treatments. The reduction in queue lengths relative to other treatments is similar at all three movements. The auxiliary lane treatment reduced the queues the most at all three locations followed by dual lanes with RTOR, dual lanes, slip lane, and finally RTOR.

The effective-green-to-cycle length (g/C) ratio for each treatment at the westbound right turn from the off-ramp is shown in TABLE 6. The lower the ratio, the more green time the northbound through movement was provided at the crossover and the shorter its queues. These values match the ordering of queue reduction for that movement as shown in FIGURE 27.

<table>
<thead>
<tr>
<th>Treatment</th>
<th>g/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auxiliary Lane</td>
<td>0.00</td>
</tr>
<tr>
<td>Dual Right Turn Lanes</td>
<td>0.50</td>
</tr>
<tr>
<td>Dual Right Turn Lanes with RTOR</td>
<td>0.50</td>
</tr>
<tr>
<td>RTOR</td>
<td>0.63</td>
</tr>
<tr>
<td>Slip Lane</td>
<td>0.60</td>
</tr>
</tbody>
</table>

The strong reduction in queue lengths for the northbound through movement at the downstream adjacent intersection under the auxiliary lane treatment was also expected. As the addition of a third through lane increased the capacity of the movement by 50%, it would be expected to reduce the average maximum queue by roughly the same amount. Why then, did the treatments which were the second and third best at reducing the queue at the off-ramp result in an increased queue length at the downstream signal then? Those two treatments, providing dual right turns lanes and providing dual right turn lane with RTOR permitted, both served to increase the capacity of the off-ramp though an added
lane and increase the capacity of the northbound through movement at the northern
crossover through an increase in g/C. The treatments did nothing, however, to increase
the capacity of the northbound through movement at the downstream adjacent
intersection. Therefore, all the extra vehicles which were processed by the crossover only
served to increase the demand of the movement and worsen the volume to capacity ratio.

5.1.2 Average Delay per Vehicle

The delay results for three OD paths are provided in FIGURE 28 with the statistical
results in TABLE 7. The 95% confidence interval is included for each data point. The
first data point is the average delay per vehicle under the control model with a volume for
the right turn westbound to northbound right turn from the off-ramp of 1,000 vph. The
dotted lines extending from those points serve as reference for the following treatments.
FIGURE 28 Delay for varying treatments at 1,000 vph right turn volume from the off-ramp.
**TABLE 7  Statistical Analysis of Average Delay for Varying Treatments at the Control Volume**

<table>
<thead>
<tr>
<th>MOE</th>
<th>OD Path</th>
<th>Treatment</th>
<th>n</th>
<th>( x ) (sec/veh)</th>
<th>s</th>
<th>Treatment minus Control</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>p</td>
</tr>
<tr>
<td>Average</td>
<td>WB:NB</td>
<td>Control</td>
<td>15</td>
<td>85.96</td>
<td>14.82</td>
<td>x</td>
</tr>
<tr>
<td>Delay per</td>
<td></td>
<td>Auxiliary</td>
<td>15</td>
<td>27.00</td>
<td>1.35</td>
<td>0.0000</td>
</tr>
<tr>
<td>Vehicle</td>
<td></td>
<td>Dual</td>
<td>15</td>
<td>82.42</td>
<td>27.12</td>
<td>0.9960</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual &amp; RTOR</td>
<td>15</td>
<td>69.61</td>
<td>25.86</td>
<td>0.1988</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RTOR</td>
<td>15</td>
<td>78.39</td>
<td>16.14</td>
<td>0.8908</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slip</td>
<td>15</td>
<td>77.76</td>
<td>18.83</td>
<td>0.8533</td>
</tr>
<tr>
<td>Average</td>
<td>NB:NB</td>
<td>Control</td>
<td>15</td>
<td>123</td>
<td>15.05</td>
<td>x</td>
</tr>
<tr>
<td>Delay per</td>
<td></td>
<td>Auxiliary</td>
<td>15</td>
<td>73</td>
<td>4.34</td>
<td>0.0000</td>
</tr>
<tr>
<td>Vehicle</td>
<td></td>
<td>Dual</td>
<td>15</td>
<td>92</td>
<td>6.64</td>
<td>0.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dual &amp; RTOR</td>
<td>15</td>
<td>93</td>
<td>7.89</td>
<td>0.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RTOR</td>
<td>15</td>
<td>112</td>
<td>9.59</td>
<td>0.0223</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slip</td>
<td>15</td>
<td>112</td>
<td>9.19</td>
<td>0.0223</td>
</tr>
</tbody>
</table>

For vehicles on the NB:NB route, delays were statistically lower for both the dual turn lane strategy and the dual with RTOR strategy despite the increase in queue lengths at the downstream intersection. This is likely due to the increase green time the northbound through movement received at the northern crossover. Delays were also decreased for the NB:NB route for the slip lane and RTOR strategies. While all treatment show a decrease in the average delay for vehicles turning right from the off-ramp and traveling northbound through the corridor along the WB:NB route, the auxiliary lane treatment was the only one in which the decrease was statistically significant at the alpha level of 0.05. The decrease in delays for the NB:NB route without decreases in the WB:NB route suggest the corridor was coordinated to serve the NB:NB route while the WB:NB route was not coordinated and instead reached the downstream adjacent intersection when the northbound phase was not being served.
5.1.3 Average Number of Stops

The average number of stops are presented in FIGURE 29 with the statistical results in TABLE 8. The first data point is the average number of stops under the control model with a volume for the right turn westbound to northbound right turn from the off-ramp of 1,000 vph. The dotted lines extending from those points serve as reference for the following treatments.
FIGURE 29  Number of stops for varying treatments at 1,000 vph.
The number of stops for the NB:NB route decreased for the dual and dual with RTOR treatments but remained unchanged for the WB:NB route. This supports the evidence from the average delay MOE that the northbound through movement from the outbound crossover tended to arrive on green at the downstream intersection and avoid queues more often than vehicles turning right from the off-ramp. Northbound through vehicles at the outbound crossover under the slip and RTOR treatments received relatively less green time at the outbound crossover and therefore may have been more likely to arrive on red at that intersection as opposed to the dual and dual with RTOR treatments, resulting in no change at the average number of stops.

The auxiliary treatment was the only treatment which significantly reduced the number of stops for the WB:NB route. This is not surprising given the significant queue reduction seen at both the off-ramp and downstream intersection for this treatment.

### Summary

Five treatments were tested against the control to determine if the treatment improved any one of three different MOEs. TABLE 9 presents a summary of the results.
TABLE 9 Summary of Results of MOEs for Various Treatments under a Control Volume of 1,000 vph

<table>
<thead>
<tr>
<th>Treatment</th>
<th>MOE</th>
<th>Mvmt or OD Path</th>
<th>Improvement over Control Model?</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Auxiliary Lane</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue</td>
<td></td>
<td>WBR at NX</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at NI</td>
<td>y</td>
</tr>
<tr>
<td>Delay</td>
<td></td>
<td>WB:NB</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td>Stops</td>
<td></td>
<td>WB:NB</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td><strong>Dual Turn Lanes</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue</td>
<td></td>
<td>WBR at NX</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at NI</td>
<td>n*</td>
</tr>
<tr>
<td>Delay</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td>Stops</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td><strong>Dual Turn Lanes with RTOR</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue</td>
<td></td>
<td>WBR at NX</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at NI</td>
<td>n*</td>
</tr>
<tr>
<td>Delay</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td>Stops</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td><strong>RTOR</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue</td>
<td></td>
<td>WBR at NX</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at NI</td>
<td>n</td>
</tr>
<tr>
<td>Delay</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td>Stops</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>n</td>
</tr>
<tr>
<td><strong>Slip Lane</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue</td>
<td></td>
<td>WBR at NX</td>
<td>y</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at NI</td>
<td>n</td>
</tr>
<tr>
<td>Delay</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>y</td>
</tr>
<tr>
<td>Stops</td>
<td></td>
<td>WB:NB</td>
<td>n</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NB:NB</td>
<td>n</td>
</tr>
</tbody>
</table>

*statistically significant negative impact

Overall, queues along the southern half of the corridor remained largely unchanged which was expected given the treatments were applied to the northern half of the corridor. The reduction in queue length for the northbound movement at the outbound...
crossover was heavily dependent on the amount of green time that movement received. The auxiliary lane, dual turn lane, and dual turn lane with RTOR provided a significant increase in capacity to the off-ramp right turn thereby allowing more green time to be dedicated to the northbound movement. This was reflected in the queue lengths for that movement. This was not necessarily advantageous to the corridor as a whole, however.

The dual turn lane and dual turn lane with RTOR treatments increased capacity at both the northern through and westbound right movement at the off-ramp but did not increase the capacity of the northbound through movement at the downstream adjacent intersection. The result was increased queue lengths for the downstream intersection. Conversely, the auxiliary treatment increased capacity for all three movements simultaneously and the results was decreased queue lengths for all three movements. The RTOR and slip lane treatments were able to increase the capacity of both movements at the crossover, although to a lesser extent than the other three treatments, and in doing so neither overwhelmed the downstream adjacent intersection nor increased the queue lengths.

Decreased average delays and stops for the dual lanes and dual lanes with RTOR for the NB:NB route while the same MOEs remained steady for the WB:NB route suggest the corridor was coordinated to progress northbound corridor vehicles as opposed to off-ramp vehicles. The reduction in delays and stops for the NB:NB route did not extend to the slip lane and RTOR treatments. Given the northbound through movement at the outbound crossover received less green time under these treatments, it signals the savings in delay and stops for the dual lanes and dual lanes with RTOR is seen at the outbound crossover. The auxiliary treatment, having increased the capacity for both movements at the outbound crossover and the northbound through movement at the downstream intersection unsurprisingly reduced the average delay and number of stops for both the NB:NB route as well as the WB:NB route.

The results for the slip lane treatment were particularly impacted by the volume splits at the downstream adjacent intersection. If there was a greater demand for right turns at the downstream intersection, it would be expected that the treatment would have a greater
impact in reducing queues, delay, and stops. The volume reduction the slip lane provides to the northbound approach at the downstream adjacent intersection is 7%; not substantial enough to significantly reduce queues. The greater the demand for right turns at the downstream approach, the greater the volume reduction will be and the more queues can be expected to decrease at both the downstream approach and the off-ramp approach.

5.2 COMPARISON OF AT HIGHER VOLUMES
The second research objective was to determine how much added capacity could be provided by each treatment. Because the operational challenge considered in this research is queue spillback, the average maximum queue length is the main factor in determining the added capacity. Capacity in this sense are the additional vehicles that can be processed by the system before the average maximum queue lengths for the right turn from the off-ramp and the northbound through movement at the downstream adjacent intersection reach the length measured under the control volume with no treatment. The westbound to northbound right turn from the off-ramp was increased in 100 vph increments.

5.2.1 CTL with Auxiliary Lane
The volume under the auxiliary treatment was incremented by 100 vph up to 1,700 vph. At no point did the maximum average queue length at the off-ramp or maximum average queue length at the downstream intersection exceed those seen under the control volume, no treatment scenario as shown in FIGURE 30.
FIGURE 30  Average maximum queue length for auxiliary lane treatment at increasing volume of interest.
The queues for the northern through movement at the downstream adjacent intersection increased uniformly as would be expected with a uniform increase in volume. The queues for the northern through movement at the outbound crossover were constant because the \( g/C \) remained constant. While the volume of the right turn from the off-ramp was increasing, the movement was not signalized under the auxiliary lane treatment, so the phase splits were dictated by the volume of the westbound left from the off-ramp, the northbound through at the outbound crossover and the southbound through at the crossover. Since these volumes all remained constant, the splits remained constant and the queue for the northbound movement remained constant. There was some variation in the queue length for the westbound left turn from the off-ramp because it shared an approach with the westbound right movement.

Saturation flow rate tests conducted to verify CAP-X results as explained in Section 4.2.2 showed that the saturation flow rate for the westbound to northbound right turn from the off-ramp was 1,924 vph. Because the auxiliary lane provided a free-flowing movement, the capacity was 1,924 vph assuming no downstream blockage impeded the flow. Additionally, there should not have been a queue at the movement as long as the volume remained below the capacity of 1,924 vph. The presence of a strictly positive, monotonically increasing average maximum queue for the right turn movement at the off-ramp given a v/c less than 1.0, then, suggests there is a downstream disturbance. The average maximum queue length for the downstream adjacent intersection at 1,700 vph rarely spilled back to the outbound crossover as shown in FIGURE 31. Given the relatively low v/c of the NBT movement at 0.62 as shown in TABLE 10, the spillback was likely due to a lack of sufficient storage space as opposed to a disparity in the volume to capacity ratio. This rare spillback was enough to disrupt the right turn from the off-ramp as shown in FIGURE 32.
TABLE 10 Volume to Capacity Ratio for WBR at Northern Crossover and NBT at Northern Downstream Adjacent Intersection under the Auxiliary Lane Treatment at Varying Volumes of Interest

<table>
<thead>
<tr>
<th>Volume of Interest</th>
<th>Volume / Capacity Ratio</th>
<th>WBR at NX</th>
<th>NBT at NI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>0.52</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>1,100</td>
<td>0.57</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>1,200</td>
<td>0.62</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>1,300</td>
<td>0.67</td>
<td>0.71</td>
<td></td>
</tr>
<tr>
<td>1,400</td>
<td>0.73</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>1,500</td>
<td>0.78</td>
<td>0.77</td>
<td></td>
</tr>
<tr>
<td>1,600</td>
<td>0.83</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>1,700</td>
<td>0.88</td>
<td>0.83</td>
<td></td>
</tr>
</tbody>
</table>
FIGURE 31 Proportion of cycles in which fraction of storage is used under the auxiliary lane treatment with a volume of interest of 1,700 vph.
FIGURE 32  Trajectory plot of WB:NB route under auxiliary treatment with 1,700 vph volume of interest.
It can be seen in FIGURE 32 that from simulation second 1,021.1 to 1,069.1, the vehicles turning right from the off-ramp were not doing so at a true free-flow speed. This, then, reduced the capacity of the movement from 1,924 vph to near 1,700 vph, an increase in capacity of 700 vph over the control model. It is during the times when the moderate queue on the corridor reduced the speeds of right turning vehicles from the off-ramp that the maximum queue length on the off-ramp used 90% or more of the available storage space. The bimodal distribution of FIGURE 31 suggests the system was unstable at a volume of 1,700 vph. When vehicles were able to flow at free-flow speeds, the storage space used was under 10%, but any reduction in the speed of the turn resulted in a significantly increased, although short lived, queue length.

5.2.2 Dual Right Turn Lanes

The volume under the dual right turn lane treatment was incremented by 100 vph up to 1,400 vph. Queues for the downstream adjacent intersection reached the levels of the control model at 1,100 vph while the average maximum queue lengths seen at the right turn of the off-ramp of the control model were not met until 1,300 vph as shown in FIGURE 33.
FIGURE 33  Average maximum queue length for dual right turn lane treatment at increasing volume of interest.
Despite the modest volume to capacity ratios for the westbound right turn movement from the off-ramp as shown in TABLE 11, the average maximum queue quickly grew with each additional 100 vph. The capacity was found by multiplying the effective green time to cycle length ratio by the saturation flow rate. The increase in this queue, as well as the queue of the northern through movement at the outbound crossover can be explained by the queue spillback from the downstream adjacent intersection.

**TABLE 11 Volume to Capacity Ratio for WBR at Northern Crossover and NBT at Northern Downstream Adjacent Intersection under the Dual Right Turn Lane Treatment at Varying Volumes of Interest**

<table>
<thead>
<tr>
<th>Volume of Interest</th>
<th>Volume/Capacity WBR at NX</th>
<th>Volume/Capacity NBT at NI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>0.52</td>
<td>0.87</td>
</tr>
<tr>
<td>1,100</td>
<td>0.55</td>
<td>0.91</td>
</tr>
<tr>
<td>1,200</td>
<td>0.58</td>
<td>0.93</td>
</tr>
<tr>
<td>1,300</td>
<td>0.60</td>
<td>0.95</td>
</tr>
<tr>
<td>1,400</td>
<td>0.63</td>
<td>0.97</td>
</tr>
</tbody>
</table>

FIGURE 34 shows that the average maximum queue for the northern through movement at the downstream adjacent intersection exceeded the available storage space during more than 40% of the 450 cycle lengths. FIGURE 35 is a trajectory plot taken from one simulation between simulation seconds 1,000 and 1,270. It can be seen that vehicles which left the northern through movement at the outbound crossover at second 1,040.1 would have arrived at the downstream adjacent intersection on green if the vehicles from the westbound right turn from the off-ramp were not already queued. Northbound vehicles proceeding through the outbound crossover beginning at section 1,070.1, as well as the following vehicles from the off-ramp, experience cycle failure. The dual turn lane provided added capacity to both the westbound right turn and the northbound through at the outbound crossover, yet this capacity was largely unused because no extra capacity was available at the downstream adjacent intersection.
FIGURE 34 Proportion of cycles in which fraction of storage is used under the dual lane treatment with a volume of interest of 1,000 vph.
FIGURE 35  Trajectory plot of WB:NB route under dual lane treatment with 1,300 vph volume of interest.
5.2.3 *Dual Right Turn Lanes and RTOR*

The volume under the dual right turn lane treatment with RTOR was incremented by 100 vph up to 1,400 vph. Queues for the downstream adjacent intersection reached the levels of the control model at 1,100 vph while the average maximum queue lengths seen at the right turn of the off-ramp of the control model were not met until 1,400 vph as shown in FIGURE 36.
FIGURE 36  Average maximum queue length for dual right turn lane and RTOR treatment at increasing volume of interest.
The average maximum queues seen under the dual right turn lanes with RTOR treatment were very similar to those seen under the dual right turn lane treatment which is to be expected. As TABLE 12 shows, the capacities of both the westbound right turn at the outbound crossover and the through movement at the downstream adjacent crossover were unchanged from the dual right turn lane treatment. Similarly, the extensive queue spillback from the downstream adjacent intersection resulted in very few vehicles being able to turn right on red. FIGURE 37 shows only six vehicles were able to turn right on red from either of the turn lanes during the cycle that extends from simulation second 1,038 to 1,158. Those right turns on red occurred between seconds 1,077 and 1,110. An additional three vehicles per lane per cycle accounts for 13% of the needed capacity per cycle to meet the demand of 1,300 vph.

FIGURE 38 presents a histogram of the maximum queue storage lengths as well as the cumulative distribution function. Adding the RTOR to the dual turn lanes resulted in a reduction in the number of cycles during which the downstream adjacent queue spilled back by roughly 7%. The more drastic change, however, was the impact it made on the maximum queue length at the off-ramp. The spillback at that location occurs 7% less frequently but the 50th percentile maximum queue length reduces from using 40% of the available storage on the off-ramp to just 20%.

TABLE 12  Volume to Capacity Ratio for WBR at Northern Crossover and NBT at Northern Downstream Adjacent Intersection under the Dual Right Turn Lane with RTOR Treatment at Varying Volumes of Interest

<table>
<thead>
<tr>
<th>Volume of Interest</th>
<th>Volume/Capacity</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WBR at NX</td>
<td>NBT at NI</td>
<td></td>
</tr>
<tr>
<td>1,000</td>
<td>0.52</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>1,100</td>
<td>0.55</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>1,200</td>
<td>0.58</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td>1,300</td>
<td>0.60</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>1,400</td>
<td>0.63</td>
<td>0.97</td>
<td></td>
</tr>
</tbody>
</table>
FIGURE 37 Trajectory plot of WB:NB route under dual lane with RTOR treatment with 1,300 vph volume of interest.
FIGURE 38 Proportion of cycles in which fraction of storage is used under the dual lane with RTOR treatment with a volume of interest of 1,000 vph.
5.2.4 RTOR

The volume under the RTOR treatment was incremented by 100 vph up to 1,300 vph. Average maximum queue lengths for the downstream adjacent intersection as well as westbound right from the off-ramp the reached the level of the control model at 1,100 vph as shown in FIGURE 39.
FIGURE 39  Average maximum queue length for RTOR treatment at increasing volume of interest.
Unlike the first three treatments examined, the RTOR treatment did not provide sufficient added capacity to the westbound right turn from the off-ramp as shown in TABLE 13. Whereas the first three treatments had sufficient capacity but were limited by queue spillback from the downstream adjacent intersection, the volume to capacity ratio for the westbound right from the off-ramp under this treatment quickly reached unsustainable levels nearing 1.0.

**TABLE 13 Volume to Capacity Ratio for WBR at Northern Crossover and NBT at Northern Downstream Adjacent Intersection under the RTOR Treatment at Varying Volumes of Interest**

<table>
<thead>
<tr>
<th>Volume of Interest</th>
<th>Volume/Capacity</th>
<th>WBR at NX</th>
<th>NBT at NI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>0.82</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>1,100</td>
<td>0.88</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>1,200</td>
<td>0.93</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td>1,300</td>
<td>0.99</td>
<td>0.95</td>
<td></td>
</tr>
</tbody>
</table>

Furthermore, there were more instances of queue spillback for the westbound right from the off-ramp – 45% of the cycles – than for the northbound through movement at the downstream adjacent intersection – 20% of the cycles as depicted in FIGURE 40. Despite reducing the average maximum queue length under the control volume of 1,000 vph for the westbound to northbound turn from the off-ramp, the treatment could not sustain additional demand.
FIGURE 40 Proportion of cycles in which fraction of storage is used under RTOR treatment with a volume of interest of 1,000 vph.
5.2.5 *Slip Lane*

The volume under the slip lane treatment was incremented by 100 vph up to 1,400 vph. Average maximum queue lengths for the downstream adjacent intersection as well as westbound right from the off-ramp the reached the level of the control model at 1,100 vph as shown in FIGURE 41.
FIGURE 41 Average maximum queue length for slip lane treatment at increasing volume of interest.
The slip lane treatment did not provide much added capacity. TABLE 14 shows the volume to capacity ratio quickly climbed toward 1.0 for the westbound right turn at the ramp and the northbound through movement at the downstream adjacent intersection.

**TABLE 14 Volume to Capacity Ratio for WBR at Northern Crossover and NBT at Northern Downstream Adjacent Intersection under the Slip Lane Treatment under Increasing Volumes of Interest**

<table>
<thead>
<tr>
<th>Volume of Interest</th>
<th>Volume/Capacity</th>
<th>WBR at NX</th>
<th>NBT at NI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>0.73</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>1,100</td>
<td>0.77</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>1,200</td>
<td>0.83</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td>1,300</td>
<td>0.88</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>1,400</td>
<td>0.92</td>
<td>0.97</td>
<td></td>
</tr>
</tbody>
</table>

Unlike the RTOR treatment, the slip lane did a better job of avoiding queue spillback onto the freeway which occurred 18% less frequently. The distribution of the storage space usage, however, was similar between the two treatments with peaks at 40% usage. Both treatments had similar distributions of storage space usage for the northbound through movement at the downstream adjacent intersection. The slip lane, therefore, was a stronger treatment operationally than RTOR because it better reduced queue spillback onto the freeway while matching the queue spillback from the downstream intersection.

The results for the slip lane treatment were particularly impacted by the volume split at the downstream adjacent intersection. If a greater demand existed for right turns at the downstream intersection, it would be expected that the treatment would better reduce queues both at the off-ramp and at the downstream intersection. With 15% of vehicles turning right at the downstream adjacent intersection, the volume reduction the slip lane provided to the northbound approach at the downstream adjacent intersection was 7%; not substantial enough to significantly reduce queues. The greater the demand for right turns at the approach, the greater the volume reduction could be and the more queues could be expected to decrease at both the downstream and off-ramp approaches.
FIGURE 42 Proportion of cycles in which fraction of storage is used under the slip lane treatment with a volume of interest of 1,000 vph.
5.2.6 Summary

All five treatments were tested under increasing volumes for the westbound to northbound right turn from the off-ramp in increments of 100 vph. TABLE 15 shows the additional capacity each treatment provided over the control model.

TABLE 15 Summary of Results of MOEs for Five Treatments under Various Volumes

<table>
<thead>
<tr>
<th>Treatment</th>
<th>MOE</th>
<th>Mvmt</th>
<th>Improvement of Control Model?</th>
<th>Added Capacity (vph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auxiliary Lane</td>
<td>Queue</td>
<td>WBR at NX</td>
<td>y</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at Ni</td>
<td>y</td>
<td>700</td>
</tr>
<tr>
<td>Dual Turn Lanes</td>
<td>Queue</td>
<td>WBR at NX</td>
<td>y</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at Ni</td>
<td>n*</td>
<td>0</td>
</tr>
<tr>
<td>Dual Turn Lanes with RTOR</td>
<td>Queue</td>
<td>WBR at NX</td>
<td>y</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at Ni</td>
<td>n*</td>
<td>0</td>
</tr>
<tr>
<td>RTOR</td>
<td>Queue</td>
<td>WBR at NX</td>
<td>y</td>
<td>&lt;100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at Ni</td>
<td>y</td>
<td>&lt;100</td>
</tr>
<tr>
<td>Slip Lane</td>
<td>Queue</td>
<td>WBR at NX</td>
<td>y</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NBT at Ni</td>
<td>y</td>
<td>100</td>
</tr>
</tbody>
</table>

*statistically significant negative impact

The auxiliary lane treatment provided the strongest reduction in queues at both the off-ramp and at the downstream adjacent intersection. The amount of storage space most frequently used at 1,700 vph was less than 10% of what was available, but it was an unstable binomial distribution. When vehicles were able to flow at free-flow speeds, the storage space used was less than 10%, but any reduction in the speed of the turn – due to moderate queue lengths or other disruptions downstream – resulted in a significantly increased, although short lived, queue length.

Second in increased capacity for the off-ramp right turn movement at 1,400 vph was the dual turn lane with RTOR treatment. The average maximum queues seen under the dual right turn lanes with RTOR treatment at the downstream adjacent intersection were very similar to those seen under the dual right turn lane treatment which was expected. The more drastic change, however, was the impact it made in the maximum queue length at the off-ramp. The spillback at that location occurs 7% less frequently but the 50th
percentile maximum queue length reduces from using 40% of the available storage on the off-ramp to just 20%. The dual turn lane treatment, which provided an additional 300 vph added capacity to the right turn off-ramp movement, along with the dual turn lane with RTOR treatment, provided added capacity to both the westbound right turn and the northbound through at the outbound crossover. This capacity was largely unused, however, because no extra capacity was available at the downstream adjacent intersection. Queue spillback from the downstream adjacent intersection into the crossover occurred with just 100 added vph to the right turn off-ramp movement for both treatments.

The slip lane treatment allowed for 100 added vph to the right turn movement from the off-ramp before the average maximum queue lengths matched those seen in the control model. The results for the slip lane treatment were particularly impacted by the volume splits at the downstream adjacent intersection. If there were a greater demand for right turns at the downstream intersection, it would be expected that the treatment would have had a greater impact reducing the queues both at the off-ramp and at the downstream intersection. The RTOR treatment could not sustain an additional 100 vph for the right turn from the off-ramp.

Overall, there was a trade-off among most of the treatments between limiting queue spillback from the downstream intersection into the crossover and queue spillback from the right turn at the off-ramp onto the freeway. The dual turn lane and dual turn lane with RTOR treatment were strong at the latter by increasing the capacity of the right turn. At the same time, the capacity of the northbound through movement at the outbound crossover was increased. This dual increase in capacity with no increased capacity at the downstream adjacent intersection resulted in extensive spillback into the crossover – 33% of the time for the dual turn lane with RTOR treatment and 40% of the time for the dual turn lane treatment under the control volume. Conversely, the slip lane and RTOR treatments were strong at keeping queue spillback on the corridor low – occurring only 18% of the time for each treatment under the control volume – but this was because they did little to increase the capacity of the right turn from the off-ramp or the northern
through movement at the crossover. The slip lane was stronger than the RTOR treatment, however, with spillback occurring 26% of the time under the control volume compared to 43%.

Of all five treatments, the auxiliary lane was the only one to provide increased capacity for the right turn from the off-ramp, the northbound through movement at the outbound crossover, and the northbound through movement at the downstream adjacent intersection. Under the control volume, queue spillback never occurred at the off-ramp or downstream intersection. Even at the increased volume of 1,700 vph, spillback at the off-ramp occurred during 3% of the cycles and spillback at the downstream adjacent intersection occurred during 5% of the cycles.

It is important to note that the slip lane and auxiliary lane results were strongly dependent on the downstream volume splits. If there have been a greater northbound to eastbound right turn demand at the downstream intersection, the usage of the slip lane would have increased, providing further reductions in MOEs – particularly the queue lengths at the off-ramp and at the downstream intersection. The auxiliary lane treatment allowed right turning vehicles from the off-ramp to join with traffic heading north through the northern crossover without merging; however, increasing in the demand for downstream left turns could have caused weaving challenges as vehicles from the auxiliary lane would have attempted to merge with arterial traffic in an effort to move toward the median.

Design of the DDI, whether it is a brand new interchange, a retrofit, or an expansions of an existing DDI, cannot be limited to the DDI itself. The construction limits of the project must reach the downstream intersection to ensure vehicles do not experience a hurry-up and wait condition.
6 CONCLUSIONS AND RECOMMENDATIONS

This thesis sought to determine how five geometric treatments applied to the right turn from the off-ramp affected queue spillback, delay, and stops throughout a DDI corridor. The FHWA CAP-X model was used to develop initial volumes for the base model. The right turn volume from the off-ramp was then increased until substantial queues formed on the off-ramp as well as at the outbound through movement at the downstream adjacent intersection. This was the control volume. A calibrated VISSIM model of a DDI bounded by standard, four critical phase intersections on both sides of the corridor was used in the microsimulation. The five treatments – channelize turn lane with auxiliary lane, dual right turn lanes, dual right turn lanes with RTOR allowed, RTOR allowed, and a slip lane – were then applied to the control model. Finally, the volume of the right turn movement was increased to determine the extra capacity provided by each treatment.

6.1 MAJOR FINDINGS

The following is a discussion of the findings as they relate to the stated objectives of the research:

1. **Determine the impact potential right-turn geometric treatments have on queue spillback at the off-ramp as well as queue spillback at the downstream adjacent intersection.** Each treatment was simulated 15 times with data collected for an hour in each simulation. This resulted in 15 average delay per vehicle measurements for each route, 15 average number of stops per vehicles measurements for each route, and 450 maximum queue length measurements for each movement. The results of testing each treatment under the control volume showed the following:

   - All five treatments improved the average maximum queue length for the right turn from the off-ramp at the 95% confidence level.
   - The auxiliary lane, RTOR, and slip lane treatments improved the average maximum queue length for the outbound through movement at the downstream adjacent intersection at the 95% confidence level. The auxiliary lane was able to increase the capacity of the right turn from the off-ramp movement at the crossover, the outbound movement at the crossover, and the outbound through
movement at the downstream adjacent intersection. The RTOR and slip lane treatments did not substantially increase the capacity of any of the movements.

- The dual turn lane and dual turn lane with RTOR treatments increased the average maximum queue length at the 95% confidence level for the outbound through movement at the downstream adjacent intersection. This was because both treatments increased the capacity of both the right turn from the off-ramp and the outbound movements at the crossover but failed to increase the capacity of the outbound through movement at the downstream adjacent intersection.

2. *Measure the excess capacity the treatments provide to the right turn movement relative to the control, or no-build, scenario.* The volume of the right turn from the off-ramp was increased in 100 vph increments until the average maximum queue lengths for both the right turn from the off-ramp and the outbound through movement at the downstream adjacent intersection met or exceeded the lengths seen under the control model.

- The auxiliary lane treatment was able to increase the capacity of the right turn movement from the off-ramp by 700 vph, for a total volume of 1,700 vph, without meeting or exceeding the average maximum queues seen under the control model. This volume, however, was unstable as it was near the saturation flowrate for the right turn from the off-ramp of 1,924 vph.

- The dual right turn lane with RTOR treatment added an additional 400 vph capacity to the right turn movement at the off-ramp. This capacity was largely unused, however, as the treatment did not increase the capacity of the outbound movement at the downstream adjacent intersection resulting in queue spillback with the addition of 100 vph.

- The dual right turn lane treatment acted similarly to the dual right turn lane with RTOR treatment and added an additional 300 vph capacity to the right turn movement from the off-ramp. This capacity was largely unused, however, as the treatment did not increase the capacity of the outbound movement at the
downstream adjacent intersection resulting in queue spillback with the addition of 100 vph.

- The slip lane treatment increased the capacity of the right turn from the off-ramp by 100 vph. This volume was highly dependent on the demand for outbound right turns from the arterial to the minor street at the downstream adjacent intersection.
- The RTOR treatment was unable to increase the capacity of the right turn from the off-ramp by 100 vph.

The key finding of this research is that the treatments which were effective at reducing the queue of the right turn movement at the off-ramp did so by increasing the capacity of not only that movement but also the capacity of the outbound through movement at the downstream adjacent intersection. The treatments which simultaneously increased the capacity of the right turn from the off-ramp and the outbound through movement at the crossover without increasing the capacity downstream worsened the queue spillback challenges from the downstream adjacent intersection.

Limitations to this work include:

- The adjacent intersection included fixed turning percentages. Increasing the volume of outbound left turns from the arterial onto the minor street may reduce the impact of the auxiliary lane treatment. A heavy left turn demand may cause vehicle to weave from the far right lane though two lanes of traffic into the outside left turn lane. This weave could create safety and operational challenges. Conversely, if the demand for the outbound right turn from the arterial to the minor street were increased, the slip lane may see operational improvements.
- While the base model was calibrated and validated, the treatment models were not re-calibrated. Some calibration factors may be impacted by the installation of treatments including the speed at which drivers turn right from the off-ramp.

6.2 RECOMMENDATIONS FOR FUTURE WORK

These results highlight the need for consideration of the entire corridor when designing or modifying a DDI. Because of the reduced number of critical phases at the crossovers, the DDI can process more vehicles than the adjacent intersections. Therefore, treating one
part of the DDI to operate more efficiently will only add to the burden at the adjacent intersections. Treatments must be considered within a holistic framework.

This research provides an initial analysis of the impacts of the five treatments in isolation. Each treatment has its own unique design considerations which could be varied in future studies. The following lists potential areas of further research:

- The impact of varying the volume splits at the downstream intersection and, in particular, its impact on the slip lane and auxiliary lane treatments. Increasing the right turn volume percentage may improve the MOEs for the slip lane, as more vehicle will use the treatment. Increasing the volume of left turning vehicles may increase the weaving incidence (and drive up delay) as vehicles move from the auxiliary lane to the left turn lane.

- The impact of the location of the lane drop after the auxiliary lane. Various locations should be considered both upstream and downstream of the adjacent intersection to determine the impact of the additional lane of capacity through the intersection.

- The impact of the spacing between the outbound crossover and downstream intersection. A larger spacing would result in a longer queue storage and, therefore, a lower incidence of queue spillback onto the off-ramp and into the crossover.

- Potential signal timing strategies for the off-ramp including coordination treatments with the downstream adjacent intersection. Coordinating the northbound through movement at the crossover with the downstream through movement should be compared to coordinating the westbound to northbound right turn from the off-ramp with the downstream through movement.

- Geometric and signal timing treatments at the downstream intersection to improve operations and reduce queue spillback. Improvement in the throughput of the downstream intersection may reduce queue spillback into the DDI. Once that queue is reduced, traditional geometric treatments at the off-ramp tested in this document may become more useful. For instance, the dual right turn lanes
improve queues at the off-ramp, but ultimately proved futile because of the increased queueing at the downstream adjacent intersection. Improvement of the throughput at that intersection, through geometric and signal timing strategies, should be tested in conjunction with dual turn lanes.
REFERENCES


24. Qi, Y., and D. Li. When should Right Turn on Red be used? Synthesis on Safety of RTOR. In TRB 91st Annual Meeting Compendium of Papers, 2012, pp. 18.


APPENDIX A – AVERAGE DELAY PER VEHICLE RESULTS

Below are the delay measurements for all OD paths.
FIGURE 43  Average delay for base model under increasing volume of interest – extended results.
FIGURE 44  Average delay for varying treatments at 1,000 vph – extended results.
FIGURE 45  Average delay for auxiliary lane treatment under increasing volume of interest.
FIGURE 46 Average delay for auxiliary lane treatment under increasing volume of interest – extended results.
FIGURE 47  Average delay for dual right turn lane treatment under increasing volume of interest.
FIGURE 48 Average delay for dual right turn lane treatment under increasing volume of interest – extended results.
FIGURE 49 Average delay for dual right turn lane and RTOR treatment under increasing volume of interest.
FIGURE 50  Average delay for dual right turn lane and RTOR treatment under increasing volume of interest – extended results.
FIGURE 51 Average delay for RTOR treatment under increasing volume of interest.
FIGURE 52 Average delay for RTOR treatment under increasing volume of interest – extended results.
FIGURE 53  Average delay for slip lane treatment under increasing volume of interest.
FIGURE 54 Average delay for slip lane treatment under increasing volume of interest – extended results.
APPENDIX B – AVERAGE STOPS PER VEHICLE RESULTS

Below are the stop measurements for all OD paths.
FIGURE 55 Average stops for base model under increasing volume of interest – extended results.
FIGURE 56 Average stops for varying treatments at 1,000 vph – extended results.
FIGURE 57 Average stops for auxiliary lane treatment under an increasing volume of interest.
FIGURE 58  Average stops for auxiliary lane treatment under increasing volume of interest – extended results.
FIGURE 59  Average stops for dual right turn lane treatment under increasing volume of interest.
**FIGURE 60** Average stops for dual right turn lane treatment under increasing volume of interest – extended results.
FIGURE 61  Average stops for dual right turn lane and RTOR treatment under increasing volume of interest.
FIGURE 62 Average stops for dual right turn lane and RTOR treatment under increasing volume of interest – extended results.
FIGURE 63  Average stops for RTOR treatment under increasing volume of interest.
FIGURE 64  Average stops for RTOR treatment under increasing volume of interest – extended results.
FIGURE 65  Average stops for slip lane treatment under increasing volume of interest.
FIGURE 66 Average stops for slip lane treatment under increasing volume of interest – extended results.