RESEARCH & DEVELOPMENT

Exploring Corridor Operations in the Vicinity of a Diverging Diamond Interchange (DDI)

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Abstract

This research effort examined the corridor impacts of various signal timing and geometric strategies to improve the operational challenges observed at DDIs. A microsimulation analysis was conducted using a calibrated and validated DDI modeled after the National Avenue and US-60 interchange in Springfield, Missouri. Four heavy volume scenarios were tested in combination with seven categories of strategies. These strategies were selected from a larger pool of strategies under the guidance of the NCDOT research panel and national expert recommendations. In addition to the microsimulation effort, a cost analysis was conducted for the same strategies. Considerations were made for implementation cost, disruption to user during implementation, and crash modification impacts. Finally, three sites in North Carolina were selected for field study. In the microsimulation analysis, those strategies which reduced the number of phases at the downstream adjacent intersection had the greatest benefit on the corridor routes for all four heavy volume scenarios. The reduction in phases reduced loss time and increased capacity for the intersection. Unfortunately, these strategies were also the most expensive alternatives studied, were likely to be the most disruptive to users during implementation.

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EXECUTIVE SUMMARY

This research effort examined the corridor impacts of various signal timing and geometric strategies to improve the operational challenges observed at DDIs. A microsimulation analysis was conducted using a calibrated and validated DDI modeled after the National Avenue and US-60 interchange in Springfield, Missouri. Four heavy volume scenarios were tested in combination with seven categories of strategies. These strategies were selected from a larger pool of strategies under the guidance of the NCDOT research panel and national expert recommendations. In addition to the microsimulation effort, a cost analysis was conducted for the same strategies. Considerations were made for implementation cost, disruption to user during implementation, and crash modification impacts. Finally, three sites in North Carolina were selected for field study. In the microsimulation analysis, those strategies which reduced the number of phases at the downstream adjacent intersection had the greatest benefit on the corridor routes for all four heavy volume scenarios. The reduction in phases reduced loss time and increased capacity for the intersection. Unfortunately, these strategies were also the most expensive alternatives studied, were likely to be the most disruptive to users during implementation.

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1.1 OVERVIEW OF DDI OPERATION

1.1.1 General

Diverging Diamond Interchanges (DDI) also known as Double Crossover Diamonds (DCD) were first built in France during the 1970s, but haven't been used in the U.S. until much more recently. Chlewicki popularized the design in America with a paper explaining the interchange at the 2nd Urban Street Symposium in Anaheim, California in July 2003 *(1)*. Since the first American DDI opened in Springfield, Missouri at I-44 and MO-13, more than 70 additional DDIs have opened across the country with 12 under planning or construction in North Carolina as of October 2013. At the time of this report, nine DDIs are open to traffic in North Carolina, with various others in the planning or construction stage.

Like a diamond interchange, the DDI features two signals and four access ramps. Right turns from the off-ramp onto 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 which makes a DDI unique. At the two-phased signal, inbound arterial traffic switches from the right side of the road to the left, switching back to the left side at the outbound signal. The switch means left turns both from the off-ramp and onto the on-ramp do not need to 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 1.

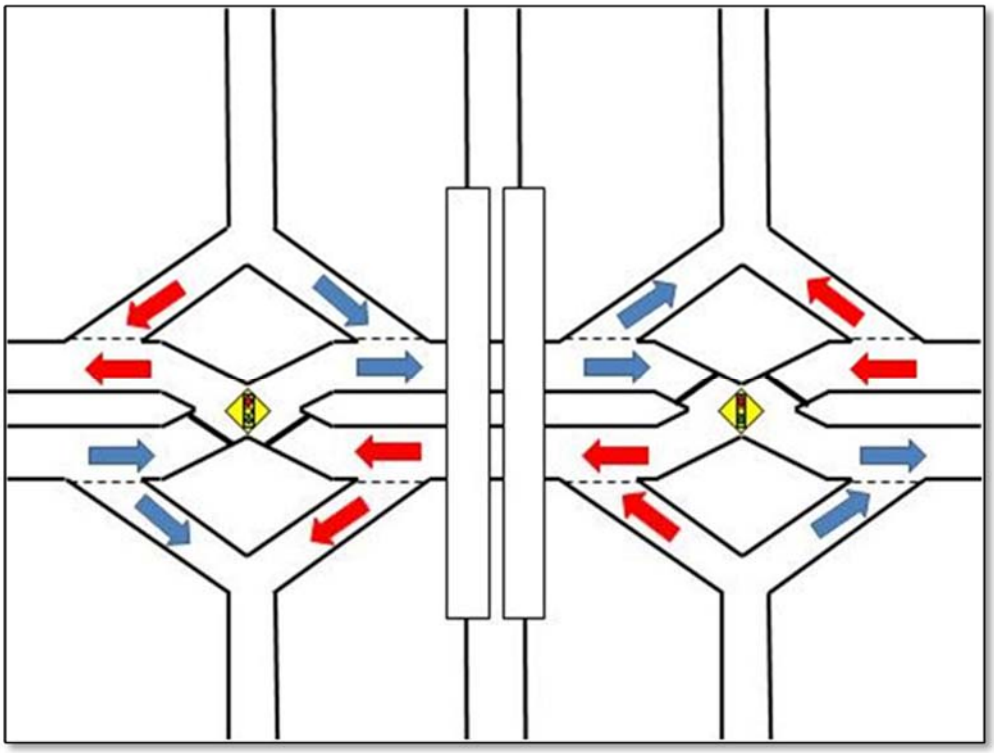


Figure 1 DDI interchange conceptual diagram

1.1.2 Geometry

DDIs are a popular solution because of their relatively low retrofit cost compared to other interchanges. Unlike some other alternative interchange designs, the DDI can often be built on the same footprint as an existing diamond interchange little need for additional right of way acquisition. When compared with 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 right of way to be acquired in the areas highlighted in red in Figure 2.

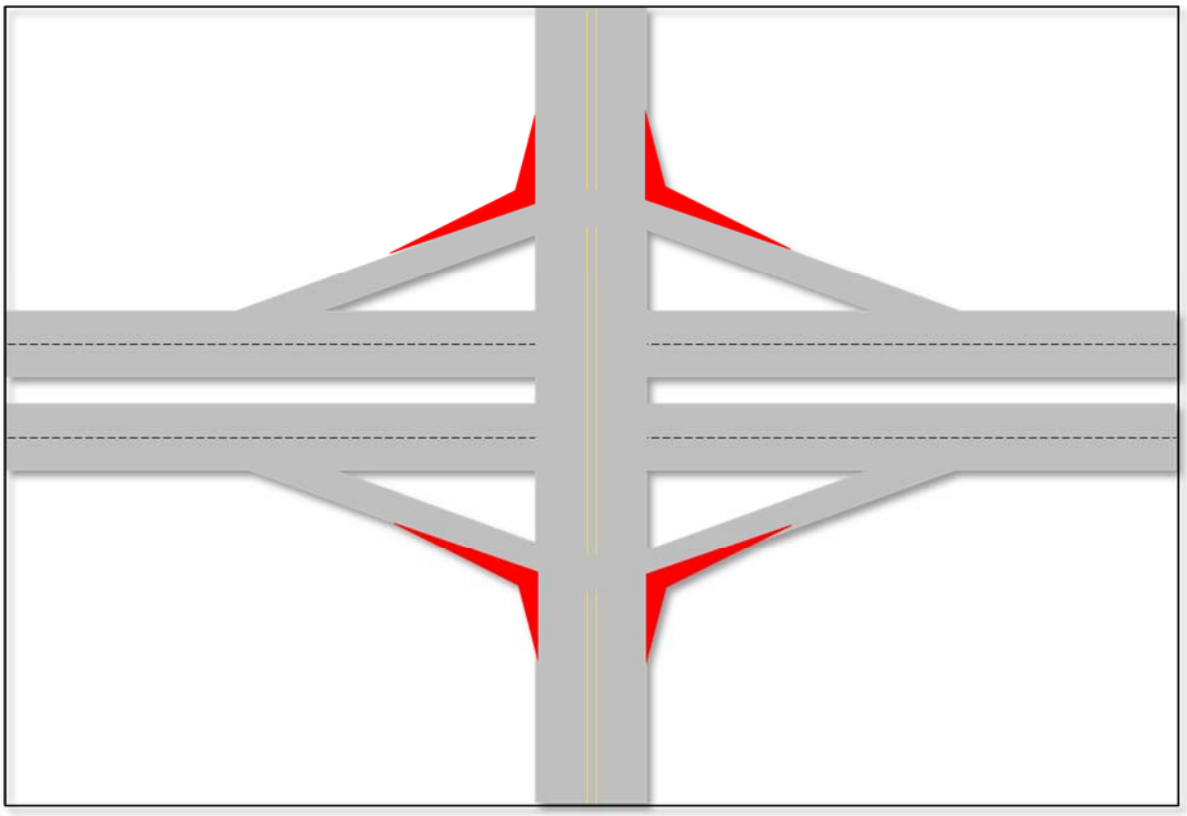


Figure 2 Additional Right-of-Way Required for DDIs

Despite the right-of-way acquisitions that may be necessary, the design cuts costs by reusing the existing bridge. Given a set number of lanes, DDIs have more capacity than the standard diamond interchange and therefore widening of the bridge is often not necessary when the DDI is installed *(2)*. Most DDIs in operation in the United States are designed with one bridge, but some designs use existing twin bridges for the opposing traffic flows.

1.1.3 Signal Operations

From an efficiency standpoint, DDIs are most beneficial at interchanges with high through or left-turn volumes. The signal control at DDIs can be simplified to as little as two critical phases, which can provide additional green time to through traffic as well as left turns from the off-ramp. Left and right turns onto the freeway entrance ramp can be free-flowing but 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-critical phase signalization scheme. Figure 3 shows the signal timing at a DDI with unsignalized movements from the freeway. The two crossover signals feature a simple alternating pattern between phases 8 and 6, and 4 and 2, respectively. Ring 1 corresponds to the left hand signal while Ring 2 controls the right signal.

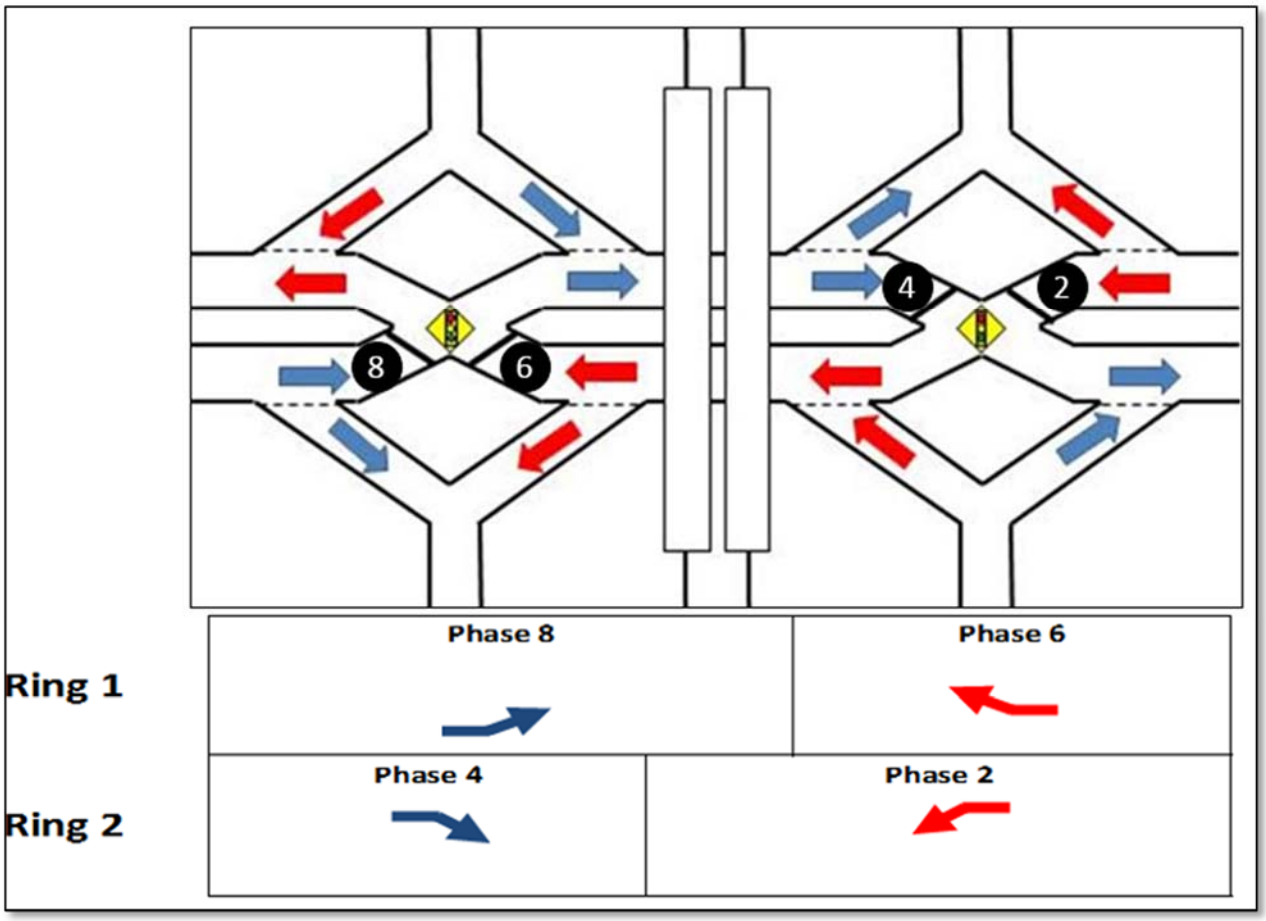


Figure 3 Two Ring Signal Phase System used in Springfield, MO (3)

The addition of signalized turns to and from the on- or off-ramps can lead to more complicated phasing and additional research is underway federally to provide more specific DDI signal timing guidance.

1.1.4 Safety

Adding to the attractiveness of the DDI is the reduction of conflict points from 30 in the diamond interchange to 18 in the DDI. Edara et al. found a conversion from a standard diamond interchange to a diverging diamond interchange resulted in 62.6% fewer crashes *(4)*. The safety benefits of DDIs have since been confirmed by a FHWA-sponsored study, which found a 33% reduction in crashes *(5)*.

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 may believe the requirement of drivers to move to the left side of the road to be unsafe. Jackson et al. found, however, that once the DDI opens for operation, the public soon becomes accustomed to the system *(6)*. Furthermore, studies have shown that after the intersection becomes operational, drivers believe it leads to a reduction in delay, improvement in flow, and safer conditions than the standard diamond interchange *(7)*.

1.2 PROJECT OBJECTIVES AND SCOPE

The objectives of this research are to provide NCDOT and other agencies in North Carolina with objective, scientific guidance on how DDIs compare in a corridor context, and more importantly how the DDI performance can be improved by using the available capacity offered by the efficient two-phase signals.

With NCDOT being on the forefront of unconventional intersection designs, superstreet intersections are one high-potential strategy for intersections adjacent to a DDI, which similarly allow for a two-phase signal operation. However, other alternatives exist such as 1) using a half-cycle concept to better integrate short DDI cycles with long cycle lengths at adjacent intersections, 2) using other alternative intersections ideas, 3) signal timing strategies (creative lead/lag timing), or 4) considering other geometric improvements to increase capacity. In particular, the project will inform the NCDOT on which variations and alternatives might be worthy of detailed analysis given set of demands and a physical context.

The main components of the research were to identify a list of DDI corridor variations, determine which of those are worth testing, to design those variations and alternatives, to simulate those variations and alternatives for traffic operations, and to estimate other measures for those variations and alternatives. The primary research products are recommendations on whether and where NCDOT and other agencies should use the various DDI variations and with what configuration of adjacent intersections. The recommendations can be implemented by traffic engineers, planners, and designers at many stages of thinking about interchange improvements, from new TIP projects to quick safety countermeasures.

The specific scope and objectives are further described below.

1.2.1 Simulation

The VISSIM microsimulation package was used to perform the analyses allowing the team to conduct detailed sensitivity analyses of multiple strategies under four volume scenarios as outlined in Section 5.2.4.2. Each simulation included a corridor with a DDI at the center and a standard, four leg intersection on either side of the arterial. The physical scope of the simulation study area extended on the arterial from upstream of the closely spaced adjacent intersection to immediately downstream of the downstream adjacent intersection. At the interchange, the study area began at the gore between the freeway and the exit ramp and extended to the gore between the freeway and the entrance ramp.

1.2.2 Field Study

Data was gathered from three sites within North Carolina including I-85 and Poplar Tent Road in Concord, I-77 and Catawaba Avenue in Cornelius, and I-85 and NC 73 in Concord. Signal timing strategies were implemented at all three sites in an effort to improve travel times through the corridor. These sites were used as case studies to evaluate the effectiveness of the solutions and provide validation of the simulations.

1.2.3 Guidance

Assistance was sought from industry and research leaders in the field of alternative interchange design and signal timing to assist in identifying potential corrective measures for operational challenges observed at DDIs. The NCDOT Steering Committee along with the national leaders assisted in reducing the preliminary list of corrective measures to a final set for analysis. This reduction was based on the solution's potential for success, the ability for recreation in a microsimulation environment, and the potential for implementation in North Carolina.

1.2.4 Recommendation

The ultimate objective of this research was to provide NCDOT and other agencies with objective, scientific guidance on how DDIs perform in a corridor context, and how strategies applied to the DDI and adjacent intersections perform operationally. In particular, the project informs the NCDOT on which strategies might be worthy of detailed analysis given a set of demands and a physical context.

2 LITERATURE REVIEW

2.1 INTRODUCTION

This literature review summarizes various aspects of DDI design, signal timing, and operations. It is divided into four sub-sections: (1) simulation analysis of DDIs, (2) field-evaluation results of DDIs, (3) DDI signal timing, and (4) a brief review of other alternative interchange forms.

Due to the relative newness of DDIs, many of the issues addressed in this report have not yet been studied with specific respect to the impacts at DDIs. Therefore, the literature review focuses on existing reports on DDI design, simulation, and analysis and is supplemented with research on issues of interest as they relate to standard interchanges and intersections. A review of research on other alternative interchanges in corridors is also included.

2.2 SIMULATION ANALYSIS

Chlewicki *(1)* compared the operations of a DDI and a standard diamond interchange with fixed time signals. Synchro 5 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 being 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 delay, the DDI still had the 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. This is of particular note when the signalized left turn is desired for safety reasons.

Bared et al. *(8)* 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 with 15% left turns, 18% right turns, and 28% through traffic. The four-lane DDI was directly compared to a standard diamond interchange of unspecified size on the measures of total delay per vehicle, stops per vehicle, stop time per vehicle, and maximum queue lengths. 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 (ie. 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. No calibration or validation of the model was mentioned likely because the simulation predated the first operational DDI in America.

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 approximately 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 *(9)* 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 1000 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. As with Bared et al., the simulation predated the first American DDI, so it was unknown how closely the DDI simulation modeled real world conditions. 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 – eastbound to north bound, heavy left turn – westbound to southbound, heavy through movement along the crossroad, and heavy exchange movement. In addition to the four movement scenarios, an adjacent intersection was added to either side of the interchange to feed traffic into the system. Three scenarios were used: the arterial – heavy turning movement feeds into and out of the interchange area, the collector – turn vehicles mainly reach and leave 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 the effectiveness increased when ramp traffic increased but surface street 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) *(2)*. Using VISSIM, three different DDI geometries and two different traditional diamond geometries were tested using 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 of high cost

The simulations done for the AIIR increased volumes proportionally instead of testing different volume combinations.

2.3 FIELD EVALUATION OF DDIS

After the first DDI was built at MO-13 and I-44 in Springfield, Missouri, the Missouri Department of Transportation (MoDOT) *(7)* 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 of the DDI having created any new types of crashes not seen at standard signalized intersections. 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) *(10)* conducted a similar observational study. Key observations included:

- The DDI works well with high demand from the interstate to the cross street or high through traffic over the interchange and 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 the heavy demands exist 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.

Hummer et al. *(5)* conducted a multi-year study of seven DDIs located across the United States. 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. 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.

2.4 SIGNAL TIMING

2.4.1 Closely Spaced Intersections

Because delay is the measure of effectiveness for interrupted flow level of service, all sources of delay must be considered in designing the optimal signal timing plan. Closely spaced intersections have unique types of delay not seen at independent intersections. El-Zohairy and Benekohal *(11)*, Benekohal and Kim *(12)*, and Kim and Benekohal *(13)* all consided platooning impact on delay while Rouphail and Akcelik *(14)* as well as Ahmed et al. *(15)* quantified delay caused by downstream queue interaction.

Chaudhary and Chu *(16)* created guidelines for phasing and control strategies of one intersection located close to a diamond interchange. Their research suggested cycle length had a great impact on the effectiveness of the control strategy. The interchange signals and adjacent signal must operate on a common cycle length which is dependent on the phasing strategy selected. Lieberman et al. *(17)* used principles of shockwave theory to derive an optimization policy for signal timing of oversaturated systems. Focus was given to maximizing throughput, preventing cycle failure, and metering traffic by controlling queue spillback.

To provide a more detailed comparison of signal timing techniques along a corridor with closely spaced standard intersections, French and French *(18)* used Synchro/SimTraffic models to analyze four different signal timing alternatives on a congested, five intersection corridor. Calibration of the SimTraffic model was achieved through iterative adjustments to inputs, mostly involving signal timing. Queue discharge headways, travel time, and cycle failure were used to validate the models.

The four test models included: Synchro optimized coordinated plan with the original phase scheme; customized coordinated timing plan with the original phase scheme; a coordinated corridor with a new phase scheme; and an actuated corridor with a new phase scheme. The new phase scheme was developed after site visits determined it was unlikely additional capacity could be achieved through timing changes alone. A permitted left turn at the closely spaced, two intersection bottleneck was converted to a protected turn to provide more green time to the opposing through movement. The phase change combined with coordinated signals throughout the corridor instead of individually actuated signals provided the greatest benefits by reducing corridor delay between 20 and 60% as measured by SimTraffic.

The Signal Timing Manual *(19)* provides direction for coordination of signals and NCHRP 03-90 details specific strategies for oversaturated signals.

2.4.2 Corridors

Zhang et al. *(20)* used a network kinematic wave model to develop local synchronization control (LCS) strategies to limit the amount of traffic entering heavily queued areas while increasing the amount of traffic discharging from the same area. This technique is commonly known as traffic metering and works to distribute the queue over a wider area, allowing better throughput through the entire system. Unlike other techniques, LCS needs only to detect queues and communicate information locally. LCS was found to compare favorably to global optimal control and outperform isolated control on the basis of total network travel time.

Along corridors, a common means for reducing delay is to create bands of green time for the mainline movement allowing vehicles to arrive on green at a series of intersections. Tian et al. *(21)* tested 100 signal systems on corridors of two to five signalized intersections in an effort to find the maximum bandwidth solution. Each intersection pair had a randomly assigned travel time between 15 and 25 seconds to represent different speeds and spacing. Different travel times in opposing directions were allowed. More than 70% of all scenarios achieved the maximum bandwidth by using a combination of lead-lag phasing instead of leading or lagging phasing. Zhao and Tian *(22)* further studied the impact of spacing on progression and determined for systems with random, or non-uniform spacing, leading and lagging phasing seemed more advantageous. No matter the phasing scenario, providing progression for both peak and non-peak directions proved increasingly difficult over an increasing number of signals, particularly if the signals are randomly spaced. Zhao and Tian concluded a maximum bandwidth solution could not be found if the system contained more than 16 intersections.

Not only would it be difficult to coordinate, but a bandwidth provided over a long distance may go largely unused if the band is so narrow that vehicles fall out of progression as speeds fluctuate. In an attempt to widen the bandwidth, Tian and Urbanik *(23)* used a system partition technique. In partitioning, the peak direction's band is maximized while the off-peak direction is provided with good progression through a defined subsystem of three to five intersections. Thus, the peak direction had bandwidth throughout the entire system while the off-peak direction stops at a few user defined intersections, but progresses through the majority of intersections within the system. VISSIM simulation found that for large systems, the bandwidth efficiency and attainability were at least doubled in both directions using this technique.

In 1996, the Texas cities of Richardson and Garland retimed 119 signals resulting in 160 second cycle lengths *(24)*. For intersections where two major streets intersected, or major-major intersections, the cycle lengths were tolerable because each approach received similar green time. At intersections where minor streets intersected the major street, or major minor intersections, the long cycle lengths led to excessive delays for the minor approaches. By 2006, the cities decided to use uneven double cycles, also known as uneven half cycles, with skip phasing to reduce delays and improve progression. In the 160 second background cycle shown in Figure 4, the major-minor intersections ran two cycles which were uneven in length but totaled

160 seconds. During one background cycle, all through movements would be served twice but protected left turns served only once. Lead/lag phasing was also used as necessary to achieve the optimal progression. Figure 4 shows one example of the phasing sequence. Note the first cycle is 84 seconds while the second cycle is 76 seconds.

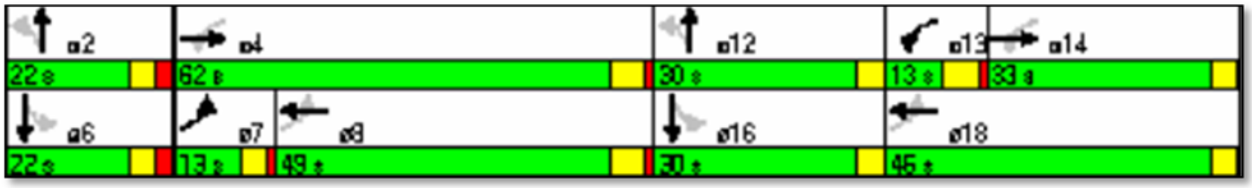


Figure 4 Example Signal Phase for Uneven Half Cycle (24)

Two years after implementation, few complaints had been received from citizens about left turn service only once every two cycles *(25)*. Kurfees surmises this may be because sequence variation was commonly used before the implementation of uneven half cycles and as such citizens did not expect a particular left turn to always lag or always lead. Additionally, permitted turns were allowed at many intersections, so some service was provided during the cycle not receiving the protected left. The use of uneven half cycles allowed for progression in both directions on almost all major arterials, alleviated congestion at the major-major intersections, and reduced wait times at the major-minor intersections.

While many DDI projects focus only on the two intersections within the system, thought should be given to retiming the adjacent signals. The average cost of developing four time-of-day plans is $2500 for 25 to 30 hours of work, but Sunkari *(26)* found the benefits can outweigh the costs as much as 40:1 once emissions, fuel consumption, and collisions due to frequent stopping were considered.

2.4.3 DDI Specific Timing

Due to the unique nature of DDIs where the mainline movements conflict and cannot be run concurrently, brand new signal timing strategies must be developed. The research focus thus far has been on developing timing for the crossovers first with little or no consideration of how to coordinate the system with the corridor. It has been established that DDIs can be run on one or two controllers, actuated or pretimed *(2)*.

Because of the long distance between the mainline overlap and the left turn on-ramp entrance, dummy phases were used in Kansas City, MO. The dummy phase (ϕ3), shown in Figure 5, allowed for two different all-red times, one for the inbound and outbound traffic and a second, longer all-red for the inbound and left turn off-ramp traffic to account for the additional clearance time for inbound traffic to clear the off-ramp approach.

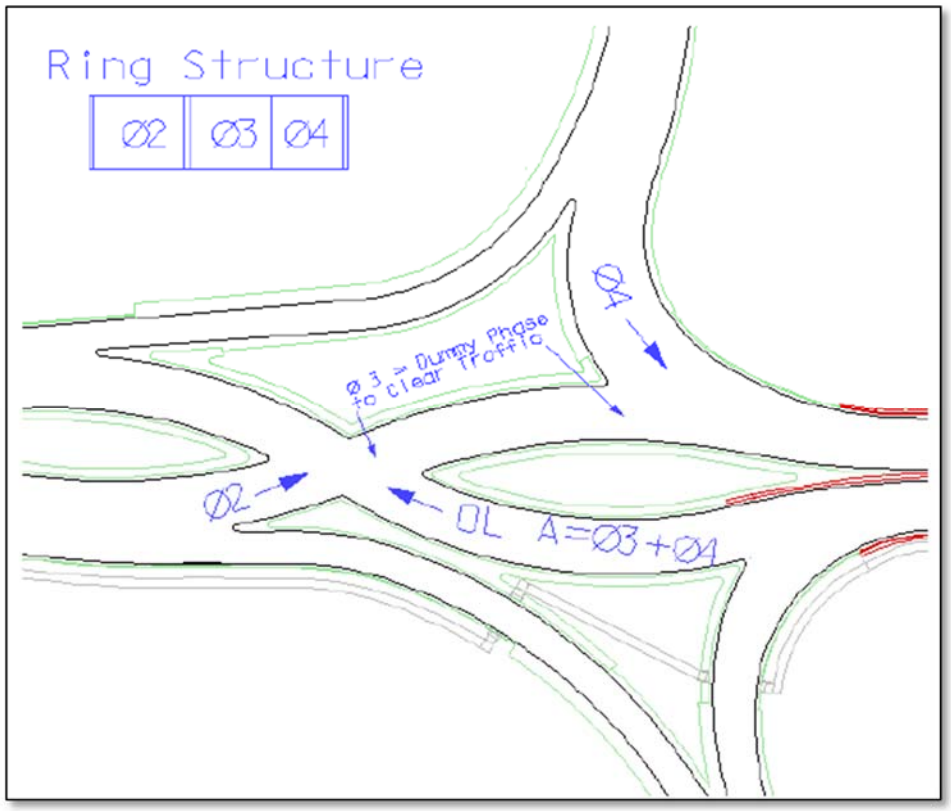


Figure 5 Phasing system for DDI in Kansas City, MO. (2)

Dummy phasing can also be used for outbound traffic and signalized right turns from the off-ramp.

Hu et al. *(27)* developed a signal phase scheme for DDIs using one controller for both intersections. The ring and barrier diagram and phase location diagram are shown in Figure 6 and Figure 7.

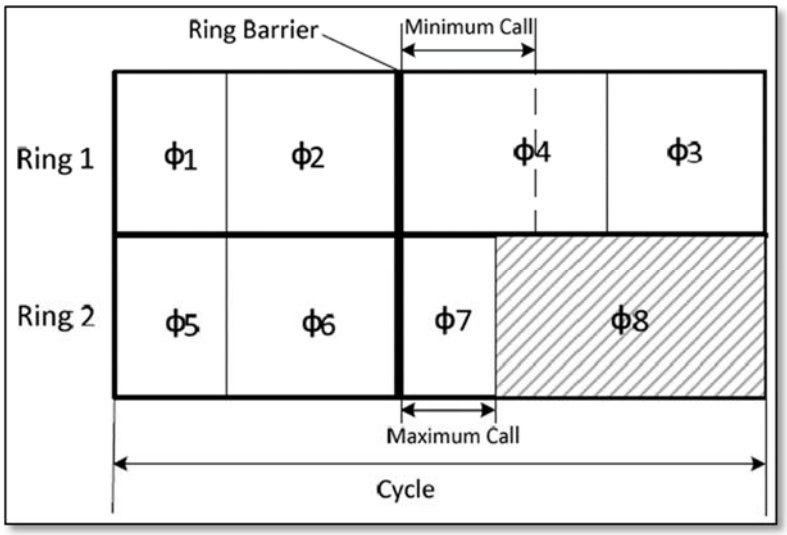


Figure 6 Ring and Barrier Diagram for DDI Signal Phase Scheme (27)

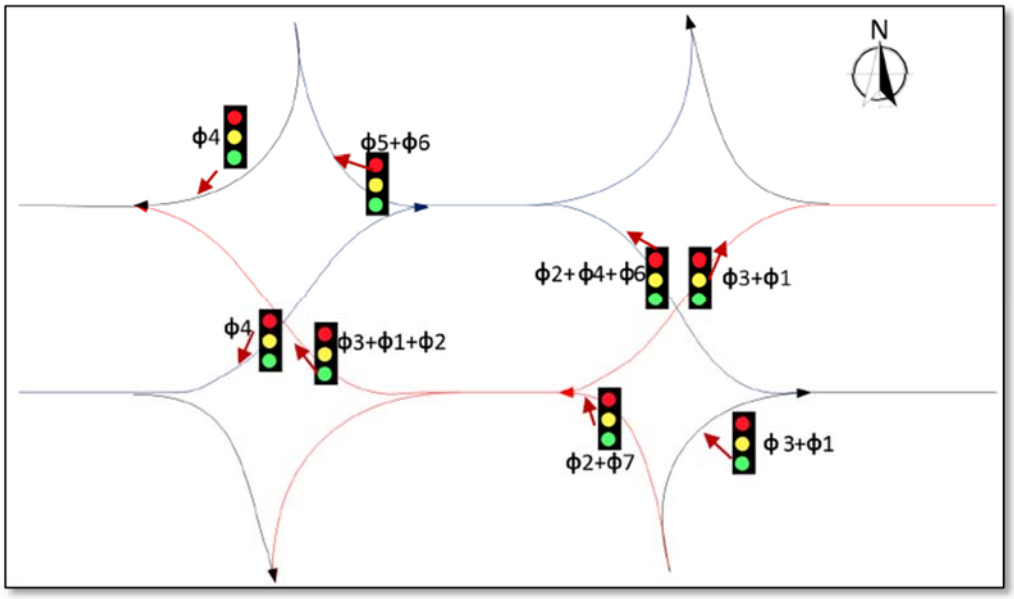


Figure 7 Phase Location Diagram for DDI Signal Phase Scheme (27)

This phasing scheme can be implemented with pre-timed, semi-actuated, or fully actuated controllers but careful consideration must be given to the necessary settings to avoid conflicts. While Phase 8 is not assigned to any movements, it is necessary to call the phase with "Min Recall" to prevent Phase 3 and Phase 7 from running simultaneously as these two phases conflict. Hu also suggests Phase 4 should run longer than Phase 7 to prevent queue spillback. Research conducted by Rouphail et al. *(3)*shows queue spillback on the off-ramp is due to right-turning vehicles, not left turning vehicles. This spillback prevents left turning vehicles from reaching the off-ramp intersection and therefore also prevents queue spillback at the outbound crossover negating the need to stunt the green time of Phase 7.

Hu et al. used VISSIM to simulate the proposed phasing scheme as well as the phasing scheme in use at Moana Lane and U.S. 395 in Reno, Nevada. Details on calibration and validation were not provided. Twelve movements were simulated with both the AM peak volumes and the PM peak volumes. Eight of the twelve movements saw decreased average delay resulting in an overall reduction of 17% in the AM and 28% in the PM. Maximum queue lengths also decreased or remained the same in 13 of the 19 movements (no queues formed at free flow turning movements).

Simulation was also conducted using VISSIM to determine the relationship between cycle length and operational efficiency. Hu et al. developed two signal timing schemes, one for heavy left turns off of the freeway and a second for heavy left turns onto the freeway*(28)*. Five scenarios were tested of proportionally increasing volumes with the base scenario consisting of real traffic volumes at the Moana Lane/U.S. 395 interchange. The five scenarios were tested with pre-timed control and cycle lengths ranging from 60 to 150 seconds. The phase splits were found using the proposed timing schemes. Using the average total delay as the measure of effectiveness, results showed the optimal cycle length ranged from 60 seconds to 130 seconds and increased as the sum of the critical saturation flow ratios increased. The phasing schemes developed by the authors performs well when the critical saturation flow ratios were in the range of 0.76 and 0.85.

Cunningham et al. *(29)* suggested the idea of reducing the number of phases at intersections adjacent to the DDI through various access management techniques. Reducing the number of phases at the adjacent intersections allows the system cycle length of the corridor to be reduced, allowing more throughput at the adjacent intersection and reducing the potential for queue spillback. In addition, reduced cycle lengths at the DDI signals will likely reduce red-light running which was found to be problematic at several DDI's *(5)*.

Chlewicki *(30)* directly addresses the challenge of coordinating DDI signals with adjacent signals. By charting both balanced and unbalanced progression bands shown in Figure 8, a few observations can be made.

- The balanced phases completely overlap when the NB progression meets the SB progressing at ¼ the cycle length
- The NB progression completely passes through the SB progression for a range of time equal to ½ the cycle length ($t_{NB\ Greeen} - t_{SB\ Green}$)
- There is no intersection of the bandwidths when ½ the cycle length is equal to the time needed to reach the next signal. Because this results in no ideal time for the crossing road to receive the green, this spacing should be avoided.
- The dead time where neither the NB bandwidth nor SB bandwidth cross the intersection is the ideal time for the cross road to receive the green.
- Left turns should be protected during the times when only one direction is progressing. Lead-lag phasing may be a preferred strategy for adjacent signals.

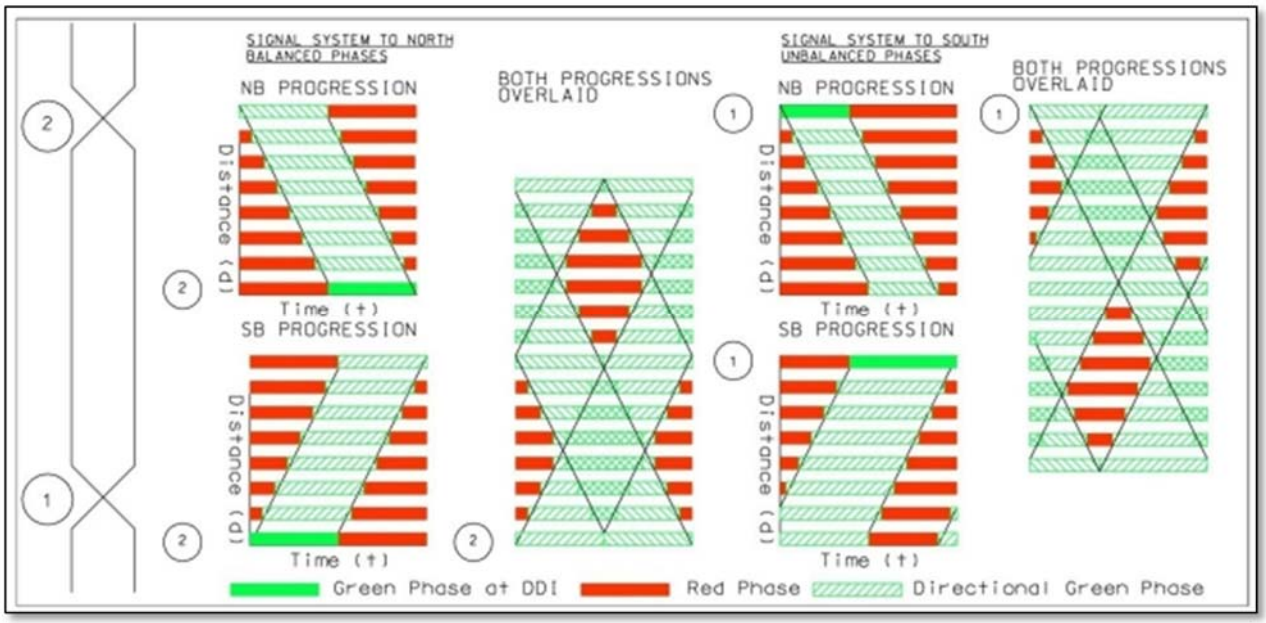


Figure 8 Time-Space Diagrams at Time Intervals Moving Away from DDI

Yang et al. *(31)* developed a model to predict queue lengths on off-ramps over several signal cycles using loop detectors located at the entrance of the ramp, as well as on the freeway and arterial adjacent to the ramp. When queue spillback reaches a predetermined location on the ramp, there is sufficient traffic volumes on the freeway, and the arterial traffic is sufficiently low, signal preemption can be used to flush the ramp of vehicles. Progression is also provided at downstream intersections in the direction of the off-ramp flow. To achieve this, the intersections of interest are adjusted by extending the green time for the corresponding phase. Should the arterial and freeway both have significant traffic volume, cycle failure is allowed at the off-ramp intersection as well as those downstream.

3 OPERATIONAL AND SAFETY CHALLENGES AND STRATEGIES

3.1 INTRODUCTION

This chapter describes the operational and safety challenges which have been commonly observed at DDIs. Additionally, the chapter lists operational and geometric strategies intended to overcome one or more of the operational and safety challenges. In combination with national experts, the team identified a list of preliminary strategies, which were eventually prioritized for evaluation in simulation. Not all strategies listed were selected for simulation or field implementation.

3.2 OPERATIONAL AND SAFETY CHALLENGES AT DDIS

DDIs oftentimes provide significant operational benefits over a standard diamond interchange due to their simplified signal phasing, and ability of left-turns to be accommodated at much-reduced delay. But federal research that conducted detailed observations at over ten DDIs in operation identified a few common operational challenges that may occur at DDIs. These challenges and any recommendation received during the national outreach are summarized below.

3.2.1 Queue spillback from downstream signal

Queue spillback can be caused by insufficient green time for the outbound movement at the adjacent intersection downstream of the DDI. The DDI has been shown to greatly increase the throughput of the interchange *(3)*, however, this efficiency cannot be realized if progression is not established along the corridor. ITRE found in their study of DDIs across the country that the capacity at intersections located close to the DDI are not as great as that of the DDI. This can result in vehicles quickly progressing through the DDI only to be delayed at the adjacent intersection downstream. Queues at the downstream adjacent signal may then spillback into the crossover of the DDI. Figure 9 is taken from the left turn lane at the outbound intersection looking downstream. 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.

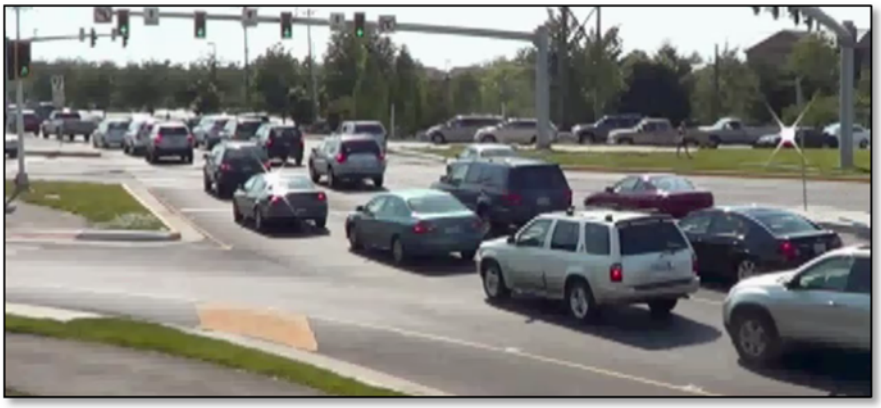


Figure 9 Example of Queue Spillback from the Downstream Adjacent Intersection into the Outbound DDI Crossover(3)

In seeking guidance nationally, one respondent cited a project which was in design at the time of the survey with a frontage road intersection 300 feet from the crossover. The intersection was to be converted to a split T intersection. One half of the intersection would remain located at 300 feet from the crossover but become a Continuous Green T-Intersection (CGT), while the other half would be relocated 600 feet from the crossover.

Another respondent also suggested making geometric adjustments to reduce or reroute the left turns at adjacent intersections in an effort to create more capacity on the mainline. From a signalization perspective, alternate side-street phasing could be used where minor movements are serviced every other cycle. The respondent recommended using a loop detector slightly downstream of the outbound crossover. When the queue from the downstream intersection approaches the crossover, the downstream intersection would be preempted to flush the queue. If no other option is feasible, the adjacent intersection upstream of the inbound DDI intersection could be used to meter traffic into the DDI and artificially reduce demand. This would take the queue formed at the outbound adjacent intersection and transfer it to the inbound adjacent intersection. While this is not ideal, the respondent considered it is safer than allowing the queue to spillback into the crossover.

3.2.2 Demand starvation at the inbound DDI signal

In a similar manner to queue spillback, demand starvation can be a problem at DDIs if the upsteam adjacent signal does not provide enough green time for the inbound traffic. The long green time at the first DDI signal, potentially 50% of the cycle length, can only produce high throughput if the demand is present. When upstream signals cannot provide the demand, the DDI green time is not fully utilized, which must be considered when calculating the capacity of the system. Both queue spillback and demand starvation are a consequence of capacity imbalance between the highly-efficient DDI and typically less efficieny signals upstream and/or downstream.

3.2.3 Failure to yield - right turn at the off-ramp

When Missouri opened the nation's first DDI in Springfield, the right turn from the freeway onto the arterial was yield controlled. But eventually, the agency added a signal for the right turn lane after numerous incidents were observed. One cause of this safety challenge 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 10. Seeing no vehicles in the expected lane of oncoming traffic, the driver attempts to merge and a collision occurs.

Figure 11 is a collection of look-back views from the perspective of a right-turning vehicles 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. Pictures (f), (g), and (h) are various sights from Utah where attempts have been made to extend the distance between the crossover and the right-turn merge location.

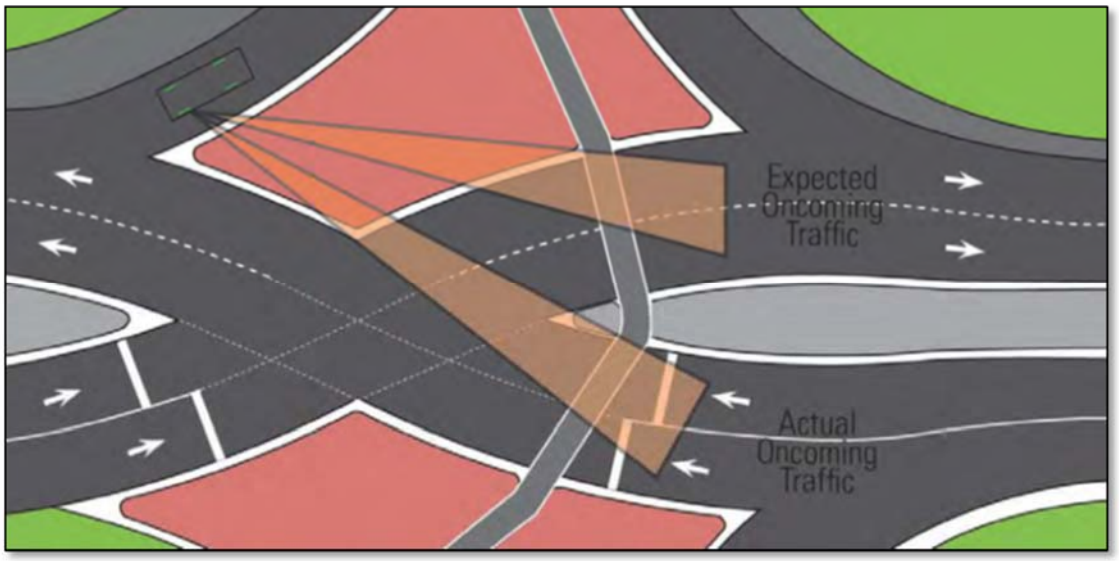


Figure 10 Right-turn Look-back View (32)

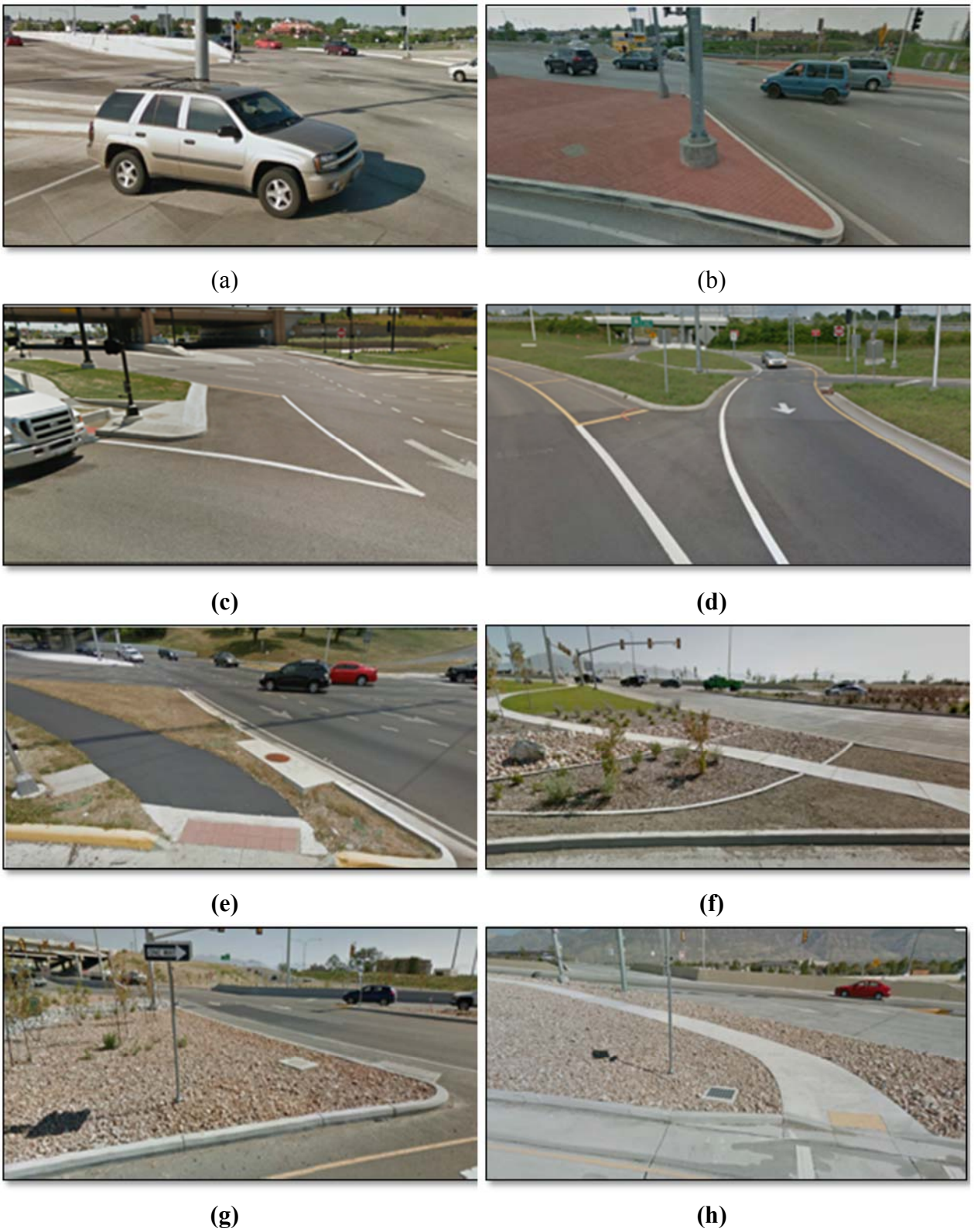


Figure 11 Right Turn Look-Back Views from (a) National Ave; Springfield, MO, (b) MO 13; Springfield, MO (c) Maryland Heights; MO, (d) Alcoa, TN, (e) Lexington, KY, (f) Pioneer Crossing; Lehi, UT (g) SR 92; Highland UT, (h) 500 East; American Forks, UT

In response to the national survey, two respondents suggested signal preemption may be an effective means to reduce queue spillback onto the freeway. A loop detector could be placed on the ramp, downstream of the freeway gore point. When the queue approaches the freeway, the DDI signal would be preempted to call an overlap phase where both the left and right turns from the off-ramp would receive a green signal. Both respondents also suggested creating a channelized turn lane with an acceleration lane. To increase capacity at the downstream adjacent intersection, an auxiliary lane that extends beyond the downstream intersection would be even more beneficial. One respondent further suggested moving the merge point further downstream of the crossover intersection where possible.

Another respondent to the national survey suggested conflicts occur because a high median wall between the crossovers can obstruct the sight distance of the right-turning driver. This can be especially dangerous when the distance between the right turn gore and crossover is particularly close. The respondent suggested locating the off-ramp further downstream of the crossover, designing the off-ramp with a tighter radius to reduce speeds at the conflict point, creating a channelized turn lane with acceleration lane, where possible, and adjusting the entry angle at the right turn entry point. The respondent believed if the stop bar for vehicles turning right from the off-ramp was parallel to the conflicting crossover traffic, drivers would have a better view of the conflicting traffic upstream of the crossover.

3.2.4 Off-ramp spillback

If the right turn at the off-ramp is signalized for safety reasons, queues can form on the off-ramp during peak times. This can result in queue spill back toward and even onto the freeway mainline, which is both an operational and safety concern. This effect can be mitigated with additional turn lanes or other operational strategies and design changes, which are summarized by Warchol *(33)*.

3.2.5 Challenging progression in off-peak direction

A standard intersection is able to simultaneously service anti-parallel through traffic. Techniques have been developed to provide progression through a corridor of uniformly spaced standard intersections for both directions including simultaneous, alternate, and double alternate signal timing. In a time space diagram for a corridor of standard intersections, traffic engineers provide progression by finding green bands as shown in Figure 12.

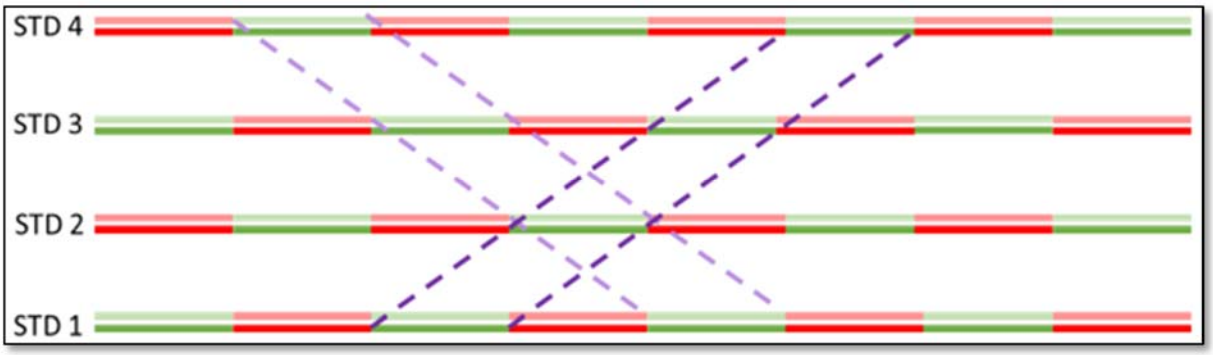


Figure 12 Time-Space Diagram with Progression Bands for Four Uniformly Spaced Standard Intersections

At a DDI intersection, however, the crossover eliminates the ability to simultaneously service both through movements. In a corridor with DDIs, the peak direction (shown in dark purple with signals in dark green and dark red) is still coordinated with the green band, but as Figure 13 shows, the off peak direction (shown in light purple with signals in light green and light red) through the DDI receives a red where the peak direction receives a green rendering traditional coordination techniques unusable.

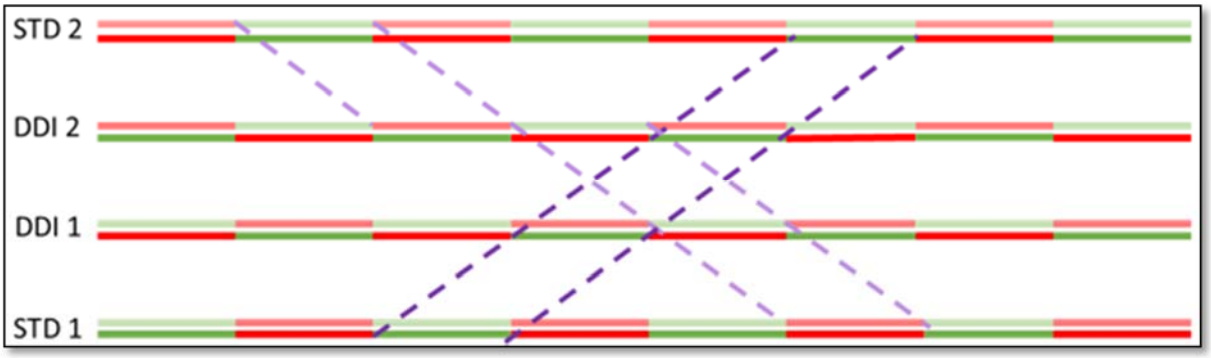


Figure 13 Time-Space Diagram with Progression Bands for Two DDI Intersections Bounded by Two Standard Intersections

It is emphasized that two-way progression through a DDI is still possible, but since the green bands in the opposing directions or not simultaneous, it is more challenging to coordinate a DDI than a conventional intersection or diamond interchange ramp terminal.

3.2.6 Imbalanced lane utilization at the inbound signal

Similar to diamond interchanges, DDIs are prone to imbalanced lane use at the inbound signal as cars pre-position to turn onto the freeway access ramp. The DDI at National Avenue and US-60 in Springfield, Missouri is shown in Figure 14 with the average queue length depicted by black cars and the 95th percentile length in grey cars.

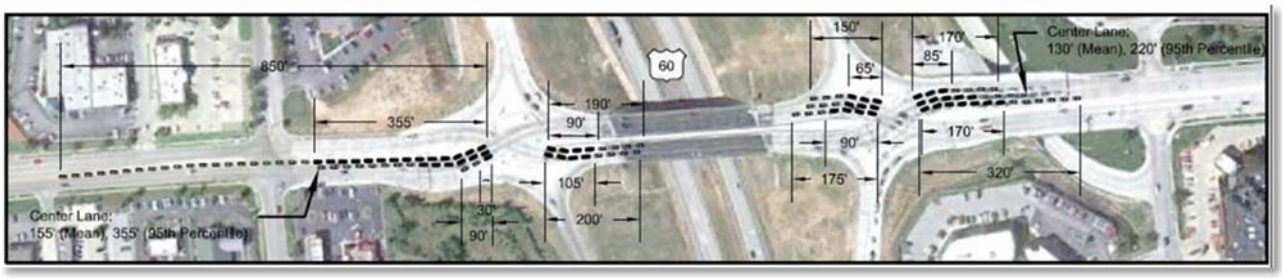


Figure 14 95% Queue in Each Lane at Springfield, Missouri DDI during the PM Peak Hour (34)

While it is not present in at National and US-60, if demand is high for access to both directions of the freeway, queues can simultaneously form in the right and left lanes while the center lane sees a short queue of through vehicles *(34)*. Similar to the demand starvation, imbalanced lane utilization negatively impacts the capacity of the inbound signal because opportunity for service of vehicles exists through the center lane but this green time is wasted because of the aversion of vehicles to wait until the overpass to weave into their necessary lane as well as the aversion of vehicles in the queue to allow them to merge.

3.2.7 Queue Spillback due to ramp metering

For areas where ramp meters are used, the DDI's greatly increased capacity of left turns onto the on-ramp could cause problems similar to the queue spillback seen at the downstream adjacent signal. With a traditional diamond, the relatively low capacity of left turns onto the on-ramp results in vehicles being stored between the interchange ramp terminals. This regulates the number of vehicles waiting at the ramp meter. When a DDI is installed, the increase in capacity can move the queue from being stored within the interchange to being stored on the on-ramp. The resulting queue spillback into the DDI intersection could affect operations for arterial through vehicles, and if severe, block vehicles turning left from the off-ramp of the opposing freeway direction.

3.2.8 Pedestrian pathways

The pedestrian paths at DDIs are not standardized. Some designs direct pedestrians down the center median while others, most often those where the arterial is an underpass, direct pedestrians along the outside of traffic. Because of the difference in traffic flow, pedestrians must be very careful to check for vehicles coming from the correct direction. This can be especially challenging for pedestrians with vision disabilities, and others who may not understand the differences between a DDI and standard intersection.

For those DDIs with pedestrian pathways located on the outside of traffic, vehicles turning left onto the on-ramp can pose a challenge. Those vehicles, preparing to enter the freeway, will be accelerating and may not expect a pedestrian crossing as they make the turn onto the on-ramp.

3.2.9 Red-light running

Hummer et al. found instances of red-light running at some DDI study sites, which may be related to sites with long cycle lengths to control the simple, two-movement crossover signals *(5)*. Because the DDI often provides long green times for each movement, there tends to be some

wasted green time as described in Section 0. This may result in perceived inefficiencies and long wait times for the opposing movement waiting at the crossover. When through movements have low to medium volume and cycle lengths are perceived by the conflicting through movement drivers as unreasonably long, drivers may refuse to comply with the red signal and proceed through the crossover during the opposing movement's wasted green time. However, most red-light running events were observed shortly after the signal transitioned from amber to red, with drivers presumably not wanting to sit through a long cycle before the next onset of green.

3.2.10 Wrong-way maneuvers

Because of the unique geometry, there is some concern of drivers becoming confused as they proceed through the intersection and making a wrong way maneuver. Wrong way maneuvers are most likely to occur as drivers on the arterial approach the crossover and stay to the right of the median instead of going left. Some drivers may also attempt to use the crossover as a location to perform a U-turn. Hummer et al. have extensively documented wrong-way observations at several DDI sites *(5)*.

3.3 PRELIMINARY STRATEGIES

After considering the issues seen at DDIs, a preliminary, comprehensive list of strategies to address those issues was developed. Each strategy was matched with the issue it was intended to solve. Table 1 gives each issue, lists whether it is an operational or safety issue, and provides strategies that address that issue, broken down by either geometrics or signal timing. Below the exhibit is the list of strategies by letter. Strategies in red were not simulated, as described below.

Table 1 Issues Caused by the DDI and Potential Strategies

Issues	Primary Focus Area	Strategies	
		Signal Timing	Geometric Design
Queue Spillback due to low g/c for the throughput at the downstream adjacent intersection	Operations	A, B, C, D	*J*
Demand Starvation due to low g/c for the throughput at the upstream adjacent intersection	Operations	A, B, C, D	*J*
Excessive queues due to poor capacity for the **Right turn at Off-Ramp**	Safety & Operations	*E*, *F*, G	*J*, K, L, M, N, *O*, *P*
Excessive queues due to poor capacity for the **Left Turn at Off-Ramp**	Safety & Operations	*F*, G	
Mainline Progression through the DDI intersections in the non-peak direction	Operations	B, C, H	*J*
Unbalanced Lane Utilization at the inbound DDI intersection	*Operations*		*R*, *S*
Ramp Metering causing queue spillback into the DDI intersection	*Operations*	*F*	*T*

Vehicles not yielding for **Pedestrians** at the left turn at the on-ramp.	*Safety*	*I*	*R*
Safety of **Emergency Vehicles** as they cross the DDI	*Safety*	*F*	
Wrong-way Maneuvers made by vehicles as in the DDI	*Safety*		*Q*

A. Metering of Interchange Inflow from Adjacent Intersection
B. Reduction in Number of Phases at Downstream Adjacent Intersection
C. Lead/Lag Phasing for Outbound Left at Adjacent Intersection
D. Alternate Side-Street Phases at Outbound Adjacent Intersection
E. No Right Turn on Red Allowed at Off-Ramp
F. Preemption of Crossover Signal for Flushing of Off-Ramp
G. Dedicated phase for Concurrent Left and Right Turn at Off-Ramp
H. Half Cycle of DDI Signals
I. Signalize Left Turn at On-Ramp
J. Access Management Control
K. Dual Right Turn Lanes at Off-Ramp
L. Relocate Right Turn at Off-Ramp Downstream of Crossover
M. Channelized Turn Lane with Acceleration Lane for Right Turn at Off-Ramp
N. Right Turn Slip Lane at Off-Ramp
O. Sight Distance Improvement and/or Realignment for Right Turn at Off-Ramp
P. Glare Screens
Q. General Design Considerations
R. Signing and Pavement Markings
S. Addition of Lanes Upstream of Inbound Crossover and/or Adjustment of Lane Assignments for Left Turn at On-Ramp
T. Add Storage Capacity to On-Ramp

This preliminary list was then analyzed to determine which strategies to pursue for simulation. Those strategies which were simulated are detailed in Section 4.2.1. Detailed descriptions of all strategies, including those which were excluded from simulation, can be found in Appendix A.

Those strategies italicized in red were not considered. Installation of glare screens (P), improving the general design (Q), and adjusting signing and pavement markings (R) were not included in the final list of strategies to be simulated because at their root, the goal of those strategies is to impact driver behavior, and that cannot be simulated. Making sight distance improvements impacts driver behavior, and that cannot be addressed in simulation. Signing and pavement markings works to reduce driver confusion which is also not a driver behavior that can be addressed in simulation. Finally, adjusting superelevation or curve radii requires field testing to determine the added comfort to drivers and improved handling of heavy vehicles.

The signalization of left turns at the *on-ramp* (I) was not included in the final list of strategies to be simulated because it did not impact corridor operations as much as it was a safety

improvement for pedestrians. It is not likely that making the left turn at the on-ramp signalized as opposed to free-flowing would create issues with queues. The green indication would be simultaneous with the outbound through movement at the outbound crossover. Therefore, any queue that formed because of the signalization of the left turn at the on-ramp would be a small addition to the queue already present for the outbound through movement.

Addition of lanes and adjustment of lane assignments (S) was not included in the final list of strategies to be simulated. These strategies would work to reduce delay by adjusting lane utilization. Research has already been conducted (ITRE) as to the lane utilization at the inbound crossover in relation to the lane assignments at the downstream left turn on-ramp. Simulating the different lane utilization scenarios to determine the delay does not provide any new information as lane utilization and the corresponding delay is already known (cite to stuff from the HCM?).

Adding storage capacity to the on-ramp (T) was not included in the final list of strategies to be simulated because it does not fit within the scope of this project. Capacity of the on-ramp is a function of the rate at which metered vehicles are released to the freeway and that is a function of the demand and capacity on the freeway. While queue spillback could occur and cause corridor issues, the simulation of freeway demand is beyond the scope of this project.

Adjusting access management control on the corridor (J) was not simulated because the impacts were not expected to be significantly different from the impacts of access management at a standard diamond interchange.

In addition to removing strategies from the final list to be simulated, allowing or disallowing RTOR at the off-ramp (E), sight distance improvements (O), relocating the right turn merge point from the off-ramp (L), the number of right turn lanes at the off-ramp (K), providing a channelized turn lane with acceleration lane at the off-ramp (M), and providing a right turn slip lane (N), were all grouped into one strategy addressing right turn (U) strategies. Relocating the right turn merge point allows drivers better sight distance upstream of the merge. The relocation point should provide drivers with enough sight distance that RTOR would be allowed. In simulation, the sight distance is variable that can be adjusted independent of the geometry. Therefore, to simulate the relocation of the right turn merge point or improving sight distance, the geometry does not need to change in simulation; allowing for RTOR from the standard merge point will provide the same results as allowing for RTOR from a location downstream.

4 METHODOLOGY

4.1 INTRODUCTION

This chapter details the methods used for developing the strategies in simulation. For each strategy simulated, the reason for selection and method of implementation are presented alongside the total number of scenarios tested. Following the explanation of the strategies, the simulation process is outlined including a matrix of which heavy volume combinations were tested with each strategy.

The chapter concludes with information about the before and after field implementation study. Details are provided on the signal timing plans in both the before and after scenarios including the rational for how the "after" timing plan was developed.

4.2 SIMULATION

After pairing down the list of preliminary strategies, a final list containing the strategies which *were both able to be simulated and within the scope of the project* was created. Table 2 is a binary matrix of the final list of strategies to be simulated along with the issues they address, where "x" indicates the strategy does address the issue and a blank indicates the strategy does not address the issue. Note: four of the issues listed in Table 1 are no longer listed in Table 2 because none of the strategies addressing those issues were included in the final list of strategies to be simulated.

Table 2 Binary Matrix of Final Strategies for Simulation and the Issues the Strategy Addresses

Strategies used for each issue

	Metering	Reduced Phases	Lead/Lag	Alternate Side Street	Dedicated Phase	Half Cycle	Right Turn Strategies
Queue Spillback	x	x	x	x			
Demand Starvation	x	x	x	x			
Right Turn at Off-Ramp					x		x
Left turn at Off Ramp					x		
Mainline Progression		x	x			x	

4.2.1 Strategies

For each strategy, there are a minimum of two scenarios that must be tested. The first is the control case where the strategy is not implemented and the second is a test case where the strategy is implemented. Each variation of the strategy crates an additional scenario to be tested. Conclusions about the effectiveness of the strategy are based on the improvements seen over the control case.

A – Metering of Interchange Inflow from Adjacent Intersection
To avoid queue spillback at the downstream adjacent intersection, demand can be artificially lowered by reducing throughput of the upstream adjacent intersection. The capacity of (left, right, and through) movements headed into the DDI to be reduced to a rate similar to that of the outbound capacity of the downstream adjacent intersection. The queue that was spilling back from the downstream adjacent intersection into the outbound crossover would then be transferred to upstream of the upstream adjacent intersection.

The capacity of the outbound approach (blue arrows in Figure 15) at the downstream adjacent intersection is calculated from the splits used in the control model and the saturation flow rate. From this volume, the estimated contribution from the off-ramps (green) are subtracted. Then, the estimated number of vehicles turning onto the freeway (grey) are added. This value is the total inbound throughput which should be produced by the upstream signal (orange). To reduce the capacity of those movements, each of the three splits from the control model which contribute to the throughput are reduced by the same percentage. The additional time is given to the outbound through and left turn at the intersection.

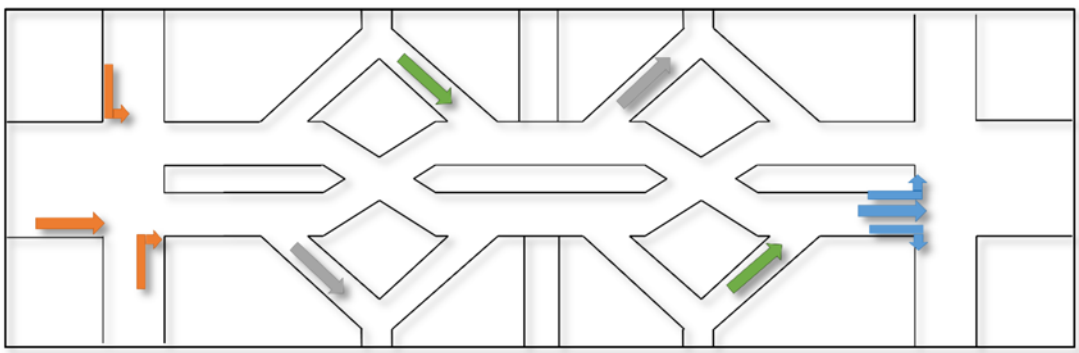


Figure 15 Movements used in the calculation of metering inbound inflow of the upstream adjacent intersection

Number of scenarios: 2

1. Throughput at the upstream adjacent intersection is served at either the capacity of the upstream adjacent intersection, or the demand present at the upstream adjacent intersection, whichever is greater.
2. Throughput at the upstream adjacent intersection is limited such that the throughput into the DDI is no greater than the capacity of the outbound approach at the downstream adjacent intersection, plus the expected volume turning onto the on-ramps, less the expected downstream volume contribution from the off-ramps.

B – Reduction in the Number of Phases at Downstream Adjacent Intersection
This strategy reduces queue spillback from adjacent signals by increasing the capacity of the downstream adjacent intersection. Different geometric adjustments can be made at the downstream intersection to reduce the number of phases and provide the outbound movement

with more green time relative to the cycle length. The added green time also opens more possibilities for increased bandwidth through the corridor for the mainline movement.

The splits and offsets are re-optimized for each scenario. The strategy is tested for a total of four geometries of the adjacent intersections: 1) Median U-turn; 2) Rotated Median U-turn; 3) Superstreet; and 4) Stretched Superstreet.

The rotated median U-turn (RMUT) allows full movement at the main intersection with the exception of left turns on the mainline. These turns are redirected with right turns onto the side street where vehicles can then make a U-turn and proceed back through the main intersection, across the arterial, and onto their destination.

With the stretched superstreet, the outbound left turn is no longer allowed, but the downstream left turn from the side street is allowed as is the concurrent through movement. Figure 16 shows all four geometries as well as the impacted movements.

Number of scenarios: 5

1. Downstream adjacent intersection with a standard eight phase intersection.
2. Downstream adjacent intersection with a Median U-Turn (MUT) intersection.
3. Downstream adjacent intersection with a rotated MUT intersection.
4. Downstream adjacent intersection with a superstreet intersection.
5. Downstream adjacent intersection with a stretched superstreet intersection.

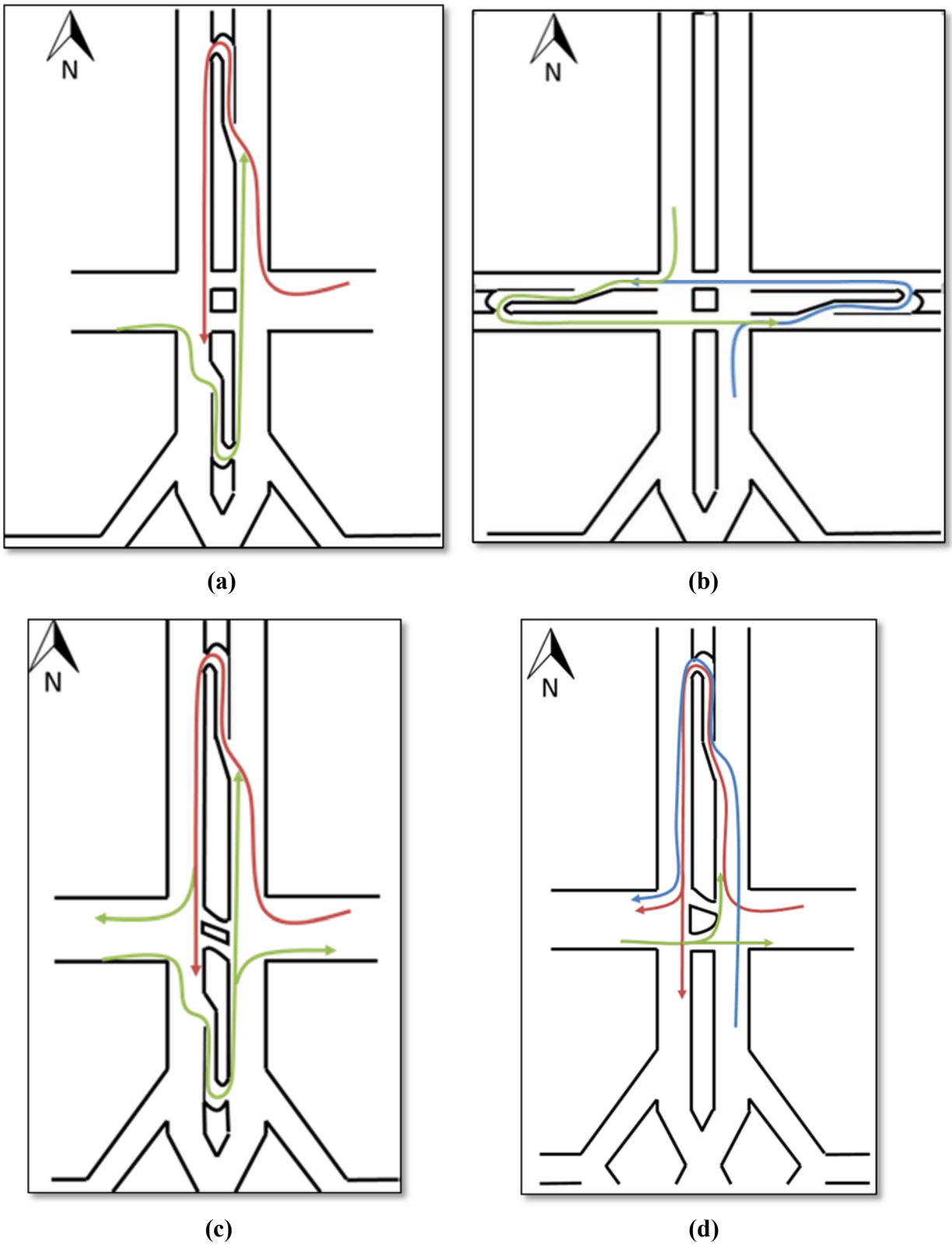


Figure 16 Movements Impacted by Alternative Intersection Geometries (a) Median U-Turn (b) Rotated Median U-Turn (c) Superstreet (d) Stretched Superstreet

C – Lead/Lag Phasing for Outbound Lefts at Downstream Adjacent Intersection
Lead/Lag phasing is being utilized much more frequently by states now that safety concerns with the "yellow trap" have been addressed through flashing yellow arrows and other innovations. By adjusting the outbound left turn to be either leading or lagging, maximum bandwidth can be achieved along the coordinated movement for the purpose of progressing traffic.

In each scenario, the split and offsets are optimized over the four intersections. Additionally, the mainline left turns at the upstream adjacent intersection as well as the inbound left turn at the downstream adjacent intersection are optimized for leading and lagging. In the first scenario, the outbound left at the downstream adjacent intersection leads the opposing traffic. In the second scenario, the outbound left lags the opposing traffic. In the final scenario, the outbound left at the downstream adjacent intersection is served twice during the cycle, once leading the inbound traffic and once lagging it. Figure 17 shows all three phasing strategies.

Number of scenarios: 3

1. The outbound left at the downstream adjacent intersection is served before the opposing through movement (leads).
2. The outbound left at the downstream adjacent intersection is served after the opposing through movement (lags).
3. The outbound left at the downstream adjacent intersection is served both before and after the opposing through movement (twice per cycle left).

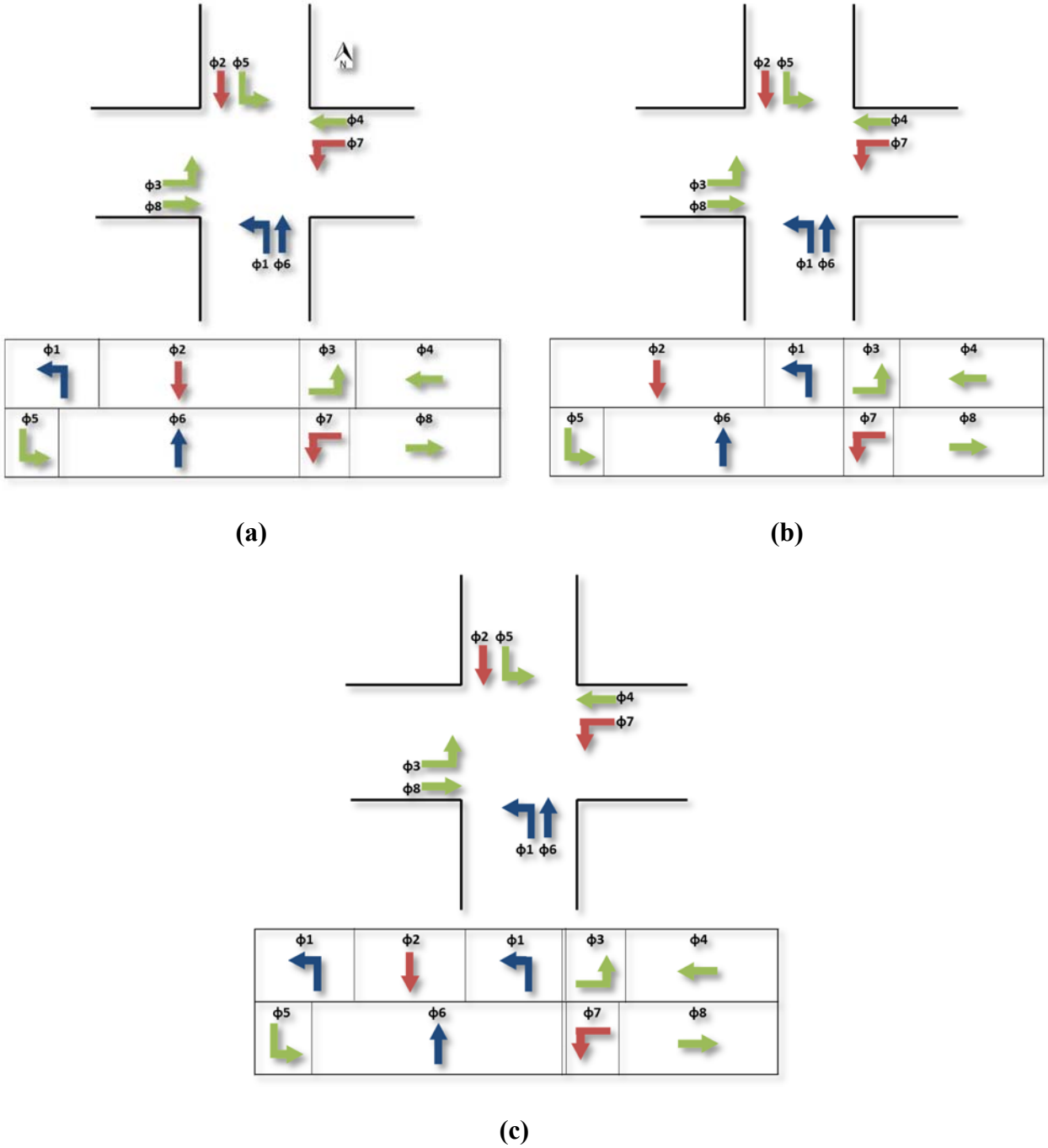


Figure 17 Outbound Left Signal Strategies (a) Leading Left (b) Lagging Left (c) Twice-Per-Cycle Left

A dummy phase is required to achieve optimization for twice-per-cycle lefts. A fifth left turn phase was coded in VISTRO. The volume of the outbound left turn at the downstream adjacent intersection was then split equally between the actual left turn phase and the dummy phase. This ninth phase (shown as the second occurrence of phase 1 in Figure 17) was called in the same ring and between the same barriers as the actual left turn phase but after the opposing traffic. The corridor was then optimized in the manner described above with no optimization of the lagging

or leading left for the outbound left at the downstream adjacent intersection. Note: Although the volume of the left turn was split equally between the actual and dummy left turns, the optimization process did not necessarily result in equal green time for the two turns due to progression opportunities.

D – Alternate Side-Street Phases at the Downstream Adjacent Intersection

The phasing scheme at the downstream adjacent intersection could be adjusted to provide additional capacity to a mainline approach by alternating side street movements every other cycle. This unusual phase scheme could be used in a time-of-day plan when traffic on the mainline is excessive and there is a great need for additional capacity to prevent queue spillback into the DDI interchange. On the next cycle, the mainline is again serviced, followed by the alternate minor approach. While this strategy is similar to strategy B, the difference is that strategy D is a signal timing strategy while strategy B is a geometric strategy.

To code the alternating side street phasing strategy, the standard cycle length was doubled. This allowed for a three ring set-up with all mainline movements being served followed by the through and left movements from one side street. Then the mainline movements are served again, and finally, the opposing side street movements are served. Figure 18 shows the phasing sequence.

Number of scenarios: 2

1. The downstream adjacent intersection serves all movements on both side-streets during each cycle.
2. The downstream adjacent intersection serves all movements on one side-street approach during a cycle. During the following cycle, all movements on the opposing side-street approach are served.

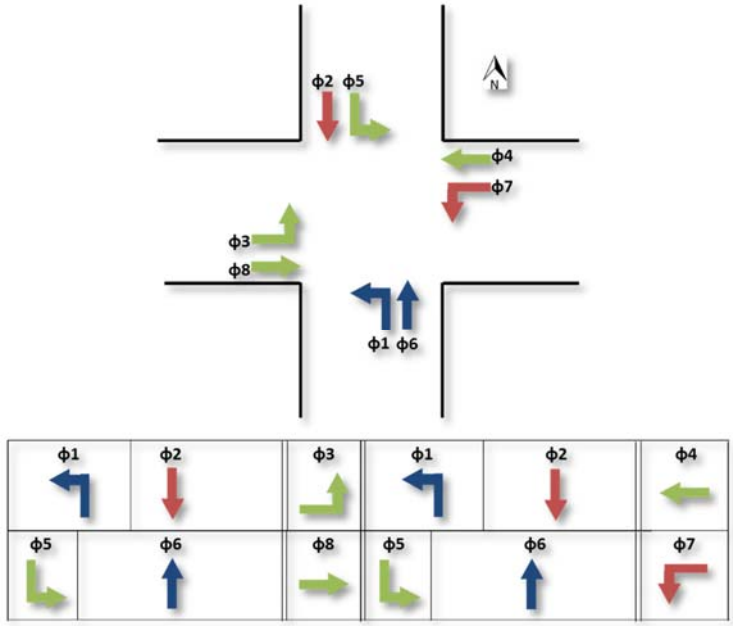


Figure 18 Alternating Side Street Phasing

To achieve optimization of this phasing strategy, the use of dummy phases is required. Four dummy phases were created to mirror the four mainline movements. The volume of each movement is then divided with half allocated to the real phase and half to the dummy phase. The entire corridor is then optimized for splits and offsets. As with twice-per-cycle lefts, the optimization process does not necessarily result in equal green time during the two services of the mainline movements.

G – Dedicated Phase for Concurrent Left and Right Turn at Off-Ramp

For continuously high demand at the off-ramp during peak hours, an additional phase can be added to the time-of-day plan which serves the left and right turns from the off-ramp concurrently, as exemplified by Phase 3 in Figure 19.

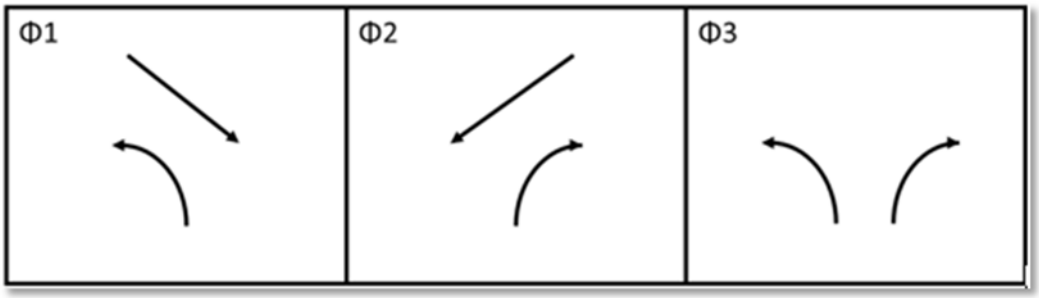


Figure 19 Phasing Schematic with Off-Ramp Turn Phase

To create the dedicated turning phase, a two ring, three-barrier system is used. In the first phase, a mainline movement proceeds with non-conflicting turn from the off ramp. In the second phase, the other mainline and off ramp movements receive green time. In the third phase, both off ramp turn movements are given green time.

To simulate this, the corridor is optimized for split and offset. Then, the green time given to the first and second phases are reduced such that the mainline movements receive only enough green to have a v/C of 0.90. The time taken away from these two phases is given to the third phase. The offsets are then re-optimized.

Number of scenarios: 2

1. No phase is assigned for concurrent left and right turns from the off-ramp.
2. A dedicated phase is programmed into the controller to allow concurrent left and right turns from the off-ramp.

H – Half Cycle of DDI Intersections

Using half cycles at some intersections may provide better progression of the off-peak traffic by opening the green band more often.

The two crossover intersections are coded with cycle lengths half that of the optimal adjacent intersection cycle length from the control model. The corridor is then optimized for split times and offsets.

Number of scenarios: 2

1. The cycle lengths of the DDI intersections are the same as those at the adjacent intersections.
2. The cycle lengths of the DDI intersections are half the length of those at the adjacent intersections.

U – Right Turn Strategies

A number of right turn strategies can be used in combination to reduce the queues at the off-ramp. To increase capacity, right turns on red can be allowed. To provide more capacity with the same signal timing, a dual right turn lane can be installed. If a large number of vehicles turning right at the off-ramp also turn right at the adjacent downstream intersection, a slip lane can be added that directs right turning vehicles from the off-ramp to the adjacent cross street as is shown in Figure 20. If the queue is so great as to need a free-flowing right turn movement, a channelized right turn lane can be added with an acceleration lane that extends through the downstream adjacent intersection.

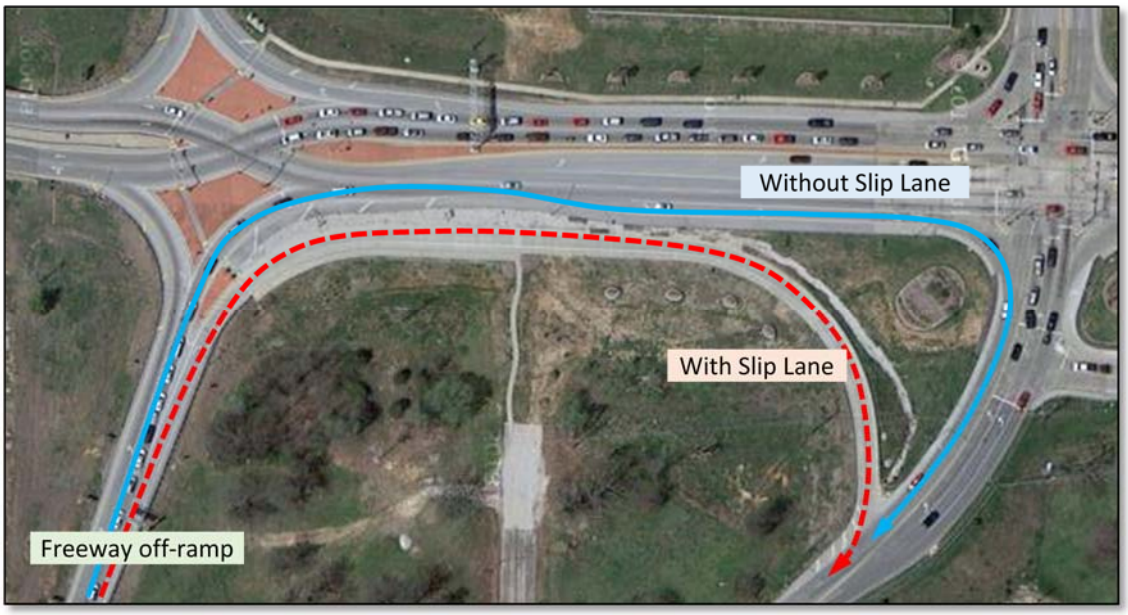


Figure 20 Slip Lane at MO-13 and I-44 in Springfield, MO (Source: Google Maps)

The right turn strategies attempt to reduce queue spillback from the off-ramp onto the freeway by increasing the capacity of the movement. There are five strategies within this category:

- Channelized Turn Lane with Auxiliary Lane: 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.
- Dual Right Turn Lane: two exclusive right turn lanes are provided on the off-ramp instead of the original single lane. 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.
- RTOR Permitted: 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.
- Dual Right Turn Lanes and RTOR permitted: dual turn lanes are provided and RTOR is allowed.
- Slip Lane: 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 20.

Number of scenarios: 6

1. There is a single right turn lane and RTOR is disallowed.
2. There is a single, channelized right turn lane with an acceleration lane which drops downstream of the downstream adjacent intersection. RTOR is disallowed.
3. There is a dual right turn lane and RTOR is disallowed.
4. There is a single right turn lane and RTOR is allowed.
5. There is a dual right turn lane and RTOR is allowed.
6. There is a single right turn lane, RTOR is disallowed, and there is a slip lane in addition to the right turn lane.

4.2.2 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 5.2, which functioned without operational challenges. This model is the *base model*.

The volume of each movement of interest (detailed in Section 5.2.4.2) was increased until the desired operational challenges were observed. The volume of interest needed to create an operational issue substantial enough that the impact created by the strategies would be distinct and measurable, but not so substantial as to render the model inoperable. The volume which achieved this goal is known as the *control volume.* The model which is identical to the base model but has one volume of interest increased to the control volume is known as the *control model.* As there were four movements of interest, there were four control models. Once the control models were created, strategies could be applied. This process is detailed in Figure 21.

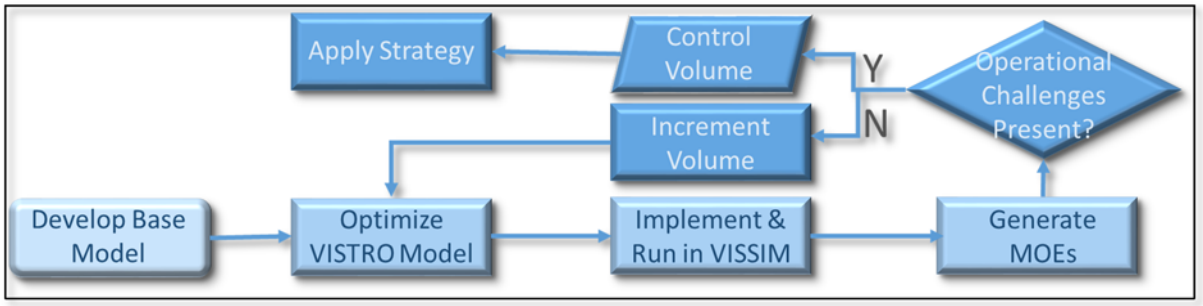


Figure 21 Flow chart of issue generation process.

Once the appropriate control volume was selected, each strategy was applied to the applicable control models. Not all strategies are effective for every movement of interest. For example, providing a dedicated off-ramp phase for the left and right turns is unlikely to improve any

operational issues created by a heavy demand for left turns onto the freeway. A chart detailing the strategies applied to each movement of interest is given in Table 3.

Table 3 Strategy Movement Combinations

	Heavy Movement			
	Through	*Right Off*	*Left Off*	*Left On*
Metering	x	x	x	
Reduced Phases	x	x	x	x
Lead/Lag	x	x	x	
Alternate Side Street	x	x	x	
Dedicated Ramp Phase		x	x	
Half Cycle	x	x	x	x
Right Turn Strategies		x		

To simulate each strategy, the strategy is applied to the control model in both Vistro and Vissim. The signal timing is optimized in Vistro and then applied to Vissim after which the model is run and MOE's are generated and analyzed.

4.3 FIELD IMPLEMENTATION

In addition to simulation efforts, the research team conducted a before-and-after study of three DDIs in North Carolina: Catawba Avenue and I-77 in Cornelius, Pioneer Crossing and I-85 in Concord, and NC 73 / Davidson Highway and I-85 in Concord. At each of these sites, the research team collected spot speed profiles, vehicles in queue, and corridor travel time data in March of 2015 as well as March of 2016. Due to the limited time frame of the project and the extensive cost and time required to implement geometric strategies, the strategies tested in the field were limited in scope to signal timing adjustments which did not require an amendment to the signal plan.

The research team met with the City of Concord (Poplar Tent and NC 73) and NCDOT Division 10 (Catawba) to determine the common challenges at each site and review any recurrent complaints from citizens. The agencies were also able to provide guidance on the most congested routes through the interchanges which should be prioritized for progression.

4.3.1 Catawba Avenue

The Catawba Avenue DDI has a three lane by two lane crossover to the west and a two lane by two lane crossover to the east. The southbound left and right turns as well as the northbound left turn from the off-ramp are two lanes, while the northbound right turn from the off-ramp as well as all four on-ramp movements are one lane. The DDI is bounded on the arterial by two four-leg signalized intersections, Torrence Chapel Road/Liverpool Parkway to the west and Highway 21 to the east, as shown in Figure 22. The image also depicts the assigned phases for each movement on the corridor.

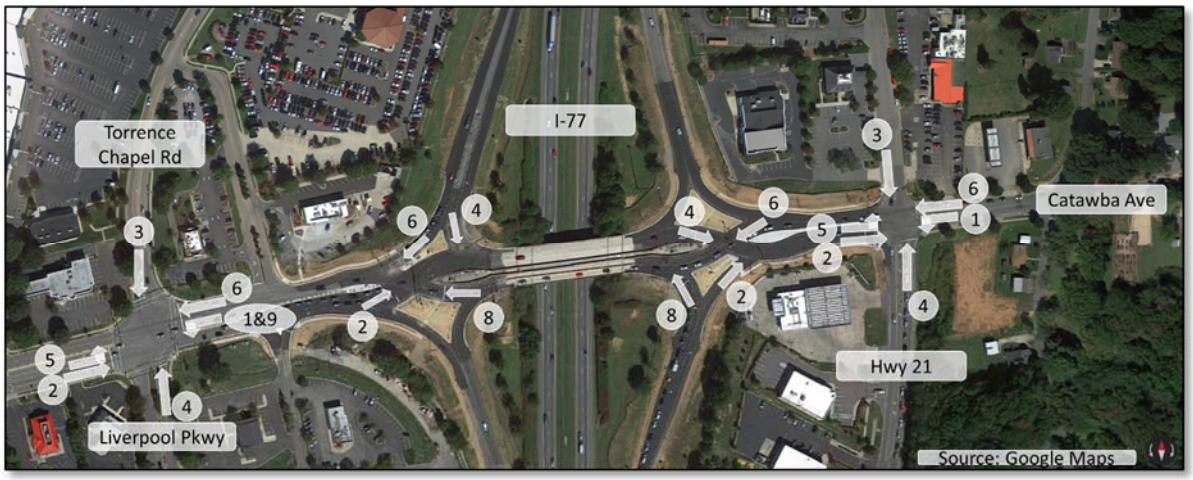


Figure 22 Catawba Avenue and I-77 DDI with Assigned Phases

Both the AM (7 am to 9 am) and PM (3:15 pm to 7 pm) peak period had the same signal timing in the before period. The cycle lengths, splits, offsets, and coordinated phases (bolded) for the system are shown in Table 4 while the ring and barrier diagrams are displayed in Table 5.

Table 4 Catawba Avenue Signal Timing

		Before				*After*			
	Phase	Liverpool	SB Ramp	NB Ramp	Hwy 21	Liverpool	SB Ramp	NB Ramp	Hwy 21
Cycle Length		140	140	140	140	120	120	120	120
Offset		24	135	82	50	60	60	20	107
Split	1	25			19	13			17
	2	**62**	**71**	**66**	**67**	**47**	**60**	**60**	**49**
	3	28			37	20			37
	4	25	69	74	17	23	60	60	17
	5	20			19	17			17
	6	**67**	**71**	**66**	**67**	**43**	**60**	**60**	**49**
	7								
	8		69	74			60	60	
	9					17			

Table 5 Catawba Avenue Ring and Barrier

	Before		After	
Liverpool	1 2 / 6 5	4 3	9 /	2 1 / 6 5	4 3
SB Ramp	2 / 6	4 / 8	2 / 6	4 / 8
NB Ramp	2 / 6	4 / 8	2 / 6	4 / 8
Hwy 21	2 1 / 5 6	3 4	2 1 / 6	4 3 / 8

In the morning peak period, there was heavy demand of vehicles headed eastbound through the west crossover and then proceeding northbound onto I-77. Similarly, many vehicles traveled northbound on Highway 21, turned westbound onto Catawba Avenue, and then proceeded northbound onto I-77.

It was observed that significant queues would develop in the afternoon peak period both at the eastbound approach of the west crossover and at the westbound approach of the east crossover. The majority of these westbound vehicles would remain on Catawba Avenue through both crossovers. At the westbound approach of Liverpool Parkway, these vehicles were joined by vehicles which turned right from southbound I-77. The heavy demand combined with a short spacing between Liverpool Parkway and the west crossover resulted in queue spillback from Liverpool into the west crossover nearly every cycle. An additional heavy path included vehicles traveling southbound from I-77, turning east onto Catawba Avenue, and then proceeding south onto Highway 21.

During the before period, the travel time between crossovers was measured at 20 seconds. From that travel time, a new cycle length was developed which was a multiple of the travel time. This allowed for better ability to provide partial progression to both mainline movements. Between March 2015 and February 2016, the Liverpool Parkway intersection was adjusted independent of this research project to include a twice per cycle left for the westbound approach. This additional phase was coded as phase 9. This timing was adopted by the research team during the after period.

The presence of two closely spaced adjacent intersections combined with multiple conflicting high demand routes and a demand at Liverpool Parkway which exceeded demand proved challenging to overcome with signal timing changes alone. NCDOT Division 10 informed the research team that initial efforts were being made to consider geometric improvements to both the Liverpool Parkway and Highway 21 intersections. Given these factors, the research team strove to provide partial progression to all major paths, requiring vehicles to stop once along the

route. Table 4 and Table 5 provide the splits, offsets, and cycle lengths as well as the ring and barrier diagrams for the system as developed by the research team and implemented in March 2016. As with the before period, one plan was developed for both the AM and PM peak periods.

The time space diagram for the plan is shown in Figure 23 with the green bands shown for the assumed travel speed. At each intersection, the bottom green/red line is the signal indication for westbound through movements and (and, at the DDI, concurrent turns from the off-ramp) while the top green/red line is the signal indication for the eastbound through movements and (and, at the DDI, concurrent turns from the off-ramp). Note that the progression speed internal to the DDI (between the two crossovers) was deliberately reduced to reflect field observations of driver behavior.

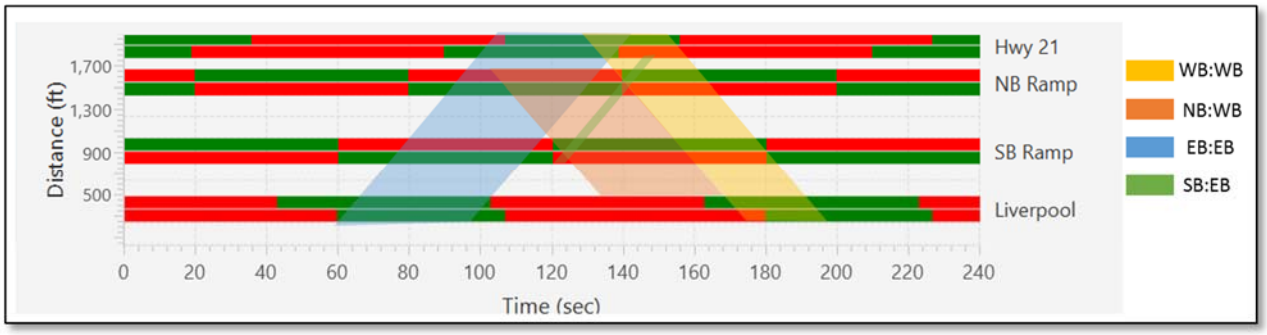


Figure 23 Catawba Avenue Time Space Diagram; After

4.3.2 Poplar Tent

The Poplar Tent Road DDI has two three lane by two lane crossovers. All movements from the off-ramps are two lanes, while all on-ramp movements are one lane. The DDI is bounded on the arterial by two left-overs from the mainline; Goodman Road and Ivey Cline Road to the west is signalized only for emergency preemption, and Pitts School Road to the east has a two-phase signal. The corridor is shown in Figure 24. The image also depicts the assigned phases for each movement on the corridor.

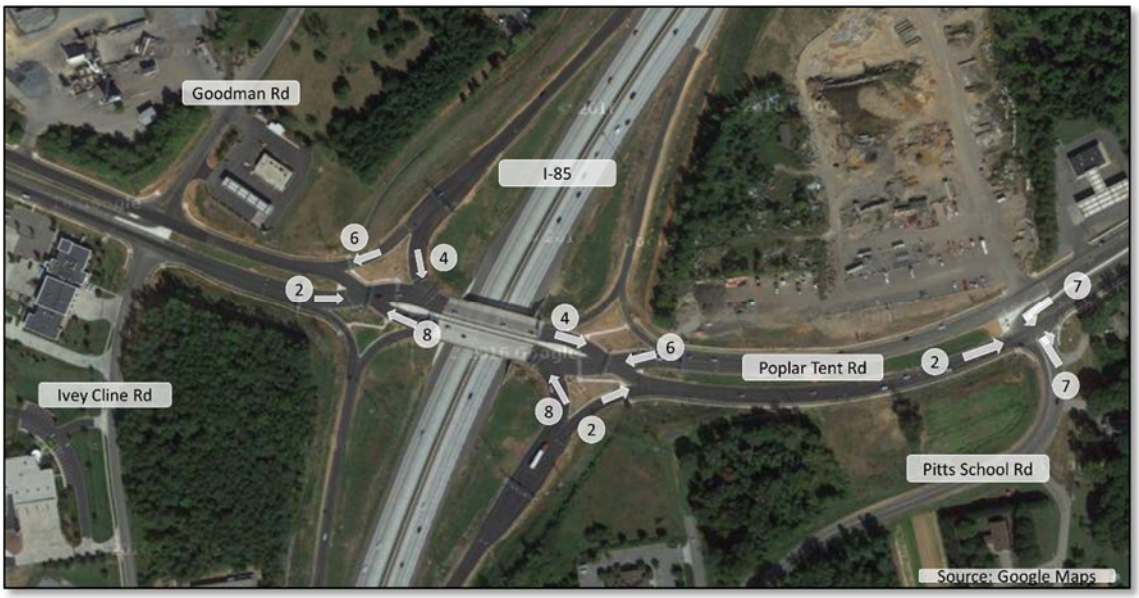


Figure 24 Poplar Tent Road and I-85 DDI with Assigned Phases

Both the AM (6:30 am to 9 am) and PM (4:15 pm to 6 pm) peak period cycle lengths, splits, offsets, and coordinated phases (bolded) for the system are shown in Table 6 while the ring and barrier diagrams are displayed in Table 7. Both the AM and PM peak period have the same ring and barrier design.

Table 6 Poplar Tent Signal Timing

		AM				*PM*			
		Before		After		Before		After	
	Phase	SB Ramp	NB Ramp	SB Ramp	NB Ramp	SB Ramp	NB Ramp	SB Ramp	NB Ramp
Cycle Length		80	80	80	80	100	100	40	80
Offset		0	47	74	3	0	50	21	2
	1								
	2	**32**	**48**	**32**	**48**	**50**	**52**	20	**42**
	3	4		4		4		4	
	4	44	32	44	32	46	48	**16**	38
Split	5								
	6	**32**	**48**	**32**	**48**	**50**	**52**	20	**42**
	7		4		4		4		4
	8	48	28	48	28	50	44	**20**	34

Table 7 Poplar Tent Ring and Barrier

	Before		After	
SB Ramp	2	3 4	2	3 4
	6	8	6	8
NB Ramp	2	4	2	4
	6	7 8	6	7 8
Pitts School	Free		Free	

The Poplar Tent DDI had a substantially lower volume to capacity ratio than the Catawba Avenue DDI which provided an opportunity to reduce the cycle length while still providing progression for all movements. In the morning period, the eastbound and westbound inbound movements at the DDI were fairly equal with roughly one third of the volume turning left onto I-85 and the remaining two thirds continuing along Poplar Tent Road. While the southbound ramp saw very low demand, the northbound left was roughly two thirds the volume of the eastbound and westbound movements. This resulted in an outbound movement at the west crossover double in volume the outbound movement at the east crossover.

In the afternoon, the demand headed eastbound along Poplar Tent was considerably heavier than the demand westbound, but again the presence of a heavy northbound to westbound left turn resulted in a higher outbound volume at the west crossover than at the east crossover.

The travel time between the crossovers as measured during the before period study was approximately twenty seconds. With an ideal cycle length being a multiple of the travel time, the 80 second cycle length in the AM signal timing plan was retained along with the splits while the offsets were adjusted slightly. The west crossover was programmed to run in actuation with the outbound through movement and southbound left being coordinated. This provided additional green time to the heavy outbound movement. The east crossover remained pre-timed as it was in the original timing.

In the afternoon peak period, the east crossover was given a cycle length of 80 seconds and was programmed to run pre-timed. To provide progression for both the westbound through and the northbound to westbound left, the west crossover was programmed with a 40 second cycle length. Again, the crossover was actuated with the outbound and southbound left coordinated. By keeping the east crossover pre-timed, it was ensured that the heavy northbound to westbound traffic would not arrive at the west crossover early.

In both the morning and afternoon peak periods, Pitts School Road retained the original signal timing plan which ran in free mode.

Table 6 and Table 7 provide the splits, offsets, and cycle lengths as well as the ring and barrier diagrams for the system as developed by the research team and implemented in March 2016. The

time space diagrams for the plans are shown in Figure 25 and Figure 26 with the green bands shown for the assumed travel speed.

Figure 25 Poplar Tent Time Space Diagram AM Peak; After

Figure 26 Poplar Tent Time Space Diagram PM Peak; After

4.3.3 NC 73 / Davidson Highway

The NC 73 DDI has two three lane by three lane crossovers. The southbound right from the off-ramp is a three lane approach while all movements from the off-ramps are two lanes, All on-ramp movements are one lane. The DDI is bounded on the arterial by a left-over from the mainline to the west and a four-leg intersection to the east. The corridor is shown in Figure 27. The image also depicts the assigned phases for each movement on the corridor.

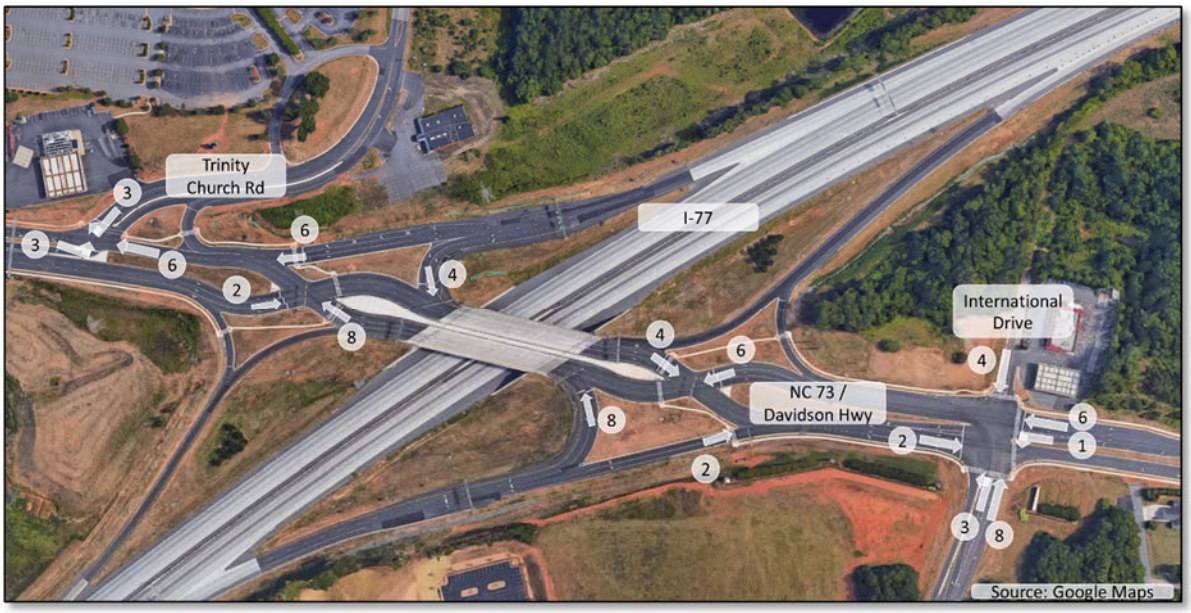


Figure 27 NC 73 and I-85 DDI with Assigned Phases

Both the AM (6:30 am to 9 am) and PM (4:15 pm to 6 pm) peak period cycle lengths, splits, offsets, and coordinated phases (bolded) for the system are shown in Table 8 while the ring and barrier diagrams are displayed in Table 9. Both the AM and PM peak period have the same ring and barrier design.

Table 8 NC 73 Signal Timing

		AM								*PM*				
		Before				After				Before				After
	Phase	Trinity	SB Ramp	NB Ramp	International	Trinity	SB Ramp	NB Ramp	International	Trinity	SB Ramp	NB Ramp	International	
Cycle Length		80	80	80	170	50	50	50	100	100	100	100	170	
Offset		39	14	0	0	0	1	1	20	48	33	91	0	
	1				25				15				25	
	2		32	48	**80**		23	28	**52**		50	52	**80**	Same as
	3	25	6		35	16	6		26	35	6		35	AM After
	4		**42**	**32**	30		**26**	**27**	17		**44**	**48**	**30**	Plan
Split	5													
	6	**55**	32	48	**105**	**39**	23	28	**67**	**65**	50	52	**105**	
	7			6				6				6		
	8		**48**	**26**	65		**32**	**21**	43		**50**	**42**	**65**	

Table 9 NC 73 Ring and Barrier

	Before		After	
Trinity		3		3
	6		6	
SB Ramp	2	3 4	2	3 4
	6	8	6	8
NB Ramp	2	4	2	4
	6	7 8	6	7 8
International	1 2	3 4	1 2	3 4
	6	8	6	8

The NC 73 DDI had a similar volume to capacity ratio as the Poplar Tent DDI which provided an opportunity to reduce the cycle length while still providing progression for all movements. In both the morning and afternoon peak periods, the eastbound and westbound movements along NC 73 were balanced. In the eastbound direction, roughly half of the vehicles originating on NC 73 turned northbound onto I-85 while half continued on NC 73. In the westbound direction, the split was closer to two-thirds remaining on the arterial. This, in addition to a heavier northbound to westbound left turn relative to the southbound to eastbound left turn, resulted in a heavier

outbound movement at the west crossover. Given the similar volume patterns in both peak periods, one signal timing plan was developed for both periods.

The travel time between the crossovers as measured during the before period study was approximately 25 seconds. With an ideal cycle length being a multiple of the travel time, a 50 second cycle length was used for both of the crossovers as well as at Trinity Church Road. Because International Drive required more than two phases, a 100 second cycle length was selected to provide adequate splits for all phases. This double cycle length allowed for coordination of the eastbound vehicles in every other cycle.

Table 8 and Table 9 provide the splits, offsets, and cycle lengths as well as the ring and barrier diagrams for the system as developed by the research team and implemented in March 2016. The time space diagrams for the plans are shown in Figure 28 with the green bands shown using the average travel speed in the individual segments.

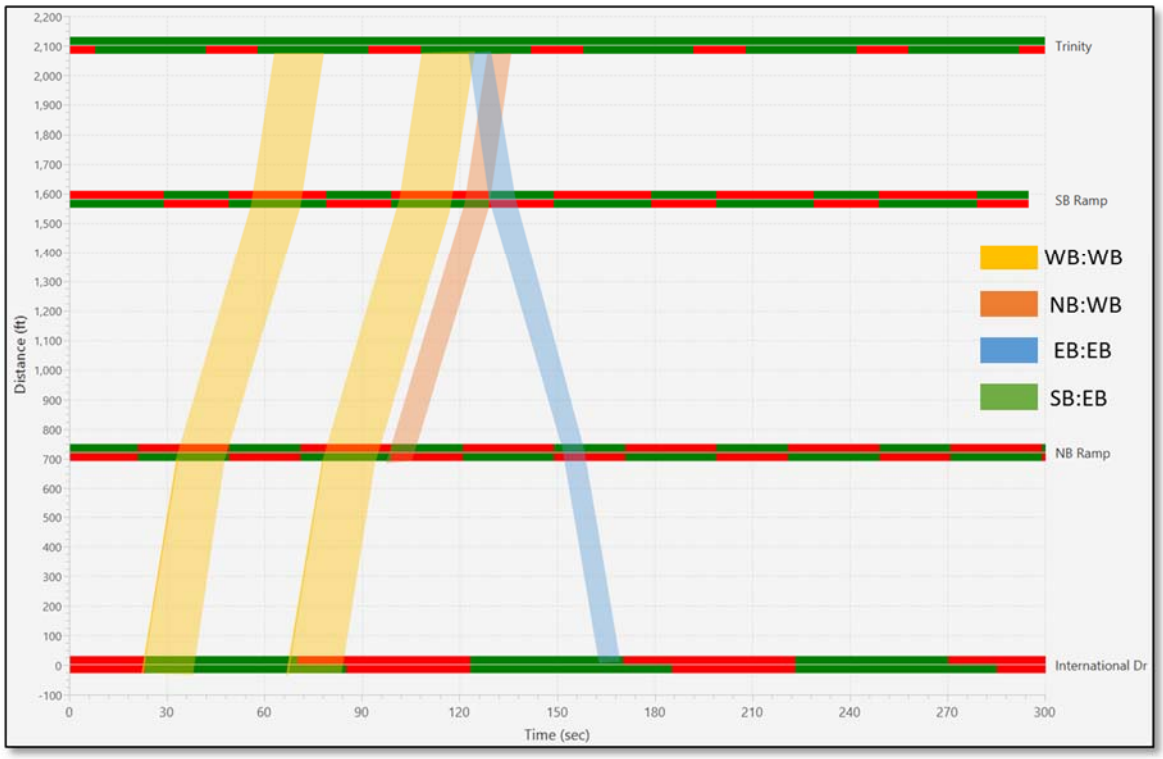


Figure 28 NC 73 Time Space Diagram; After

5 EXPERIMENT DESIGN

5.1 INTRODUCTION

This chapter presents the selection, calibration, and validation of simulation tools as well as the sequence of the simulation experiment. VISTRO was used to optimize split times and offsets with those values being used in VISSIM to simulate the corridor operations.

Software which optimizes signal timing, such as VISTRO, uses proprietary optimization equations. From those timing plans, deterministic equations found in the HCM are used to generate common measures of effectiveness such as delay, number of stops, queue length, etc.

Microscopic traffic simulation software relies on a series of algorithms with input values determined by the user and adds one or more random variables to model the behavior of each vehicle. This process is therefore stochastic in nature, so for each scenario multiple trials are necessary to draw statistically sound conclusions. These algorithms include: car-following, lane-changing, gap-acceptance, routing, response to signal indication, etc.

This chapter also details the data collection process for field implementation, including spot speed profiles, queue counts, and corridor travel times.

5.2 SIMULATION

5.2.1 Simulation Objective

The simulation experiment focused on the following objective:

Given one or more operational challenges at a DDI with closely spaced adjacent intersections, determine which strategies should be given further consideration, either in isolation or in combination.

5.2.2 Limitations

This chapter does not attempt to simulate strategies under different DDI geometries. It is also outside the scope of this chapter to provide definitive results as to a strategy's appropriateness to every design or driver population. The results should be used as a guide for design professionals as to the expectation of potential challenges given a specific volume scenario as well a starting list for potential strategies.

5.2.3 *Tool Selection*

VISSIM was selected because the positive experience the team had developing and simulating DDIs in the environment during prior research. The link-connector method of model building (as opposed to node based) contributed in large part to this. VISTRO was selected for signal optimization due to the compatibility of the ring-barrier signal controller design and offset designation of the products. Both are produced by the same parent company, facilitating transfer of inputs and settings from one tool to the other.

5.2.4 Modeling Considerations

This section describes the key input decisions which impact the ability of the model to reflect the expected real-world conditions. Many of these decisions were made during the calibration and

validation process of a previous research project. This project makes modifications to a model of the DDI at National Ave and US-60 in Springfield, Missouri. The only adjustments to the model were geometric in nature, so the balance of the section is a summary of the previous modeling efforts.

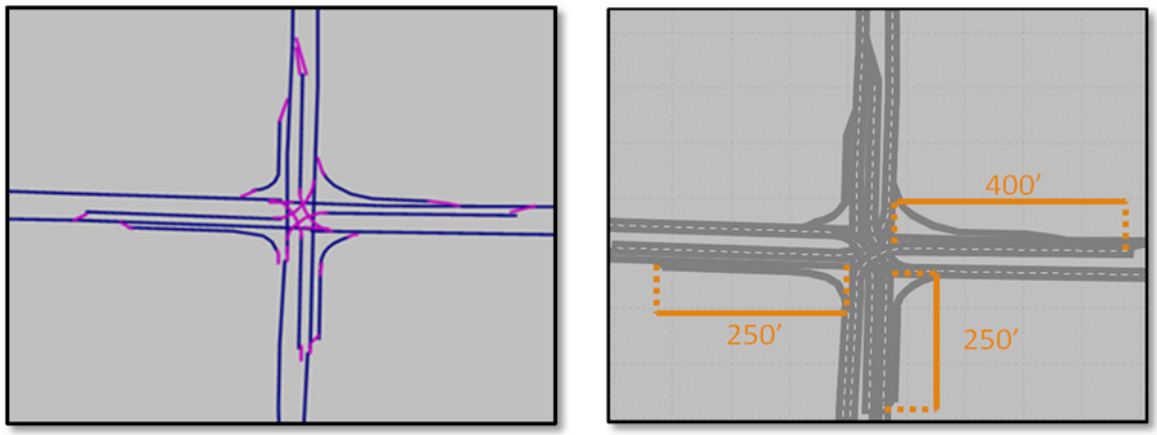
5.2.4.1 Geometrics

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.

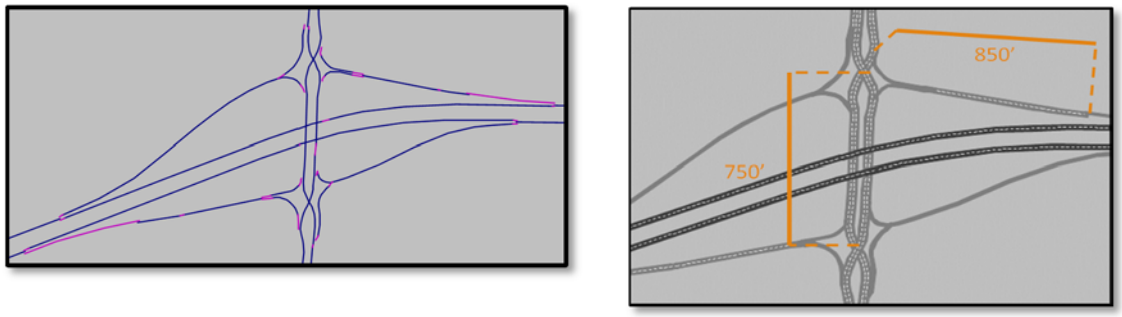
The DDI at National Avenue and US-60 in Springfield, Missouri was used as the base geometry for the model with some modifications to ensure the system was symmetric. This site was used as the base model because a calibrated and validated model was available. Additionally, the existing conditions included 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.

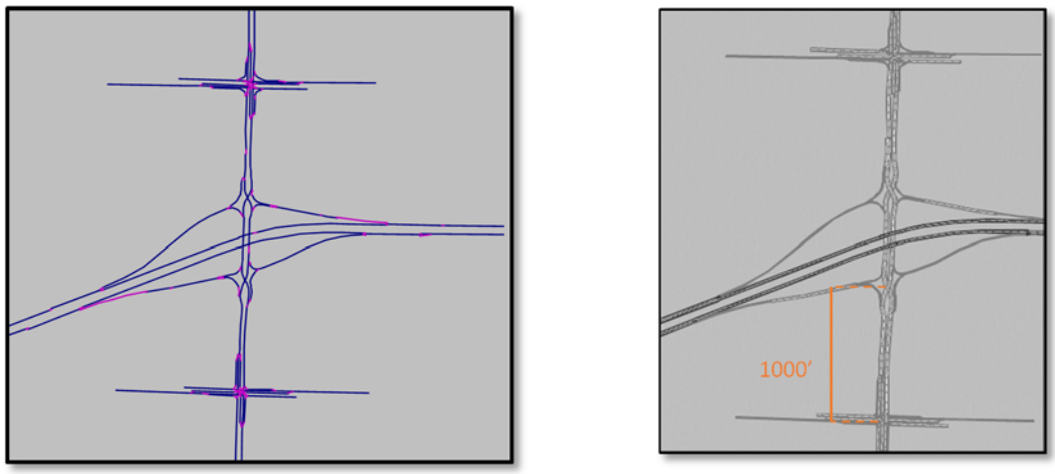
At the adjacent intersections, all left turning movements were coded with exclusive dual turn lanes, while right turning movements were exclusive single lanes. On the mainline, through movements had two exclusive lanes, while one exclusive lane was present on the minor street. The three travel lanes 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 29 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.



(a)



(b)



(c)

Figure 29 VISSIM Geometric Model of the DDI Corridor with a) the adjacent intersection (b) the DDI and (c) the entire corridor.

5.2.4.2 Volume Input and Routing

The team tested a total of four volume combinations, with each originating from the same base volume. At the DDI, the left and right turns from the freeway, as well as onto the freeway, have 275 vehicles per hour. There are 840 vehicles that drive north through the DDI and 560 vehicles per hour driving south, representing a 60/40 directional split. The base volumes through the DDIs are shown in Figure 30.

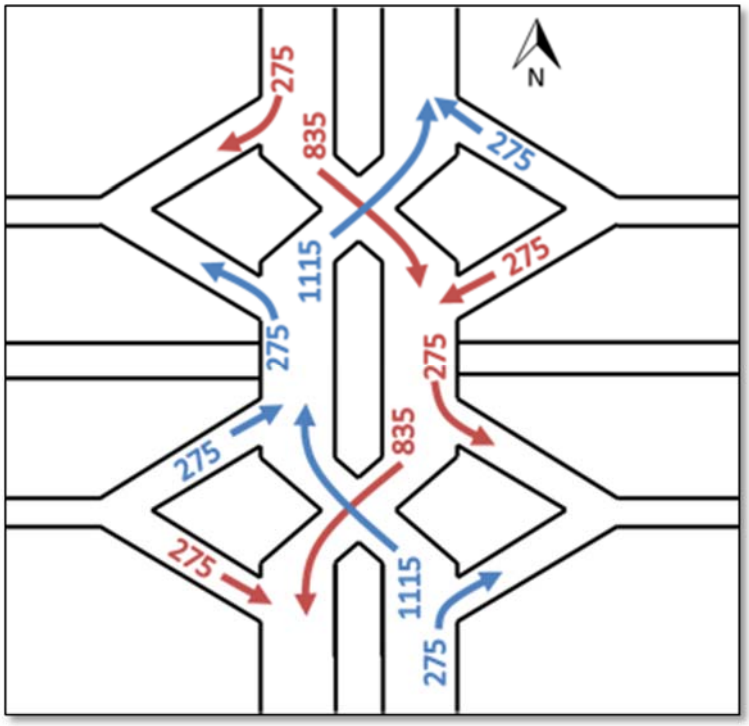


Figure 30 Base Volumes through the DDI

To generate the inbound flow to the DDI, it is assumed that 20% is contributed by the right turn and 20% by the left turn from the side streets. The remaining 60% comes from the through movement along the mainline. Figure 31 shows the origination of vehicles from the northern adjacent intersection heading south into the DDI. These source volumes are the same at the southern adjacent intersection heading south into the DDI.

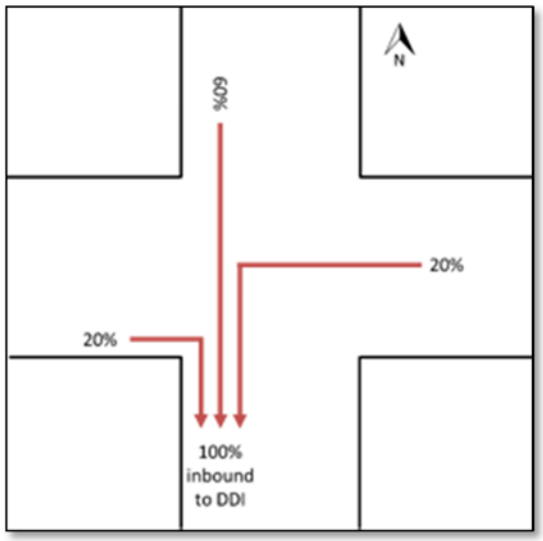


Figure 31 Origination of Inbound flow to DDI as shown at the adjacent southern intersection

For vehicle volumes coming out of the DDI, it is assumed that 15% of the mainline movement turns left, 15% turns right, and the remaining continue straight through the intersection. At the side streets, there is a 48/20/32 split for the north, through, and south movements which is in line with the 60/40 directional split along the mainline. Figure 32 summarizes the volume splits by approach for the northern intersection. The volumes splits are identical at the southern adjacent intersection.

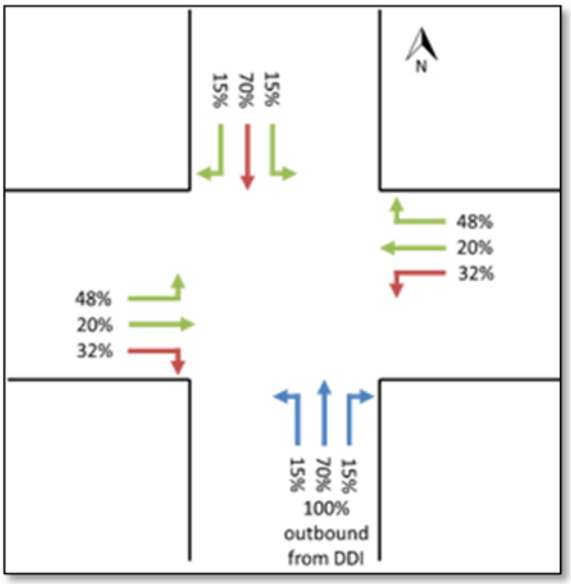


Figure 32 Volume splits by approach

The inbound volumes must be calculated first according to Figure 31. Once the inbound volumes (shown in red above) have been generated, the green volumes can be set according to Figure 32.

The three outbound volumes are independent of all other splits and can be set using the splits (shown above in blue).

From this base volume, four heavy movements were developed in an attempt to replicate the potential operational challenges at the DDI identified by prior research: queue spillback at the downstream adjacent intersection as well as at the off ramp, demand starvation and poor progression in the non-peak direction. The four volume levels, shown in Figure 33, are as follows:

- Heavy through movement through the DDI – 1380/920 N/S split of through vehicles per hour instead of 840/560 vehicles per hour
- Heavy westbound to northbound right turning movement from the off ramp – 900 vehicles per hour instead of 275 vehicles per hour
- Heavy southbound to eastbound left turning movement onto the on ramp – 900 vehicles per hour instead of 275 vehicles per hour
- Heavy eastbound to northbound left turning movement from the off ramp – 900 vehicles per hour instead of 275 vehicles per hour

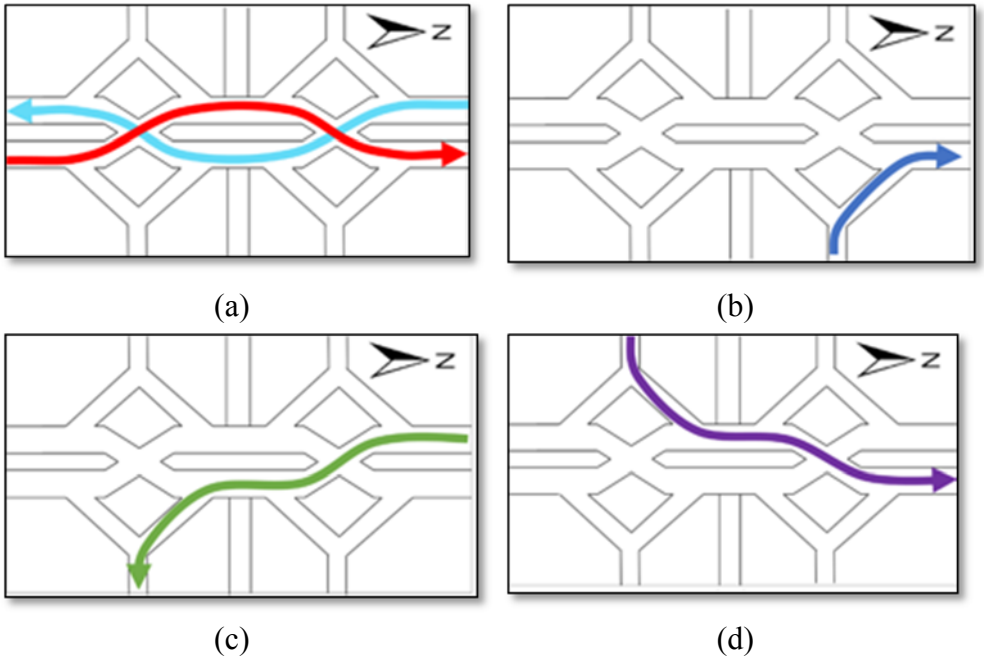


Figure 33 Heavy Movements: a) through, b) right turn off, c) left turn on, and d) left turn off

To arrive at the heavy volume for each movement, multiple scenarios were created with each scenario increasing the volume of interest by 100 vehicles per hour. For the turning movements, volumes were varied from 400 vehicles per hour to 1000 vehicles per hour. The heavy through movement varied from a total of 1400 vehicles per hour (total of north- and south bound) to 2200 vehicles per hour. Each scenario was run 15 times for 4500 seconds with data being collected between seconds 900 to 4500. The four measures of effectiveness were evaluated over all scenarios to find a volume where the issues were strong enough to serve as a control for the

future strategy testing. Ideally, the volume would create an operational issue significant enough that the impact created by the strategies would be distinct and measurable, but not so significant as to render the model inoperable.

In all instances, the heavy movement travels through the northern adjacent intersection. This decision was made for ease of comparison.

5.2.4.3 Speed Control

Prior research efforts to study the National Avenue DDI on which the model was based included a speed study at 14 points of the DII. These speeds were used to calibrate the model. Consideration was given to the speed at which vehicle turned to or from the on- or off-ramps as well as the reverse curvature through the crossover. The speeds of vehicles traveling between the crossover was also measured. The freeflow speed measurements were consistently lower than the designated speed limit of the interchange.

5.2.4.4 Unsignalized Movements

The left turns from the DDI onto the on-ramp were unsignalized. Signalization of these movements is most often used to ensure protected pedestrian movements when the pedestrian access is located outside of the roadway (as opposed to in the median). Additionally, right turns at the adjacent intersections from the side street inbound to the DDI were unsignalized as the movement had a free-flowing turn onto the added lane as can be seen in Figure 29 (a).

5.2.4.5 Signalized Movements

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 intersections and the need to meet the minimum splits required to serve demand at the four critical phases. By fixing the cycle length, a more direct comparison of the impact of the strategies at the crossover could be made. The splits, offsets, and left turn leading or lagging were optimized in VISTRO as explained in Section 4.2.2. 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 *(19)*.

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 *(35)*.

The DDI crossovers were pretimed and run on two controllers. Figure 34 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 strategies would be equally impacted.

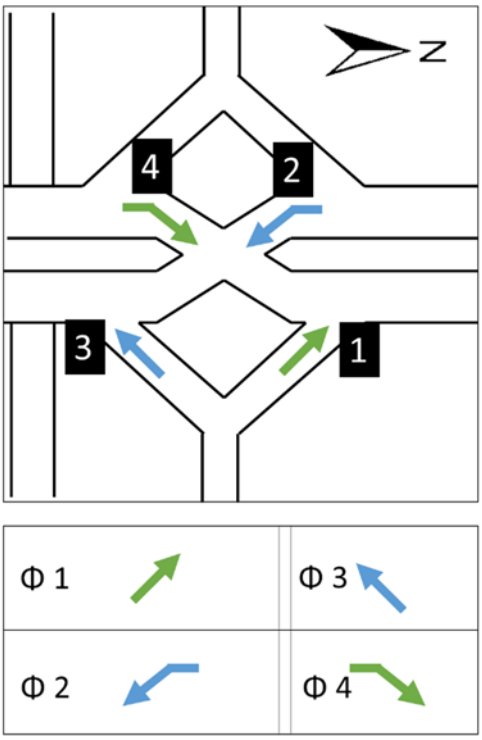


Figure 34 Phasing scheme for northern crossover.

5.2.4.6 Pedestrians and Bicyclists

While pedestrians and bicyclists were not directly modeled, the signal timing optimization did account for the presence of pedestrians at the intersections. That is, if a phase normally provided green time to both vehicles and pedestrians, the minimum green time met or exceeded the minimum green time required for a pedestrian.

5.2.4.7 Performance Data

Four measures of effectiveness were selected to monitor the impact various operational strategies had on the challenges described above: (1) queue lengths, (2) green usage, (3) total delay, and (4) number of stops. All measurements were taken either for 10 movements or over 10 paths as described below.

The maximum queue length was recorded for each 120 second increment. (One hundred twenty seconds was the cycle length for all signals except for the strategies of half cycling and alternate side street phasing as described below). The queues were recorded for ten movements as shown in Figure 35.

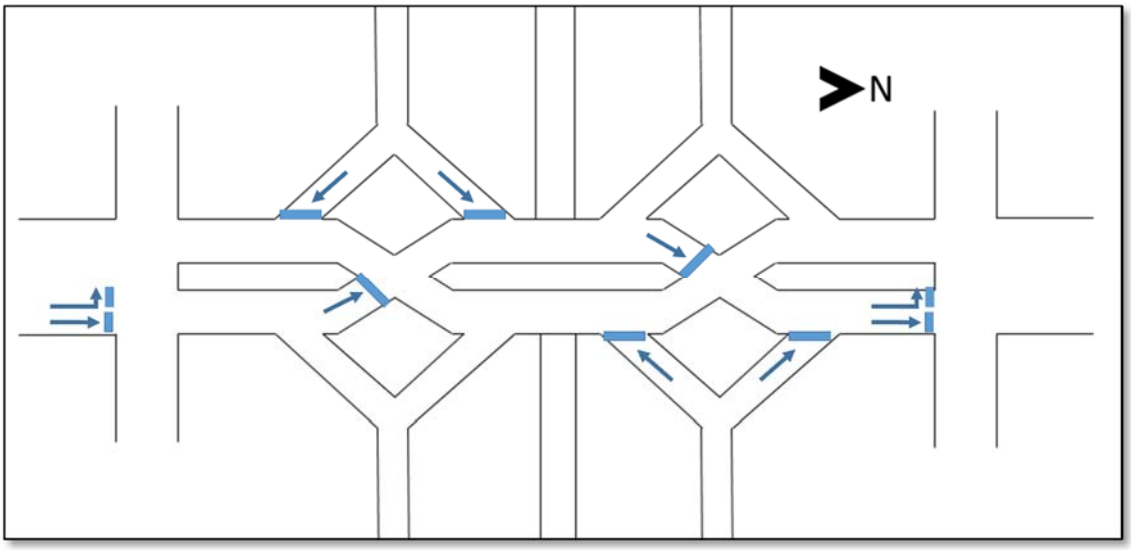


Figure 35 Locations of queue length collection points along the corridor.

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 their 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.

The same ten movements were evaluated for the fraction of green time usage at the signals. The green usage was defined as the amount of time traffic was moving through the intersection, divided by the total available green time for the movement. The opposite of this measurement is commonly referred to as wasted green time. To calculate the green usage, a small presence detector, approximately 4 feet in length, was placed just downstream of the stop bar. Every second of simulation, VISSIM reported a) if the movement of interest had a green signal and b) if there was a vehicle over the detector. Using post processing code, the amount of time for each cycle during which the signal was both green and a vehicle was present was recorded.

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 path that complete the entire path during the simulation are considered in the computation. Vissim averages the total travel time for all vehicles that have completed the path over a run and compares that to the ideal travel time to determine the total delay. The ideal travel time is calculated by dividing the distance of the path by the expected vehicle travel speed (as defined by the user). If two or more expected speeds are used on a single path, VISSIM calculates the time to travel over the path controlled by the first speed and adds that to the time to travel over the path controlled by the second speed, etc.

Travel time measurements were taken for 10 paths as shown in Figure 36.

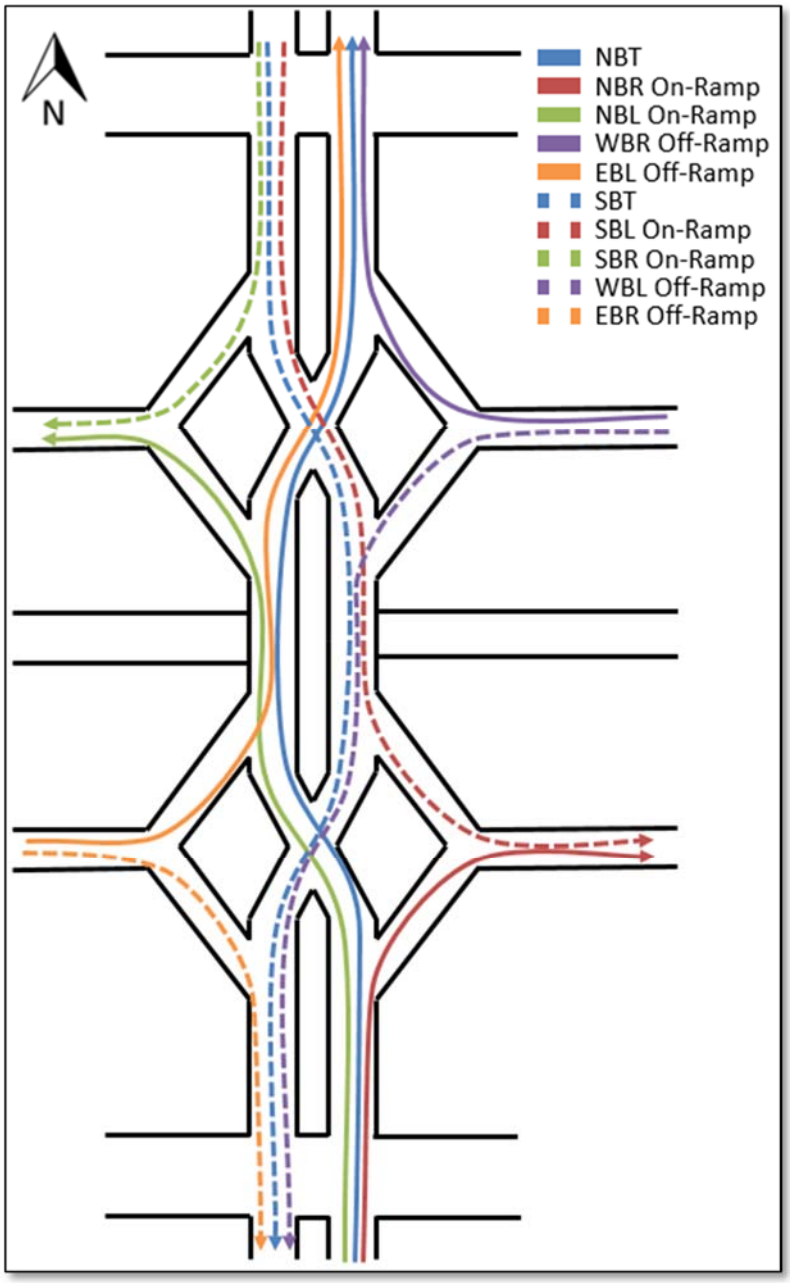


Figure 36 OD Paths for Stops and Delays

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 stop bar and the end bar. If a vehicle stops for 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 is not considered in the total stop calculation.

The ten OD paths encounter between one and four traffic signals, so looking exclusively at the total number of stops does not give a clear sense of the operations. For example, it may be appropriate for a vehicle traveling through all four traffic signals to have to stop twice, but less appropriate if that vehicle only traverses one traffic signal. A measure of stops per intersection, however, would be equally inappropriate as one stop over one intersection may be expected by a vehicle operator, but four stops over four intersections in the same corridor would likely be unacceptable. To standardize the measurement, a stop severity index (SSI) was developed as given in Equation 1

$$SSI = \frac{\#\ of\ stops^2}{\#\ of\ intersections\ traversed}. \tag{1}$$

This then means that one stop over one intersection would be as severe as two stops over four intersections. The SSI should be considered increasingly important as the percentage of heavy vehicles increase due to the lower acceleration abilities of those vehicles.

To ensure a valid sample size, each strategy or control was simulated 15 times for 3600 measureable seconds plus a 900 second warm up period.

5.2.5 Calibration and Validation Approach

The National Avenue DDI Vissim model was calibrated from and validated against field-collected data. After the validation was complete, adjustments were made to the model rendering it more symmetric and generic. These adjustments are detailed in Section 5.2.4.1. Key calibration factors included:

1. Origin-Destination 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
4. Field-implemented signal timing schemes obtained from the field controller settings

Turn movement volumes were collected at the DDI as well as the adjacent intersections. To generate Origin-Destination 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 is of particular importance with DDIs because of the documentation of prepositioning of vehicles well ahead of the inbound crossover, particularly for vehicles turning left from the arterial onto the freeway *(34)*.

During field evaluations, it was observed that the speeds of vehicles through the DDI were below the free-flow speed of vehicles on the adjacent tangential portion of the arterial. Therefore, 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
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 *(36)*.

5.3 FIELD IMPLEMNTATION

5.3.1 Spot Speed Profiles

Using laser guns, the research team collected speed profiles at 14 locations with the DDI as shown in Figure 37. The measurements were collected on vehicles which were observed in a free flow condition and were neither departing a stopped status at an upstream signal nor approaching a red indication at the downstream signal. This data were used to assist in calibration of the three DDI models which were later used to test strategies prior to implementation in the field.

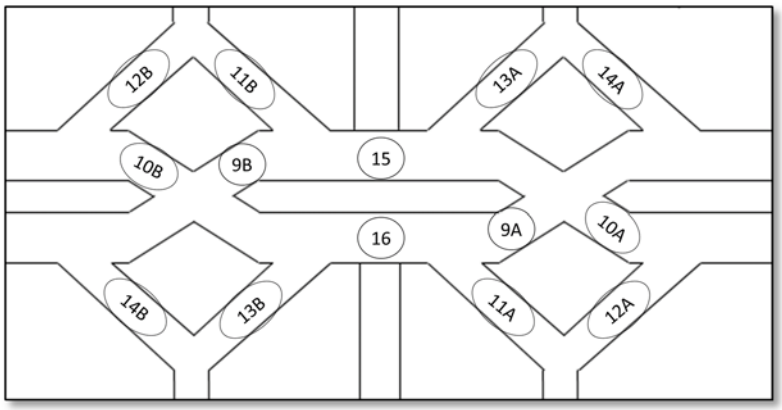


Figure 37 Location of Spot Speed Profiles Collected at the DDI

5.3.2 Vehicles in Queue

During the peak period signal plans, the maximum count of vehicles per-lane, per-cycle was recorded for each signalized approach at the DDI. In scenarios where the maximum queue length extended beyond the upstream adjacent intersection, the tally was recorded as "queue spillback". The measurements provided a sense of the potential for queue spillback as well as a general indicator of congestion levels. This data was also used to assist in calibration of the three DDI models prior to implementation of strategies for field testing.

5.3.3 Corridor Travel Time

Corridor travel time measurements were collected using in-vehicle GPS units during the same peak period of which queue lengths were measured. Both through movements across the interchange as well as all four left turns were collected. Routes which originated or terminated on the freeway were bounded by the gore of the on- or off-ramp. Routes which originated on the arterial began upstream of the first intersection prior to entering the DDI. Routes which terminated on the arterial extended immediately downstream of the first intersection downstream of the DDI.

GPS data were processed using TravTime 2.0, a GeoStats software package. Routes were created in the software and average travel time, average number of stops, and average stop delay for each route was calculated. This data were used to assist in calibration of the three DDI models as well as to compare the impacts of the adjusted signal timings.

6 RESULTS AND DISCUSSION

6.1 INTRODUCTION

This section presents the condensed results from the simulation and field implementation effort for this project.

For the simulation study, tables are provided which display the impact each strategy has on every measure of effectiveness over the four heavy movements. For complete charts, a quick reference guide is provided in Appendix B.

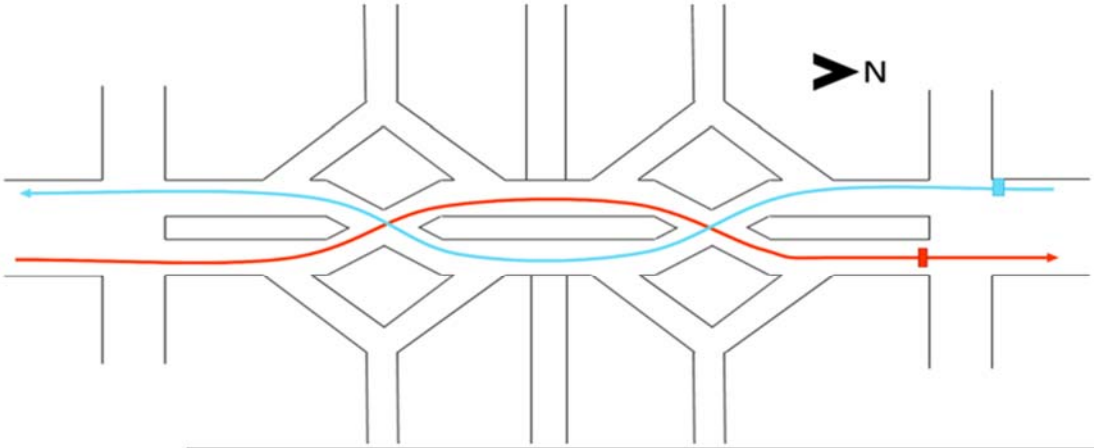
For the field implementation, the measures of effectiveness outlined in Section 5.3 are reported along with time-space diagrams from the travel time study. Concluding the chapter is a summary of the cost, user disruption, and safety impacts of each strategy.

6.2 SIMULATION

6.2.1 Heavy Through

Figure 38 provides the impact factor in each measure of effectiveness for heavy volumes *northbound and southbound along the arterial*. The impact factor is calculated by dividing the strategy measure of effectiveness by the control measure of effectiveness. A green color indicates the measure improved with the strategy. The deeper the color, the stronger the effect. Each MOE indicates if it was measured over the path (indicated by a line in the image) or at a movement (indicated by a bar in the image). The table also provides the value of each MOE under the control model. Queues were not measured in the southbound direction.

It can be seen that the most impactful strategies are those which reduce the number of phases at the downstream adjacent intersection (MUT, Rotated MUT, Superstreet, and Stretched Superstreet). By reducing the number of phases which must receive green time in a cycle, a higher percentage of time can be given to the mainline movements resulting in increased capacity.



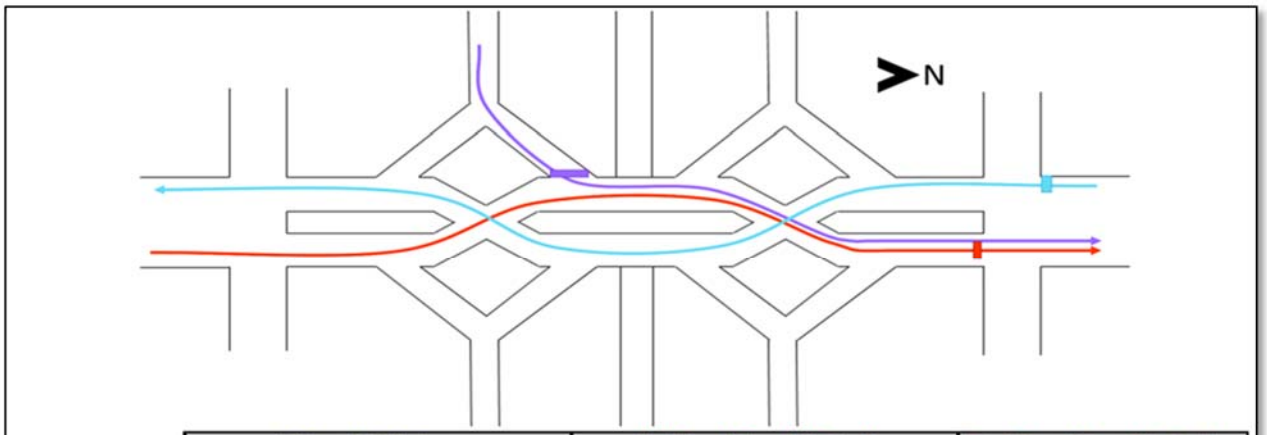
	North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	181 seconds	SSI = 3.5	56%	822 ft	119 seconds	SSI = 2.1	45%
Alternating Side Street	0.81	0.67	0.69	0.89	1.01	0.91	0.64
Half Cycle	0.51	0.70	1.01	0.75	0.70	0.55	0.92
Metered	0.99	0.96	1.00	1.01	0.97	0.97	1.00
MUT	0.91	0.77	1.20	0.55	1.01	0.71	0.98
Rotated MUT	0.83	0.68	0.95	0.60	1.14	0.91	0.90
Stretched Superstreet	0.65	0.34	-	0.54	1.03	0.71	-
Superstreet	0.62	0.42	-	0.56	0.97	0.73	-
Twice Per Cycle Left	1.09	1.22	1.01	1.08	0.96	0.96	0.99

Figure 38 Impact Factor of Each Strategy on the Measures of Effectiveness over the Heavy Movement Path (Northbound Through the Corridor) and the Southbound Corridor Path.

6.2.2 Heavy Left Off

Figure 39 provides the impact factor in each measure of effectiveness for heavy volumes from *eastbound lefts at the off-ramp as well as northbound and southbound along the arterial*.

The measures of effectiveness for the first two routes – the left turn off of the freeway and the northbound corridor route – tend to improve with most strategies. Not surprisingly, some strategies are negatively impacted for the southbound route. The conflicting mainline movements at the DDI crossover result in the prioritization of one movement over the other. Since the heavy movement is in the northbound direction, it is prioritized. This is particularly seen in the dedicated off-ramp phase strategy. The additional green time for the off-ramp provides more capacity resulting in more vehicles entering the DDI and heading northbound at the northern crossover. That movement, therefore, requires more green time. As the crossover is a two-phase system, more green time for the northbound movement results in a direct loss of green time for the southbound movement.



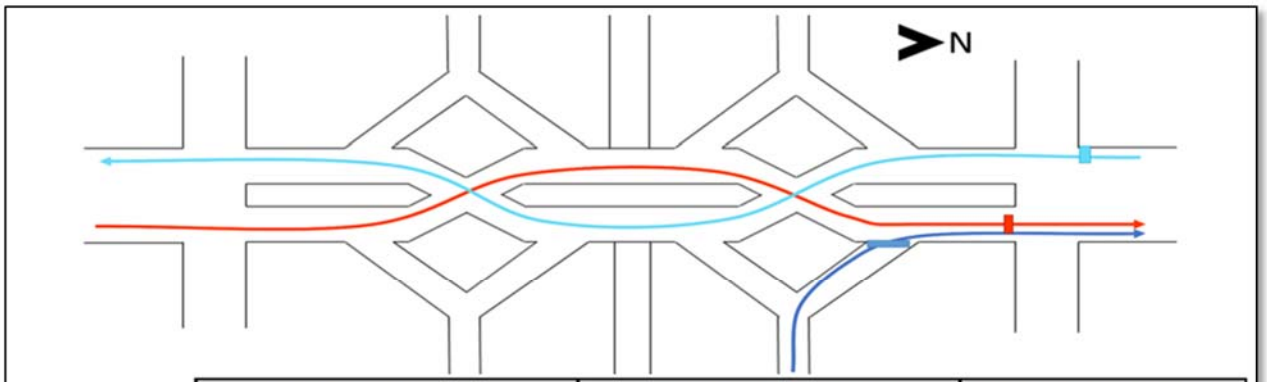
	Left Turn Off of the Freeway				North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	123 seconds	SSI = 2.4	45%	762 ft	154 seconds	SSI = 2.7	36%	882 ft	130 seconds	SSI = 2.0	37%
Alternating Side Street	0.58	0.43	1.14	0.96	0.96	0.77	0.59	0.50	0.97	0.86	0.62
Ded. Off Ramp Phase	0.87	0.75	1.05	0.65	1.12	1.06	0.98	0.77	1.14	1.61	0.69
Half Cycle	0.62	0.55	1.05	0.73	0.59	0.65	0.93	0.81	0.65	0.65	0.90
Metered	1.03	1.09	1.02	1.11	1.00	0.98	1.00	0.99	0.99	0.98	0.99
MUT	0.79	0.68	1.04	1.01	0.83	0.56	1.10	0.50	1.00	0.69	0.96
Rotated MUT	0.48	0.28	0.99	0.97	0.61	0.44	0.94	0.36	0.92	0.49	0.80
Stretched Superstreet	0.56	0.26	1.00	0.99	0.72	0.40	-	0.77	0.89	0.87	-
Superstreet	0.48	0.29	1.12	0.98	0.60	0.29	-	0.46	0.84	0.76	-
Twice Per Cycle Left	1.04	1.11	1.05	1.12	1.00	1.00	0.98	1.01	0.97	1.03	0.87

Figure 39 Impact Factor of Each Strategy on the Measures of Effectiveness over the Heavy Movement Route (Left Turn Exiting the Freeway) and the Corridor Route.

6.2.3 Heavy Right Off

Figure 40 provides the impact factor in each measure of effectiveness for heavy volumes from *westbound right turns at the off-ramp as well as northbound and southbound along the arterial*.

For the heavy right off volume scenario, a cluster of right turn strategies were tested (right turn on red permitted, dual turn lanes, slip lane, acceleration lane, and right turn on red plus dual right turn lanes). These strategies all addressed the right turn directly with only two (slip and acceleration lane) impacting the downstream intersection. As such, no significant improvement in queue lengths at the downstream adjacent intersection were measured. The increased capacity at the right turn resulted in more vehicles arriving at the downstream intersection thereby generally increasing queue lengths. The most effective strategies for the corridor, then, were those which addressed capacity issues at the downstream intersection.



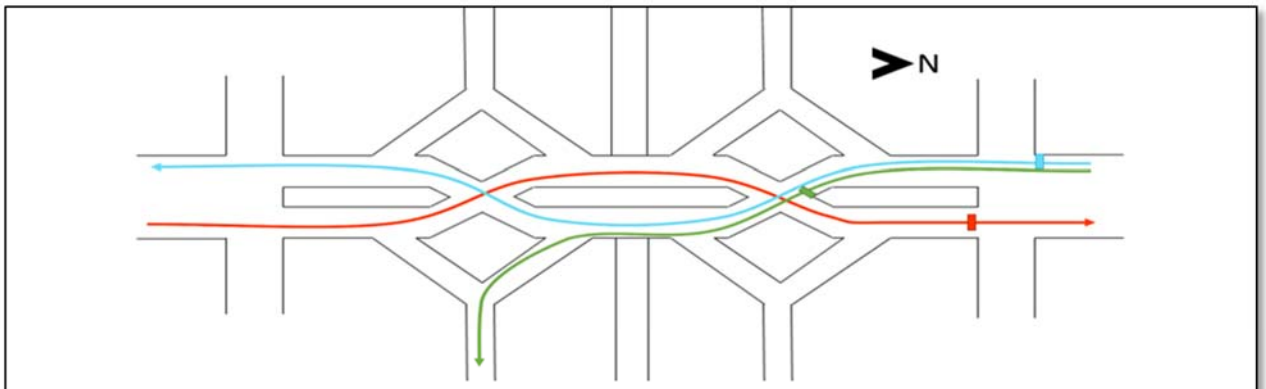
	Right Turn Off of the Freeway				North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	86 seconds	SSI = 1.5	53%	961 ft	123 seconds	SSI = 2.3	40%	759 ft	111 seconds	SSI = 1.9	32%
Accel. Lane	0.32	0.14	0.54	-	0.59	0.34	1.13	0.39	1.00	0.96	1.12
Alternating Side Street	0.80	0.80	0.96	0.92	0.86	0.69	1.35	0.69	1.21	1.15	1.08
Ded. Off Ramp Phase	0.95	0.86	0.97	0.93	1.17	1.24	1.44	1.03	1.12	1.09	0.99
Dual RT Lanes	0.96	1.09	1.10	0.46	0.75	0.66	1.44	1.03	0.99	0.98	0.98
Dual RT Lanes & RTOR	0.81	1.46	1.01	0.26	0.75	0.66	1.45	1.07	0.99	0.98	0.67
Half Cycle	0.46	0.39	1.01	0.14	0.73	0.59	1.50	0.96	0.96	1.06	1.04
Metered	0.96	0.89	0.98	1.00	0.91	0.91	1.46	0.95	1.01	1.01	1.00
MUT	0.99	0.82	1.03	1.06	1.02	0.78	1.68	0.58	1.05	0.79	1.10
Right Turn On Red	0.91	0.92	1.01	0.80	0.91	0.91	1.46	0.97	1.00	1.01	1.01
Rotated MUT	0.57	0.40	0.84	0.72	0.80	0.81	1.47	0.60	1.00	0.95	0.91
Slip Lane	0.63	0.44	0.82	0.57	1.21	1.41	1.43	0.76	1.24	1.11	1.04
Stretched Superstreet	0.63	0.36	0.86	0.76	1.01	0.84	-	0.90	1.08	0.79	-
Superstreet	0.55	0.35	1.56	0.74	0.55	0.25	-	0.38	1.08	0.68	-
Twice Per Cycle Left	1.60	2.64	0.94	0.28	1.56	1.66	1.41	1.14	1.21	1.20	0.97

Figure 40 Impact Factor of Each Strategy on the Measures of Effectiveness over the Heavy Movement Path (Westbound to Northbound Right Turn Exiting the Freeway) and the Corridor Paths.

6.2.4 Heavy Left On

Figure 41 provides the impact factor in each measure of effectiveness for heavy volumes from *southbound lefts at the on-ramp as well as northbound and southbound along the arterial*.

While only a limited number of strategies addressed issues relating to the heavy left turn onto the freeway, those strategies tested did result in generally positive improvements for all paths and movements. Under all strategies, the delay and stops decreased more for the left turn route than for the southbound corridor route. This is expected. While the path for the two routes is identical from the northern adjacent intersection through the northern crossover, the southbound route must continue through two signals – the southern crossover and southern adjacent intersection – which were generally not impacted by the strategies.



	Left Turn Onto the Freway			North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	166 seconds	SSI = 3.4	30%	166 seconds	SSI = 3.5	0.5	473 ft	183 seconds	SSI = 3.3	0.58
Half Cycle	0.29	0.20	0.91	0.75	0.54	0.98	0.86	0.53	0.59	0.91
MUT	0.70	0.48	-	0.65	0.51	-	0.94	0.93	0.89	-
Rotated MUT	0.48	0.29	0.93	0.57	0.44	0.88	0.81	0.71	0.75	0.93
Stretched Superstreet	0.28	0.10	-	0.73	0.33	-	1.08	0.65	0.50	-
Superstreet	0.81	0.68	-	0.41	0.17	-	0.32	1.21	1.29	-

Figure 41 Impact Factor of Each Strategy on the Measures of Effectiveness over the Heavy Movement Path (Southbound to Eastbound Entering the Freeway) and the Corridor Paths.

6.2.5 Summary

Those strategies which provided the greatest improvements to the measures of effectiveness for the movement of interest as well as the corridor paths tend to address the capacity challenges at the downstream adjacent intersection. With two critical phases, the DDI has less lost time than the adjacent intersection. Additionally, for the signal timing scheme tested, the DDI provides green time to an outbound movement continuously (with the exception of the lost time); either the outbound crossover movement has the right-of-way, or the right turn from the off-ramp has the right-of-way. Because of the presence of conflicting side-street movements, the downstream adjacent intersection is restricted in the green time provided to the outbound movement. Therefore, strategies which increase the capacity of the DDI but ignore the downstream adjacent intersection are less effective at treating the operational issues of the corridor.

6.3 FIELD IMPLEMENTATION

6.3.1 Catawba Avenue

In the after period data collection of the Catawba Avenue DDI, I-77 experienced severe congestion in the northbound direction. This resulted in queue spillback from the freeway, onto the northbound on-ramp, and into the DDI. Therefore, no AM period data could be analyzed. The results below are for PM data collection only.

Table 10 presents the change in travel time, stops, and stopped time over the interchange from the before period to the after period. A positive value means the measure was reduced from the before period to the after period. The volumes reported are those gathered from a November 2014 turn count provided by NCDOT. The results show a reduction in travel time for the southbound to eastbound left turn and the northbound to westbound left turn. Other movements

show minor increases in travel time, but the differences are not practically significant (virtually the same in terms of driver experience). The interchange average travel time, stops, and stop time per vehicle are practically insignificant. Table 11 presents the change in travel time, stops, and stopped time over the corridor from the before period to the after period. Because a full origin-destination study of the corridor has not been conducted, volume weighted results over the corridor cannot be provided. The interchange and corridor start and end points are shown in Figure 42.

Table 10 Catawba Avenue Change in Performance Measures Over the Interchange (Before - After)

	PM			
	Vol	*TT (min/veh)*	*Stops (per veh)*	*Stop Time (min/veh)*
EB to EB	445	-0.05	-0.02	-0.03
EB to NB	612	-0.03	0.0	-0.01
NB to WB	467	0.3	0.37	0.31
SB to EB	259	0.46	0.38	0.54
WB to SB	84	-0.03	-0.25	0.01
WB to WB	630	-0.16	-0.58	-0.1
Volume Weighted Interchange Total		0.05	-0.05	0.08

Table 11 Catawba Avenue Change in Performance Measures Over the Corridor (Before - After)

	PM		
	TT (min/veh)	*Stops (per veh)*	*Stop Time (min/veh)*
EB to EB	0.28	-0.17	0.25
EB to NB	-0.3	-0.34	-0.29
NB to WB	0.51	1.06	0.39
SB to EB	-0.06	-0.39	0.07
WB to SB	0.05	0.11	0.08
WB to WB	0.11	-0.22	0.22

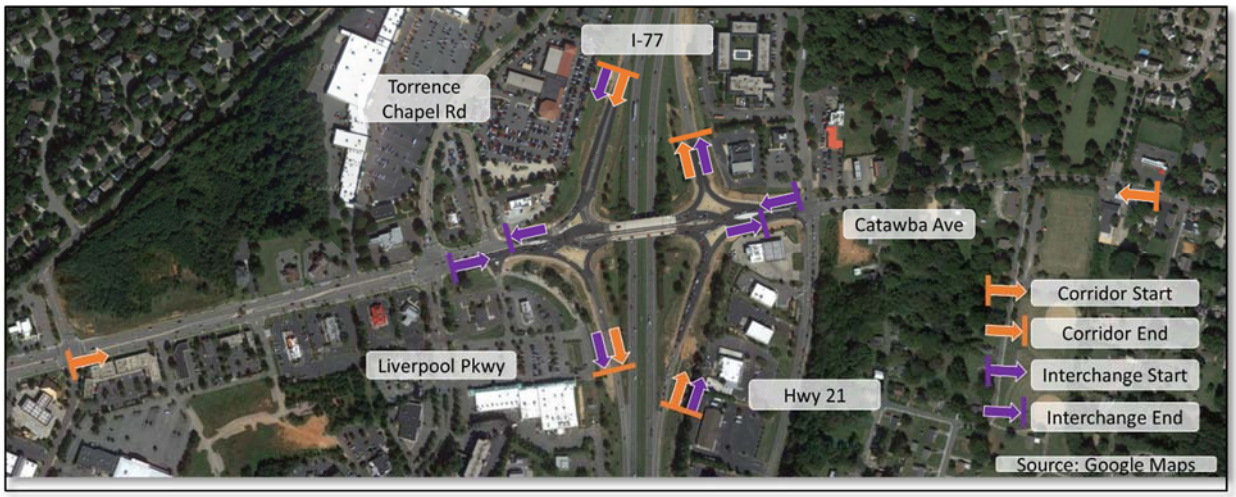


Figure 42 Catawba Avenue Start and End Points for Interchange and Corridor Paths

As can be seen in Figure 43, despite having a significant increase in stops for the westbound through path, progression was generally achieved between the crossovers. Only 25% of sample travel runs stopped at the outbound crossover. Most of the stops on the interchange path came upstream of the east crossover. The offsets at the Highway 21 intersection and the east crossover could be adjusted to provide more progression to the westbound movement, however, that would reduce the progression of eastbound traffic between the intersections. Figure 44 depicts the trajectories for the eastbound path over the corridor with most vehicles progressing through both crossovers and Highway 21. From this figure, it can also be seen that there is cycle failure at the Liverpool intersection. This is a known problem which NCDOT is currently studying.

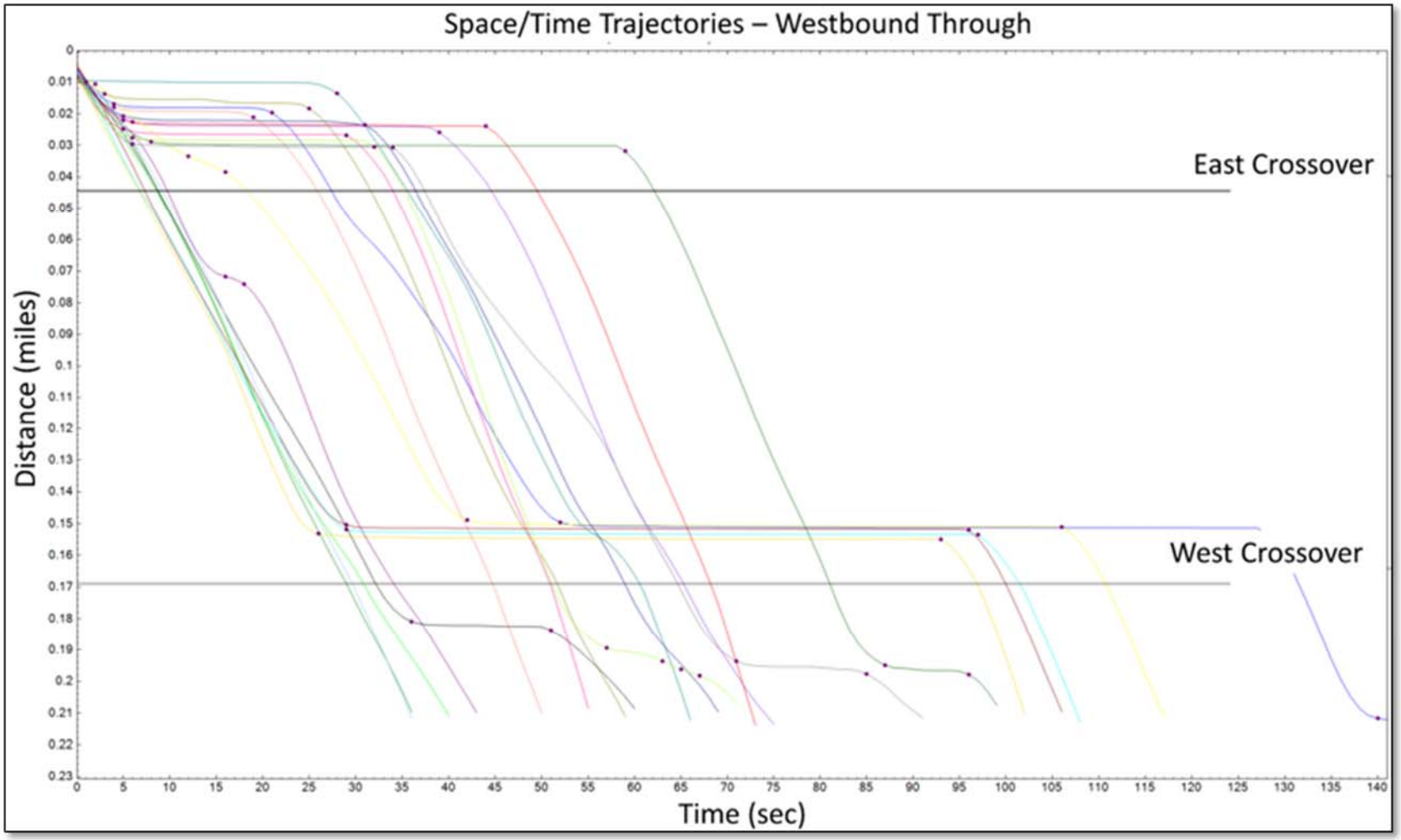


Figure 43 Catawba Avenue After Period Time Space Diagram for Westbound Path Over the Interchange

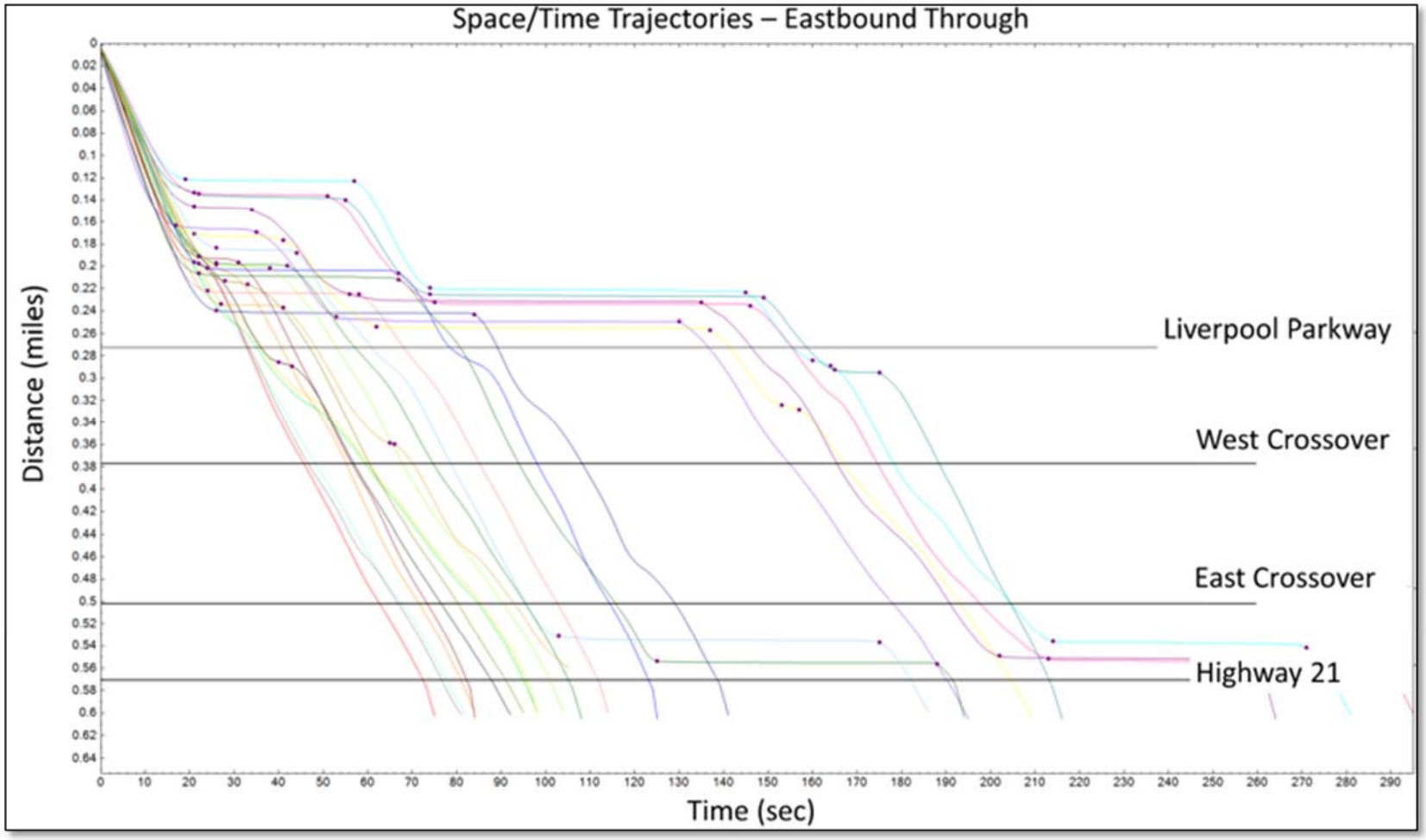


Figure 44 Catawba Avenue After Period Time Space Diagram for Eastbound Path Over the Corridor

6.3.2 Poplar Tent

Table 12 presents the change in travel time, stops, and stopped time over the interchange from the before period to the after period. A positive value means the measure was reduced from the before period to the after period. The volumes reported are those gathered from a January 2015 turn count provided by NCDOT. The results show a significant reduction in travel time for the southbound to eastbound left turn with minor increases in travel time for most other movements. Additionally, in the PM, the average number of stops over the interchange (weighted by volume of each movement) was reduced by 0.39 stops per vehicle. Over the peak hour, this would equate to a reduction on 711 stops across the interchange. Table 13 presents the change in travel time, stops, and stopped time over the corridor from the before period to the after period. Because a full origin-destination study of the corridor has not been conducted, volume weighted results over the corridor cannot be provided. The interchange and corridor start and end points are shown in Figure 45.

Table 12 Poplar Tent Change in Performance Measures over the Interchange (Before - After)

	AM				PM			
	Vol	TT (min/veh)	Stops (per veh)	Stop Time (min/veh)	Vol	TT (min/veh)	Stops (per veh)	Stop Time (min/veh)
EB to EB	408	-0.25	-0.25	-0.26	591	-0.37	-1.03	-0.30
EB to NB	254	-0.14	-0.35	-0.12	290	0.24	0.26	0.23
NB to WB	429	0.19	0.31	0.13	522	0.37	0.22	0.33
SB to EB	55	0.38	0.55	0.36	106	0.58	0.39	0.58
WB to SB	213	-0.04	-0.08	-0.05	157	-0.19	0.05	-0.18
WB to WB	444	0.17	0.18	0.14	442	-0.23	-1.09	-0.15
Volume Weighted Interchange Total		0.02	0.02	-0.01		-0.02	-0.39	0.00

Table 13 Poplar Tent Change in Performance Measures over the Corridor (Before - After)

	AM			PM		
	TT (min/veh)	Stops (per veh)	Stop Time (min/veh)	TT (min/veh)	Stops (per veh)	Stop Time (min/ veh)
EB to EB	-0.31	-0.4	-0.29	-0.35	-0.97	-0.29
EB to NB	-0.14	-0.25	-0.12	0.27	0.32	0.27
NB to WB	0.17	0.32	0.1	0.37	0.22	0.33
SB to EB	0.39	0.64	0.36	0.58	0.58	0.59
WB to SB	-0.11	-0.15	-0.08	-0.21	-0.06	-0.17
WB to WB	0.5	0.71	0.36	-0.06	-0.75	0.04

Figure 45 Poplar Tent Start and End Points for Interchange and Corridor Paths

During implementation, the new signal timing plan was intended to be introduced one week before data collection. This would provide commuters time to adjust to the new cycle lengths and splits. Unfortunately, an error prevented the signal timing plan from being implemented early. Therefore, the drivers first experienced the new signal timing the day of data collection. Research team members observing the interchange noted delayed reaction times after the signal turned green particularly at the west crossover during the PM peak. This crossover originally had a cycle length of 100 seconds which was reduced to 40 seconds. Drivers were observed arriving at the signal on red, diverting their attention from the signal to a cell phone, radio, or other object inside the car, and not returning their attention to the signal until after the green indication appeared. At least two vehicles remained stopped at the intersection for the entire length of the green signal, requiring the vehicle to remain at the intersection through another cycle.

The inability of drivers to learn the signal timing prior to data collection resulted in greater loss time at the signal and therefore lower capacity. Particularly at the westbound approach at the west crossover, queues which may otherwise have cleared the intersection grew with each cycle as the effects of inattentive drivers remaining stopped at a green indication compounded. This effect can be seen in Figure 46 where nearly all vehicles stopped between the crossovers despite the programmed green band shown in Figure 26.

The eastbound vehicles saw increased stops and travel time in the afternoon peak likely due to the half cycling. Recall, the west crossover had a cycle length of 40 seconds while the east crossover had a cycle length of 80 seconds. This allowed for progression for both the westbound through movement and the northbound to westbound left turn. However, it also resulted in every other eastbound platoon stopping at the east crossover.

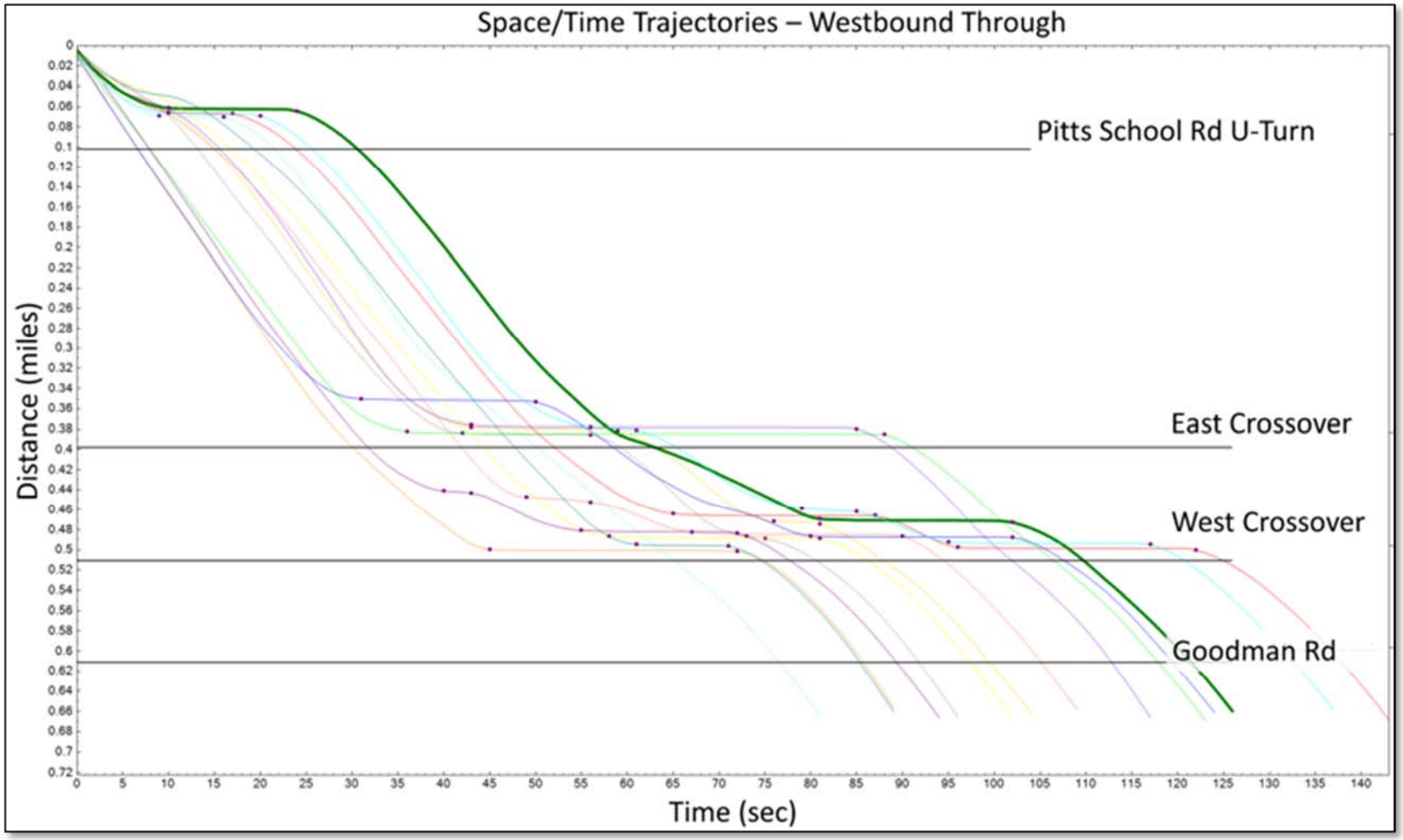


Figure 46 Poplar Tent PM After Period Time Space Diagram for Westbound Path Over the Corridor

6.3.3 NC 73 / Davidson Highway

Table 14 presents the change in travel time, stops, and stopped time over the interchange from the before period to the after period. A positive value means the measure was reduced from the before period to the after period. The volumes reported are those gathered from a January 2015 turn count provided by NCDOT. The results show a significant reduction in travel time for the southbound to eastbound left turn with minor increases in travel time for most other movements. In both the AM and PM, the volume weighted average number of stops per vehicle across the interchange was moderately reduced with only a minor sacrifice to travel time and stop time (less than 10 seconds per vehicle). Table 15 presents the change in travel time, stops, and stopped time over the corridor from the before period to the after period. Because a full origin-destination study of the corridor has not been conducted, volume weighted results over the corridor cannot be provided. The interchange and corridor start and end points are shown in Figure 47.

The same error which resulted in the Poplar Tent signal timing to be implemented on the day of data collection (instead of one week prior) also impacted the NC 73 signal timing. Therefore, the results likely include an increased loss time over what would be expected in the long run. Additionally, the cycle lengths and splits at the DDI are less than optimal due to a pedestrian signal requirement on the coordinated phases. This could be addressed, but it would require a new signal timing plan be filed with NCDOT, which was outside of the scope of this project.

In the morning peak period, four of the six paths through the interchange saw decreased travel times with the two increases in travel times each being less than six seconds. The afternoon peak period saw similar decreases in travel time with the exception of the eastbound to northbound left turn. As Figure 48 shows, only 20% of the sample travel times arrived at the west crossover signal on green. Given the split for that movement is 46% of the cycle length and arrival at the approach is random with no upstream signal to platoon vehicles, this result likely overestimates the true average travel time for the path. Figure 48 also shows cycle failure for two travel time runs. This may be due to the increased loss time from a shorter cycle length and driver inattention as noted above.

Table 14 NC 73 Change in Performance Measures over the Interchange (Before - After)

	AM				*PM*			
	Vol	*TT (min/veh)*	*Stops (per veh)*	*Stop Time (min/veh)*	*Vol*	*TT (min/veh)*	*Stops (per veh)*	*Stop Time (min/veh)*
EB to EB	256	0.18	-0.56	0.2	365	0.26	0.07	0.25
EB to NB	329	0.15	-0.22	0.16	344	-0.42	-0.62	-0.32
NB to WB	156	-0.07	-0.7	-0.04	247	0.48	0.89	0.39
SB to EB	82	0.59	0.64	0.53	65	0.16	0.02	0.21
WB to SB	311	-0.1	-0.34	-0.08	275	-0.06	-0.25	0
WB to WB	420	0.08	0.32	0.02	405	0.11	-0.22	0.24
Volume Weighted Interchange Total		0.09	-0.16	0.08		0.03	-0.14	0.09

Table 15 NC 73 Change in Performance Measures over the Corridor (Before - After)

	AM			PM		
	TT (min/veh)	*Stops (per veh)*	*Stop Time (min/veh)*	*TT (min/veh)*	*Stops (per veh)*	*Stop Time (min/veh)*
EB to EB	-0.1	-0.96	-0.02	0.28	-0.17	0.25
EB to NB	0.17	-0.25	0.18	-0.3	-0.34	-0.29
NB to WB	-0.1	-0.34	-0.02	0.51	1.06	0.39
SB to EB	0.71	0.97	0.63	-0.06	-0.39	0.07
WB to SB	-0.08	-0.27	-0.08	0.05	0.11	0.08
WB to WB	0.09	0.35	0.02	0.11	-0.22	0.22

Figure 47 NC 73 Start and End Points for Interchange and Corridor Paths

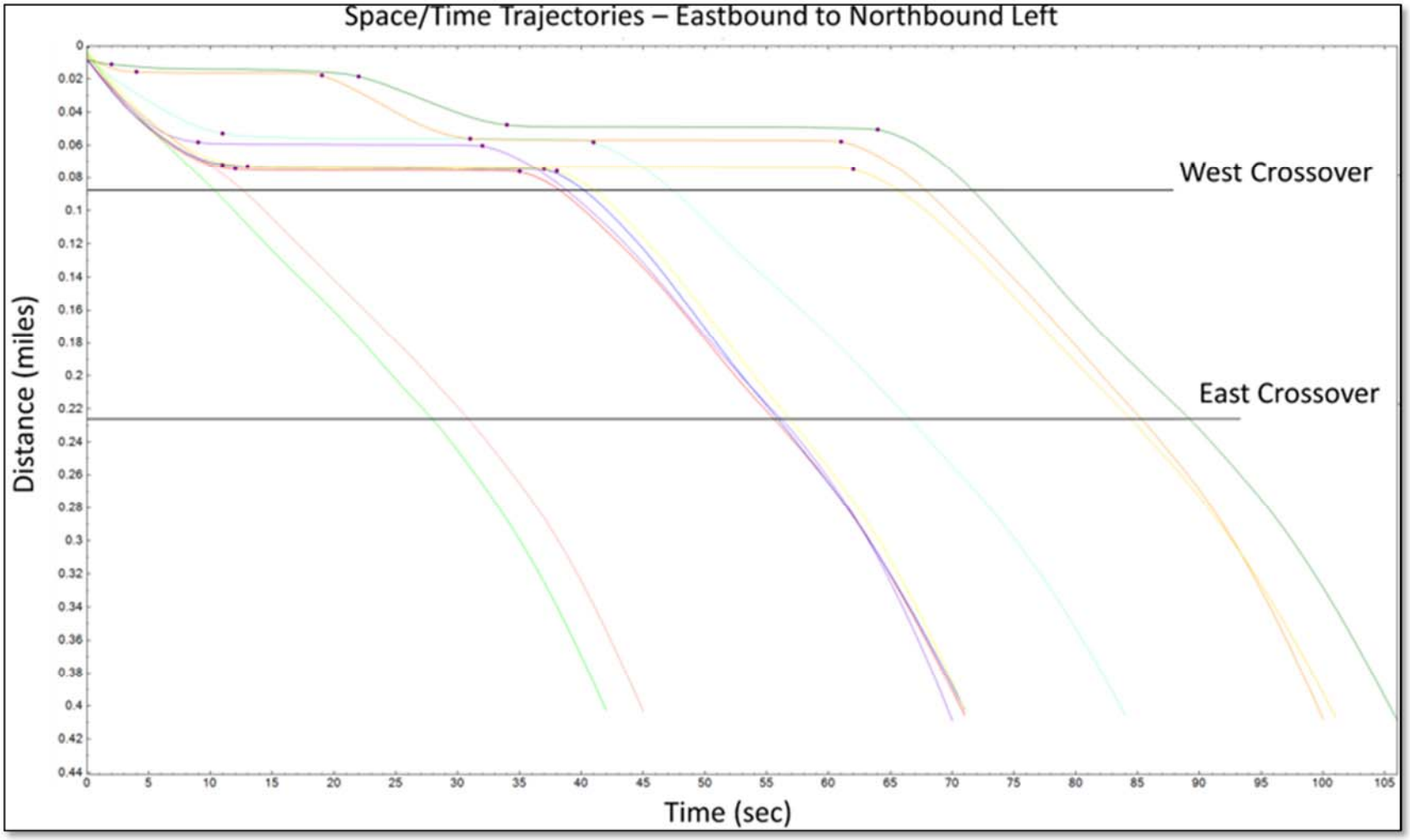


Figure 48 NC 73 PM After Period Time Space Diagram for Eastbound to Northbound Left over the Interchange

6.4 COST, SAFETY, AND DISRUPTION IMPLICATIONS

In examining the feasibility of strategies along a DDI corridor, the cost, disruption during implementation, and safety impacts are important factors to consider alongside the operational benefits that were explored in VISSIM. Cost estimates were generated by the NCDOT Estimating Office. These estimates are highly generalized and do not include the cost of right-of-way acquisition. The accuracy of the cost estimates is directly related to project site characteristics. These estimates should be used for general guidance only.

Table 16 provides estimates (in thousands of dollars) for each portion of a project. The strategies then mark the cost of each signal design element as it was implemented in the simulation. In most cases, the strategy was applied in only one direction. The exception is half cycling, in which both crossovers were retimed.

For instance, the MUT was applied at the northern adjacent intersection. To convert a standard four-leg intersection to a MUT, a designer would need to install two U-turns as well as a directional island at the main intersection. A designer would also need to retime the main intersection and time the two U-turn intersections for a total of three intersections. The estimated costs then would total $518,000.

Table 16 Approximate Cost for Each Strategy (in thousands of dollars)

	New Turn Bay	*Addt'l Lane*	*Signal Timing*	*U-Turn Bulb Out*	*Directional Island*	***TOTAL***
	$90/500'	*$750/mi*	*$6/Signal*	*$75/Turn*	*$350/Island*	
Acceleration Lane		1/5 mi	1			**$156**
Alternate Side Street Phasing			1			**$6**
Dedicated Turn Phase			1			**$6**
Dual Turn Lanes	650'					**$117**
Dual Turn Lanes with RTOR	650'					**$117**
Half Cycle			2			**$12**
Metering			1			**$6**
MUT			3	2	1	**$518**
Rotated MUT			3	2	1	**$518**
RTOR						**$0**
Slip Lane		1/4 mi				**$187.5**
Stretched Superstreet			3	1	1	**$443**
Superstreet			4	2	1	**$524**
Twice Per Cycle Left			1			**$6**

In addition to cost, the disruption to users during implementation as well as safety were also considered as impacts.

Table 17 presents the summary rankings for the strategies, given the measure of cost, disruption to the user during implementation, and safety. Five stars represents the lowest cost, least disruption, or lowest CMF for safety consideration.

Table 17 Comparative Analysis of Cost, Disruption during Implementation, and Safety

	Cost	*Disruption during Implementation*	*Safety*
Acceleration Lane	★★☆☆☆	★★☆☆☆	
Alternate Side Street Phasing(37)	★★★★☆	★★★★★	★★★★☆
Dedicated Turn Phase(38)	★★★★☆	★★★★★	★★★★★
Dual Turn Lanes(39)	★★☆☆☆	★★☆☆☆	★★★☆☆
Dual Turn Lanes with RTOR(39, 40)	★★☆☆☆	★★☆☆☆	★★★☆☆
Half Cycle(41)	★★★★☆	★★★★★	★★★★☆
Metering	★★★★☆	★★★★★	
MUT(42)	★☆☆☆☆	★☆☆☆☆	★★☆☆☆
Rotated MUT(42)	★☆☆☆☆	★☆☆☆☆	★★☆☆☆
RTOR(40)	★★★★★	★★★★★	★★★☆☆
Slip Lane	★★☆☆☆	★★★☆☆	
Stretched Superstreet(43)	★☆☆☆☆	★☆☆☆☆	★★★☆☆
Superstreet(43)	★☆☆☆☆	★☆☆☆☆	★★★☆☆
Twice Per Cycle Left	★★★★☆	★★★★★	

In general, signal timing strategies have lower associated costs and lower disruption to users during implementation. Of those strategies for which safety data could be uncovered, the crash modification factors tended to be higher than those of geometric strategies. It should be noted that the replacement of a standard, signalized intersection with a signalized superstreet results in more crashes overall with a reduction in left turn crashes; however, the replacement of a standard, unsignalized intersection with an unsignalized superstreet results in fewer crashes overall.

Those strategies which utilize access management techniques may encounter negative perceptions from business owners. The proposed addition of a median or redirection of movements to enter or exit a facility may be perceived to lead to a reduction in business due to a lack of direct access to the business. However, after installation, business owners are more likely to view the installations positively and there is no direct evidence to suggest median installations result in a negative economic impact (*44*).

7 CONCLUSION

This research effort examined the corridor impacts of various signal timing and geometric strategies to improve the operational challenges observed at DDIs. A microsimulation analysis was conducted using a calibrated and validated DDI modeled after the National Avenue and US-60 interchange in Springfield, Missouri. Four heavy volume scenarios were tested in combination with seven categories of strategies. These strategies were selected from a larger pool of strategies under the guidance of the NCDOT research panel and national expert recommendations. In addition to the microsimulation effort, a cost analysis was conducted for the same strategies. Considerations were made for implementation cost, disruption to user during implementation, and crash modification impacts. Finally, three sites in North Carolina were selected for field study. The sites included Poplar Tent Road and I-85 in Concord, NC-73 and I-85 in Concord, and Catawba Avenue and I-77 in Cornelius. Due to schedule and budget constraints, only those signal timing strategies which did not require a signal timing plan change were considered for implementation.

In the microsimulation analysis, those strategies which reduced the number of phases at the downstream adjacent intersection had the greatest benefit on the corridor routes for all four heavy volume scenarios. The reduction in phases reduced loss time and increased capacity for the intersection. Unfortunately, these strategies were also the most expensive alternatives studied, were likely to be the most disruptive to users during implementation, and had marginal safety benefits compared to other strategies.

In general, signal timing strategies had lower associated costs and lower disruption to users during implementation. Of those strategies for which safety data could be uncovered, the crash modification factors tended to be higher than those of geometric strategies. In microsimulation, the use of a half cycle at the DDI crossovers provided benefits to the route of interest without strong negative impacts to opposing routes.

The field implementation testing was limited in scope due to schedule and budget constraints, but a number of lessons were gathered:

- Half cycling does tend to reduce interchange delays when there is a clear peaking direction. Volume weighted stops may increase under half cycling if a progression green band cannot be provided for all outbound movements in both half cycles;
- Actuating the crossover at which the heavier outbound movement is located allows for flexibility in splits while maintaining coordination between the two crossovers;
- In the presence of a heavy through movement and a heavy left turn from the exit ramp both with the same destination, half cycling the outbound crossover may allow for progression of both movements; and

- When possible, the cycle length of the corridor should be driven by the travel time between the two crossovers. A cycle length which is a multiple of the travel time generally provides opportunities to progress both corridor through movements at the DDI.

Further efforts are under way at the national level to quantify the operational and safety impacts of various geometric design factors at DDIs across the country. Additionally, efforts are wrapping up on new, detailed guidance for signal timing strategies of corridors which contain DDIs. These strategies include some strategies tested in this report in addition to various phasing schemes applied exclusively to the DDI. Considering the presence of conflicting mainline movements, future research should be considered on the development of a signal timing optimization process for a DDI corridor. This research should formalize the selection of cycle length and offset selection for the DDI as well as the surrounding adjacent intersections.

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9 APPENDICIES

A. STATEGY EXPLANATIONS

Issues	***Primary Focus Area***	***Strategies***	
		Signal Timing	***Geometric Design***
Queue Spillback	Operations	**A, B, C, D**	J
Demand Starvation	Operations	**A, B, C, D**	J
Right turn at Off-Ramp	Safety & Operations	E, **F, G, U**	J, K, L, M, N, O, P
Left Turn at Off-Ramp	Safety & Operations	**F, G**	
Mainline Progression (non-peak direction)	Operations	**B, C, H**	J
Lane Utilization	Operations		R, S
Ramp Metering	Operations	F	T
Pedestrians	Safety	I	R
Emergency Vehicle Preemption	Safety	F	
Wrong-way Maneuvers	Safety		Q

All mention of specific intersections are in reference to Figure 49 considering the blue arrows as the main direction of movement.

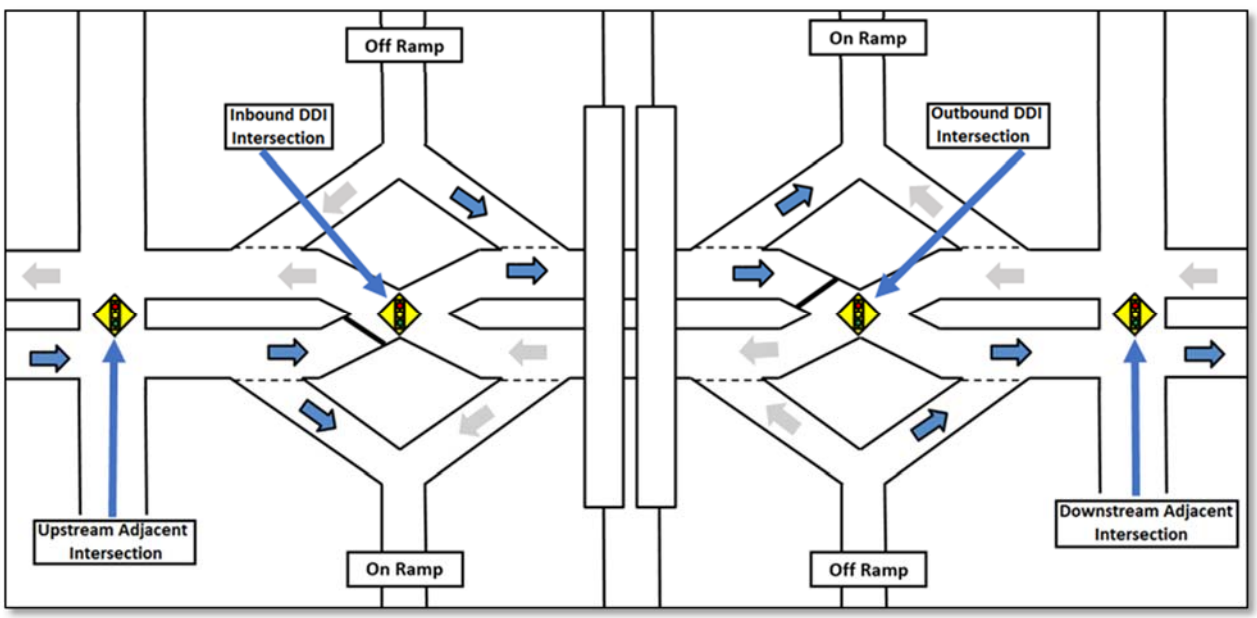


Figure 49 DDI Intersection Naming Schematic

A. Adjustment of Outbound g/C at Upstream Adjacent Intersection

All efforts should be made to optimize the downstream adjacent signal so that throughput can be maximized with minimal queue spillback into the DDI. If spillback still occurs, the throughput of the upstream adjacent signal can be artificially reduced to match the capacity of the downstream adjacent signal. This will eliminate spillback through the outbound DDI intersection, but inherently create a queue upstream of the upstream adjacent signal where traffic will now be metered.

If a downstream adjacent signal does not pose problems with queue spillback into the DDI, or if the intersection is located a significant distance away from the outbound DDI signal, the upstream adjacent signal should be looked at carefully to make sure that demand starvation is not occurring at the inbound DDI signal. The upstream adjacent signal will need to use the combined demand from three (right, though, and left) movements to utilize as much capacity as possible at the DDI.

B. Reduction in Number of Phases at Downstream Adjacent Intersection

For upstream and downstream adjacent intersections, additional mainline capacity could be achieved by eliminating phases, or by changing phasing from protected movements to permitted or permitted/protected movements. Eliminating phases is possible through a variety of alternative intersection forms, like the superstreet/RCUT or the median u-turn intersection (also called Michigan-left). If side street volumes on one of the approaches are extremely low, another option may be to provide split side street phasing. This will allow this phase to be skipped the majority of the time, providing additional capacity to the mainline as the unused time for side street phases is added to the mainline several times per hour.

C. Lead/Lag phasing for Outbound Left at Downstream Adjacent Intersection

Lead/Lag phasing is being utilized much more frequently by states now that safety concerns with the "yellow trap" have been addressed through flashing yellow arrows and other innovations. Lead/lag phasing allows signal designers flexibility to choose when to start the left turn phase in a specific ring so that maximum bandwidth can be achieved along the coordinated movement for the purpose of progressing traffic.

D. Alternate Side-Street Phases at Downstream Adjacent Intersection

The phasing scheme at an intersection adjacent to the DDI could be adjusted to provide additional capacity to a mainline approach by alternating side street movements every other cycle. This unusual phase scheme could be used in a time-of-day plan when traffic on the mainline is excessive and there is a great need for additional capacity to prevent queue spillback into the DDI interchange. On the next cycle, the mainline is again serviced, followed by the alternate minor approach.

E. No Right Turn on Red (RTOR) Allowed at Off-Ramp

Three primary issues arise for right turning vehicles at the off-ramp. First, and not unique to the DDI, is the potential weave that takes place for vehicles turning right off of the freeway, and quickly maneuvering over multiple lanes of traffic to turn left onto a minor street or driveway.

The last two, which are unique to the DDI, are the limited sight distance across the barrier often placed in the middle of the interchange and problems with motorists looking down the wrong upstream approach when turning right onto the arterial. If no geometric or signal strategy can be effectively utilized (i.e. strategies F, G, J, K, L, M, N, O, or P), then eliminating the option to RTOR will, at a minimum, improve safety of the right turn movement at the off-ramp.

F. Vehicle Preemption

Preemption provides a good mechanism for alleviating excessive queuing (i.e. "flushing"); however, it should be used sparingly and only when a safety concern cannot be addressed by some other means. Two primary safety concerns that could be addressed by preemption are emergency vehicle response time and queue spillback onto the freeway. Preemption could be used by emergency responders where excessive traffic demands prevent quick emergency response times. Preemption in this case would flush excessive demand in the emergency vehicle direction of travel allowing emergency vehicles to respond faster to emergencies.

At exit ramps with high demand from left or right turn movements, preemption could be used to help flush queues that are on the verge of spilling back onto the freeway. These movements often do not allow RTOR or LTOR due to safety concerns – a concern that many signal designers must address with the DDI compared to the diamond interchange, especially the right turn. The preemption strategy requires a loop to be installed near the exit ramp gore. When the queue approaches the freeway, the signal is preempted to add a phase for left and right turns from the ramp as shown in Figure 50 (representing the northbound freeway off-ramp onto an east-west arterial street with a DDI).

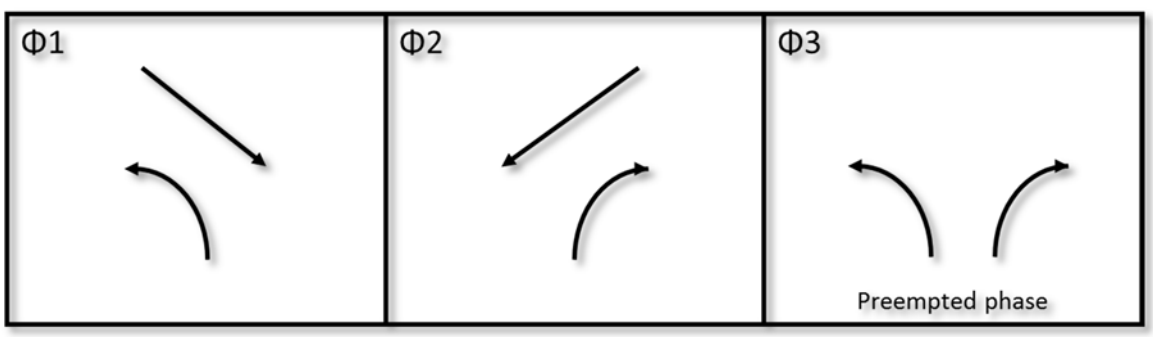


Figure 50 Phasing Schematic with Dedicated Off-Ramp Turn Phase

This ramp preemption method is used regularly by several states with queue spillback problems at interchanges; however, the first known DDI location to use this method is in St. George, Utah which is set to open by end of year.

Preemption should be used carefully for unusual safety problems that cannot be addressed by other signal or roadway design methods. Preemption of a signal on a recurring basis could cause excessive delays and queues to form at intersections as additional phases are called or split times are increased, knocking the controller out of coordination for several cycles.

G. Dedicated Phase for Concurrent Left and Right Turn at Off-Ramp

For continuously high demand at the off ramp during the peak hour, an additional phase can be added to a time of day signal plan which services left and right turns from the off ramp. A possible sequencing is show in Figure 50. This phase would not be preempted, but utilized throughout the entire plan period.

H. Half Cycle

Using half cycles at some intersections may provide better progression of the off-peak traffic by opening the green band more often. This would also discourage red light running at DDI intersections which only run two phases as compared to other signals within the same system. Half-cycling was employed recently at the first DDI in Atlanta, GA with some success.

I. Signalize Left On-Ramp

Pedestrian facilities that utilize the outside of the DDI must cross a free-flow left turn onto the on-ramp. For pedestrians, this movement could be signalized with a dedicated pedestrian phase, a beaconing device such as a pedestrian hybrid beacon (PHB) or rectangular rapid flashing beacon (RRFB). To avoid creating a left turn queue on the overpass, the pedestrian signal could be coordinated with the inbound DDI signal. When inbound traffic receives the red, the pedestrian crossing would allow movement after an offset equal to the amount of time necessary for vehicles to travel from the inbound intersection to the on-ramp.

J. Access Management

In some cases, adjacent driveways or signalized intersections become problematic, causing safety and operational concerns at interchange ramp terminals. The increased throughput of DDI's makes this problem even more alarming. Using principles of access management, median treatments and/or movement of conflicting approaches to other alternative locations (U-turn, grade separated, combine approaches, etc.) helps remove conflicts and signal phases at a main intersection while providing much better operation along the corridor. Two such examples are provided in Figure 51 for before and after DDI installations in Springfield, MO and Maryland Heights, MO.

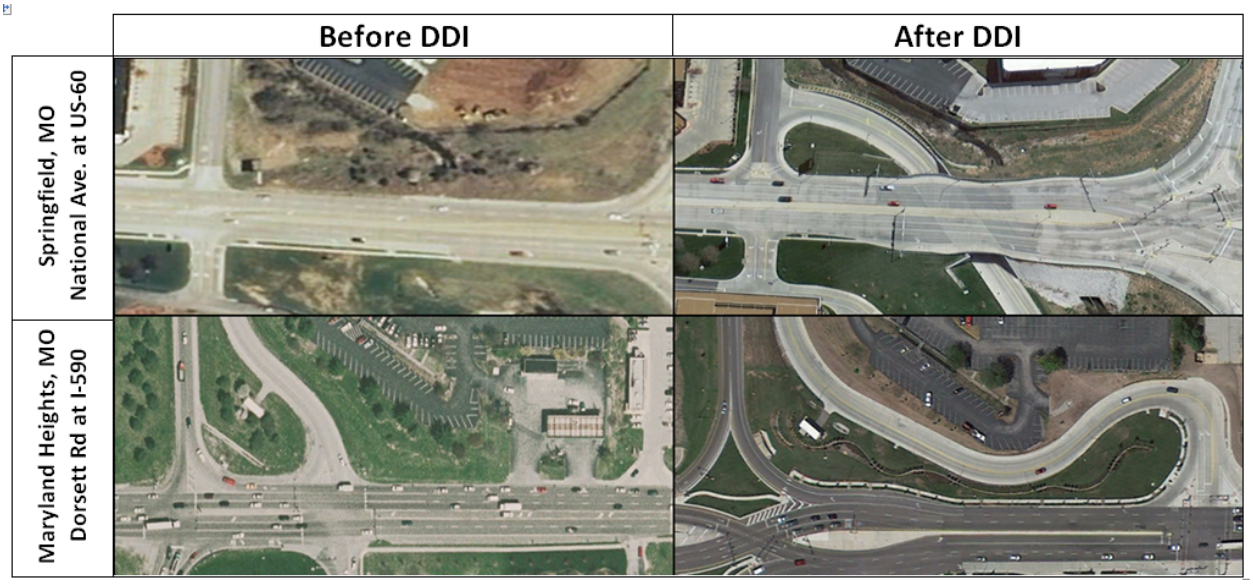


Figure 51 Access Management treatments at DDI's located at National Ave. in Springfield, Mo and Dorsett Road in Maryland Heights, MO (Source: Google Maps)

While designing and constructing the National Avenue DDI, the full access intersection at the hospital north of the DDI was converted to a right-in-right-out grade separated movement with an underpass to accommodate left turns. A second example is provided at the Dorsett Road DDI, where a closely spaced approach was tied in with an adjacent intersections further away from the DDI crossover. In addition, alternative intersections are being considered by some states such as NC, where a Restricted Crossing U-Turn (RCUT, often referred to as a Superstreet) and Continuous Flow Intersections (CFI) are being designed and constructed along a corridor where a DDI is being proposed.

K. Dual or Triple Right Turn Lanes

To counteract the reduced capacity caused by prohibited RTOR, dual or triple right turn lanes could be installed at the off ramp allowing for additional capacity during the green interval.

L. Relocate Right Turn at Off-Ramp

If right-of-way is available, the right turn from the off-ramp could be relocated further downstream of the crossover. This would provide drivers with greater sight distance when looking to merge with upstream traffic from the crossover; however, it would result in a reduced distance to weave if drivers desired to turn left at the downstream adjacent intersection. In American Fork, Utah, designers placed the gore of the right turn 240 feet downstream of the outbound DDI intersection as shown in Figure 4.

Figure 52 Look Back View for Drivers Turning Right from Off-Ramp at Pioneer Crossing and I-15 in American Fork, UT (Source: Google Maps)

M. Channelized Turn Lane (CTL) with Acceleration Lane

If right-of-way is available, a channelized turn lane could be added for right turns from the off-ramp as was done in Figure 53. If an adjacent closely spaced intersection is present, ideally the acceleration lane would continue through the downstream adjacent intersection, providing greater capacity at that signal.

Figure 53 CTL with acceleration lane at Bangerter Hwy in Salt Lake City, UT

N. Right Turn Slip Lane

A slip lane is useful when moderate to high volumes of right turning vehicles at the off-ramp also turn right at the downstream adjacent intersection. A slip lane originates at the off-ramp for right turning vehicles and continues to the downstream adjacent intersection where vehicles turn right, such as the eastbound right turn at the DDI in Figure 54.

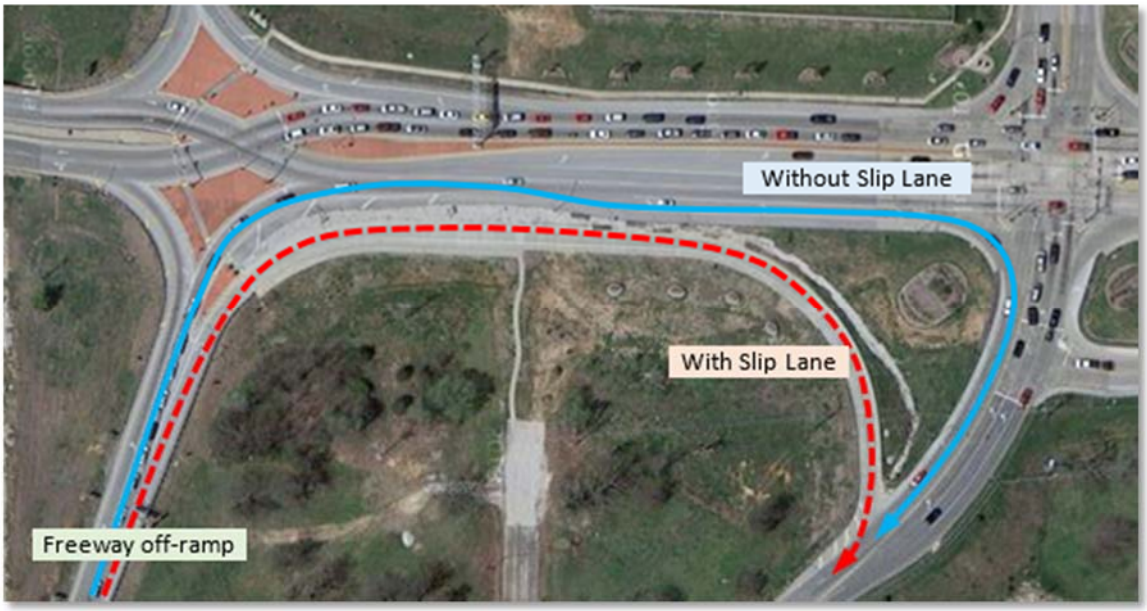


Figure 54 Slip Lane at MO-13 and I-44 in Springfield, MO (Source: Google Maps)

O. Sight Distance Improved and/or Realignment for Right Turn at Off-Ramp

In an effort to provide drivers with more sight distance from the right turn lane at the off ramp, the end of the median barrier wall could be shaved down as in or the height of the entire wall could be reduced. Figure 7 demonstrates the lack of sigh distance behind median walls at an over- and underpass. At National Avenue, the high median barrier makes it particularly challenging to see oncoming traffic and actually may encourage motorists to look down the wrong approach. Dorsett Road provides a shorter height barrier; however, the addition of aesthetic designs on the barrier and along the retaining walls detracts from the additional sight distance. In addition to sight distance considerations, the off ramp can be realigned such that the right turn lane stop bar is near parallel to the inbound traffic at the mainline crossover. This would deter merging drivers from accidentally looking for gaps in the wrong approach because drivers would look over their shoulder down the correct approach (1).

Figure 55 Shaved Median Wall at Dorsett Rd and I-44 in Maryland Heights, MO (Source: Google Maps)

Figure 56 Median Wall Blocking Sight Distance at Nat'l Ave and US-60 in Springfield, MO (Source: Google Maps)

P. Glare Screen

Glare screens were first considered at DDI's to help remove glare concerns from oncoming vehicles in the crossover. An example is provided in Figure 57. However, after opening MO-13, engineers observed there were no issues with headlight glare. Therefore, no glare screens have been installed at DDIs. Glare screens could, however, be considered as a strategy for addressing the RTOR constraint. Where right-of-way constraints remove the ability to provide RTOR operations, the glare screen could be considered to channelize the sight line into the correct approach at the crossover. Intersection sight distance will still need to be addressed if median barrier walls block the field of view.

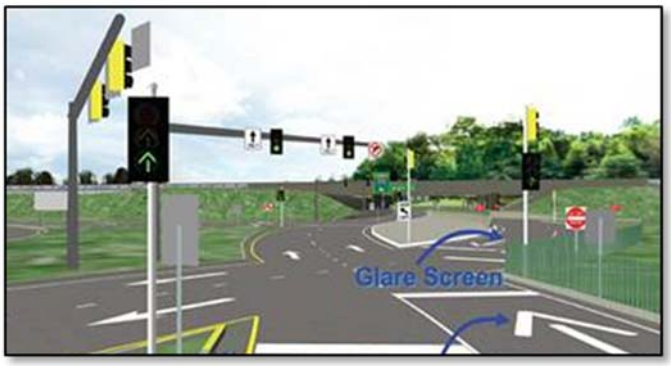


Figure 57 Example glare screen installation blocking the sight lines of the opposing appraoch at the crossover (AIIR)

Q. General Design Considerations

Heavy vehicle and wrong-way maneuver considerations should be taken into account during the initial design. Three primary concerns *exist at the crossover* related to trucks and wrong-way movements. First, the crossover angle should be carefully considered as recommended practice is to cross the two movements at 45 degrees (Hughes, et al. 2010). Second, a tangent should be added between the two reverse curves to prevent vehicle path overlap which may inadvertently

guide motorists into opposing traffic. Last, the lack of superelevation in the crossover could lead to tipping of heavy vehicles, so the curve radii leading into the crossover should be designed to slow vehicles considerably. In addition, the width of the crossover lanes should be able to accommodate trucks.

Another consideration is the left and right turn at the off-ramp. Sufficient turning radii should be provided as to not cause tipping of heavy vehicles. If multiple lanes exist at these turning movements, the likelihood of tipping is reduced; however, the potential to "pinch" a vehicle in the inside turning lane is more likely. This phenomenon has been documented for some DDIs for vehicles making a left turn from the freeway where dual left turn lanes exist, which is an unusual maneuver for heavy vehicle operators. This is especially true for vehicles at rest behind the stop bar waiting for a green light. Adding a short tangent length before and after a crossover will greatly assist in aligning vehicles with their correct receiving lane as they approach the crossover. Designers will need to weigh the tradeoffs between the available right-of-way, tangent length through the crossover, and curve radii considered during the design. For most designs, a sufficient tangent to provide necessary path alignment will require a tangent of approximately 100 feet, with slightly more of the total tangent leading into the crossover versus existing.

R. Signing and Pavement Marking

Lane configuration and discipline upstream of the interchange is very important for safety and operations at the DDI. Drivers should be directed using appropriate signage and pavement markings as to the lane assignments for left turns at the on-ramp and through movements along the arterial. This will avoid last minute weaving by drivers and reduce confusion in the interchange. Overhead signage indicating lane usage should be located in a manner such that inbound motorists can make a lane well ahead of the inbound DDI intersection. Similarly, pavement markings should be provided in a manner that provides appropriate guidance through the main intersection and to the appropriate downstream lane. Wrong-way arrows and elongated shield markings such as those shown in Figure 57 and Figure 58 are two strategies currently in use at DDIs.

Figure 58 Large pavement marking arrows at inbound and outbound movements in Rochester, NY.

Figure 59 Shield markings located at inbound movements at Dorsett Rd. and I-277 in Maryland Heights, MO.

In addition, because drivers traverse the DDI on the non-traditional side of the road, pedestirans looking for gaps at unconventional crossing locations may tend to look toward the wrong direction of travel prior to crossing. Therefore, unique sign and pavement marking treatments could be considered to assist pedestrians during the crossing task. Two solutions could be to provide supplemental signage at eye level (Figure 60) and/or embossed pavement (Figure 61) both of which could provide warning to pedestrians to look in the appropriate direction for oncoming vehicles.

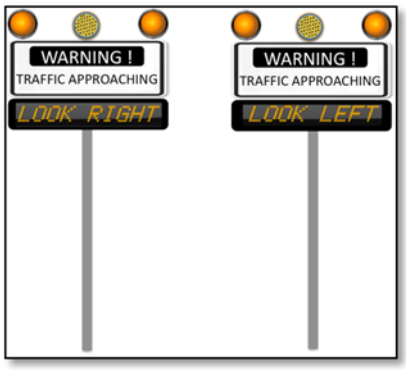


Figure 60 Pedestrian Signage for Notice of Traffic Direction

Figure 61 Pedestrian Pavement Marking for Notice of Traffic Direction at Dorsett Rd and I-270 in Maryland Park, MO. (Source: Google Maps)

S. Guidance on Where/When to Add/Drop Lanes

To allow greater capacity onto the on-ramp for left turning vehicles, the dedicated through lane immediately next to a dedicated left turn lane could be converted to a shared through/left lane adding additional capacity onto the freeway with a two-lane left turn (one shared with through). Similarly, double dedicated left turn lanes could be constructed to feed into the on-ramp. The two left-turn lanes would then merge downstream at an appropriate location.

Another option could be to provide a separate left turn maneuver through the crossover that would require vehicles to pre-position in advance of the first inbound signal of the DDI, such as the DDI in Seclin, France shown in Figure 62. A similar DDI is being considered in Round Rock, TX.

Figure 62 Dedicated left turn lanes for DDI at Route d'Avelin and Autoroute du Nord in Seclin, France (Source: Google Earth)

Configuring lane geometry in this pattern (where necessary) would allow prepositioned vehicles in the left-most lane upstream of the diverge point to be spread more evenly over two lanes, reducing the queue length at the inbound DDI signal and utilizing more available capacity. This should be paired with proper advanced signing and pavement markings so drivers are aware of the lane assignments.

Similarly, an additional lane can be added upstream of the inbound DDI intersection to create more capacity, as was done at Dorrsett Road in Maryland Heights, MO in Figure 63.

Figure 63 Lane Added Upstream of Inbound DDI Intersection at Dorsett Rd and I-44 in Maryland Heights, MO (Source: Google Maps)

T. Add Storage Capacity to On-Ramp

At interchanges that intersect with congested freeways, the on-ramp can be problematic – especially when ramp meters are used which can force vehicles to queue on the ramp to allow improved freeway operations. However, queues on short metered on-ramps can spillback into the DDI, causing unintended delays on the arterial. In Utah, an additional lane was added to the on-ramp so there were two storage lanes upstream of the meter. In addition, the meter could be moved closer to the gore if the acceleration lane on the freeway were extended. The extended lane would help reduce turbulence at the ramp influence area by providing drivers with additional time to reach freeway speeds and select a gap for merging.

U. Right Turn Strategy

A combination of strategies E, O, L, K, M, and N, strategy U tests right turn strategies.

- RTOR (E) / Sight Distance Improvements (O) / Relocate Right Turn (L)
- Dual turn lanes (K)
- CTL with acceleration lane (M)
- Right turn slip lane (N)

B. QUICK REFERENCE GUIDES BY HEAVY VOLUME

B.1 Heavy Through Movement

Movement of Interest: Northbound and Southbound Corridor Movements

Volume: 1380 vph NB; 920 vph SB

Strategies Tested:

- Alternating side street phasing
- Half cycle at the DDI crossovers
- Metering traffic at the upstream adjacent intersection
- Median U-Turn
- Rotated Median U-Turn
- Stretched superstreet with DDI between the u-turns
- Superstreet
- Twice per cycle left at the downstream adjacent intersection

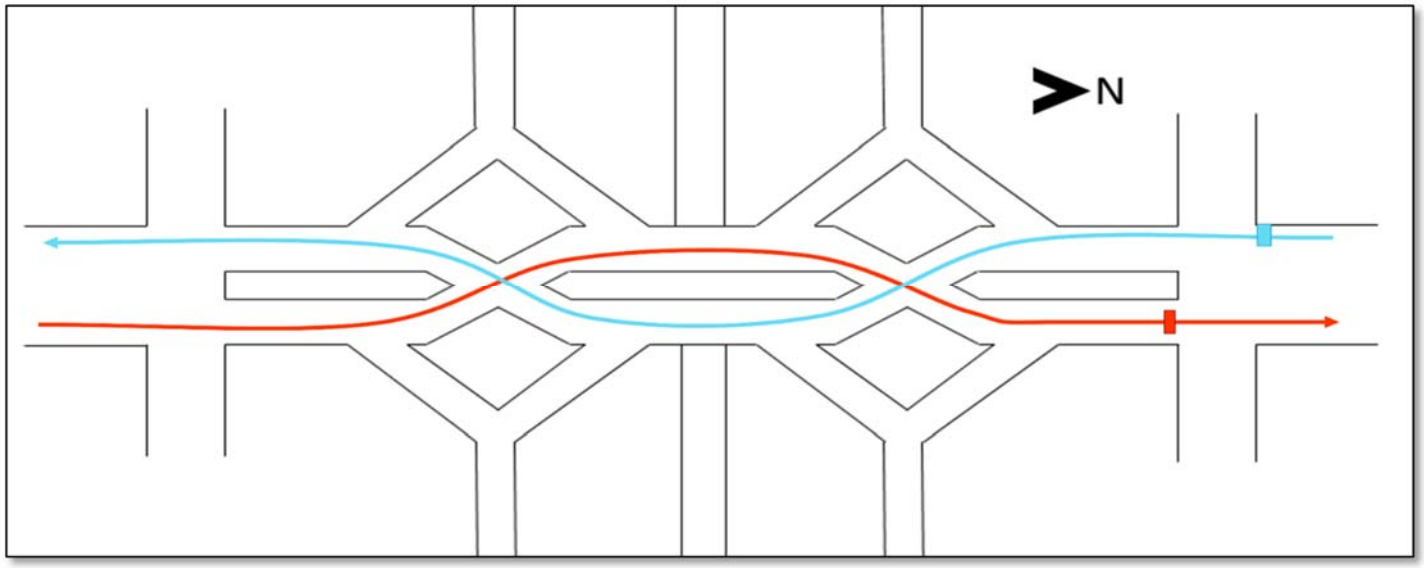
Measures of Effectiveness:

- Average Maximum Queue Length as measured at a movement – the maximum queue length was measured once per cycle and those results were averaged
- Delay as measured over a origin-destination path – this delay includes all times when the vehicle was driving less than the desired speed due to stop, control, and geometric delay
- Stop Severity Index as measured over a path – this measurement considers the number of times a vehicle stopped as well as the number of intersections it traveled through given the following equation $SSI = \frac{\#\ of\ stops^2}{\#\ of\ intersections}$
- Green Usage as measured at a movement – the percentage of time during which the green signal was used as defined by a vehicle passing the stop bar

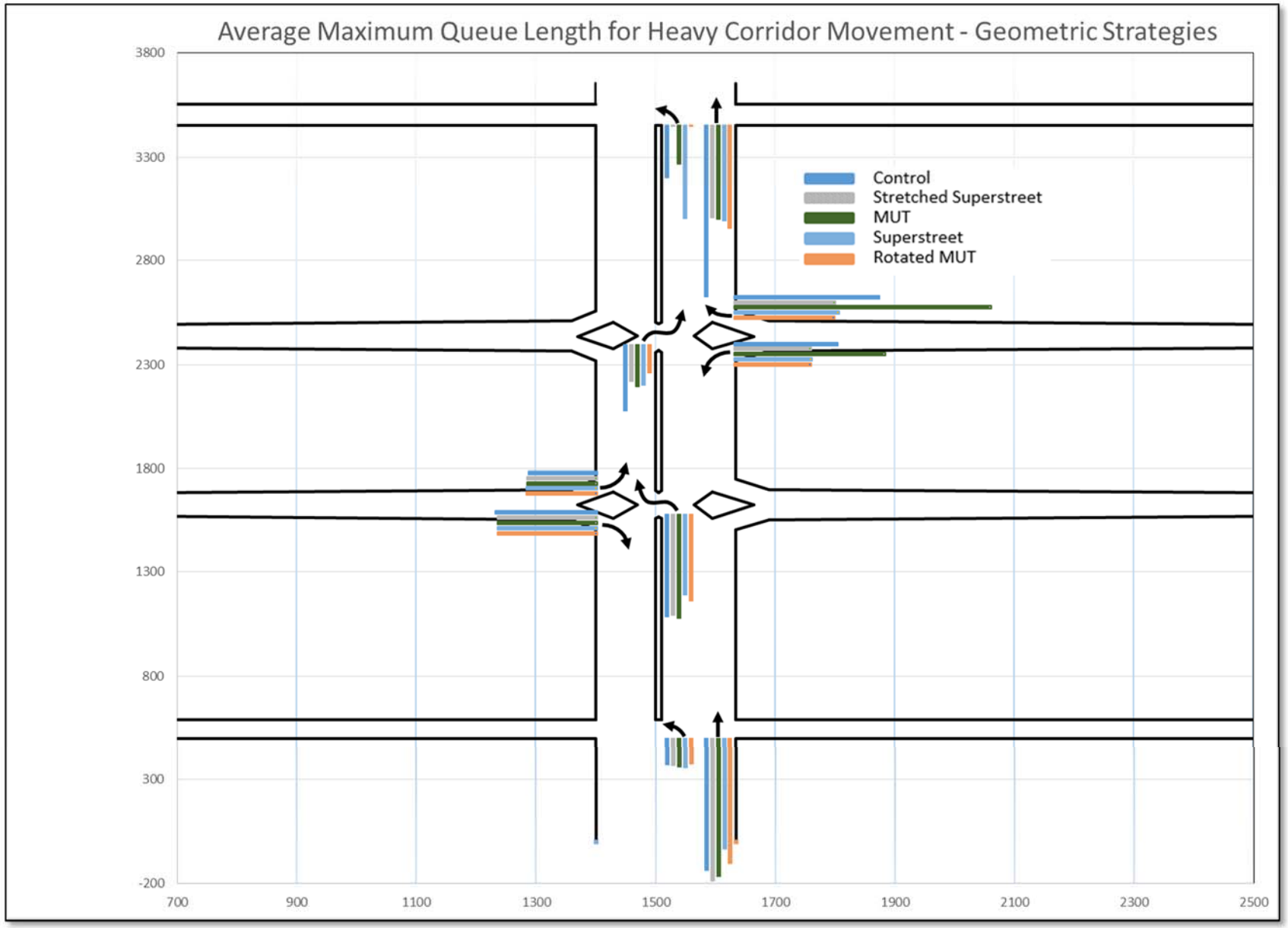
Overview of Results: Presented below is a diagram of a DDI with the two corridor movements. Also included is a table featuring the four measures of effectiveness and their value under the control (no strategy) scenario for each of the movements highlighted in the diagram. Below the control values are the improvements seen in the measure of effectiveness for each strategy relative to the control. An improvement of 50%+ is shaded dark green while an improvement of -50%+ is dark red. The path over which delay and the stop severity index (SSI) is measured is indicated by a line while the movement at which the average maximum queue length and green usage is measured is indicated by a block.

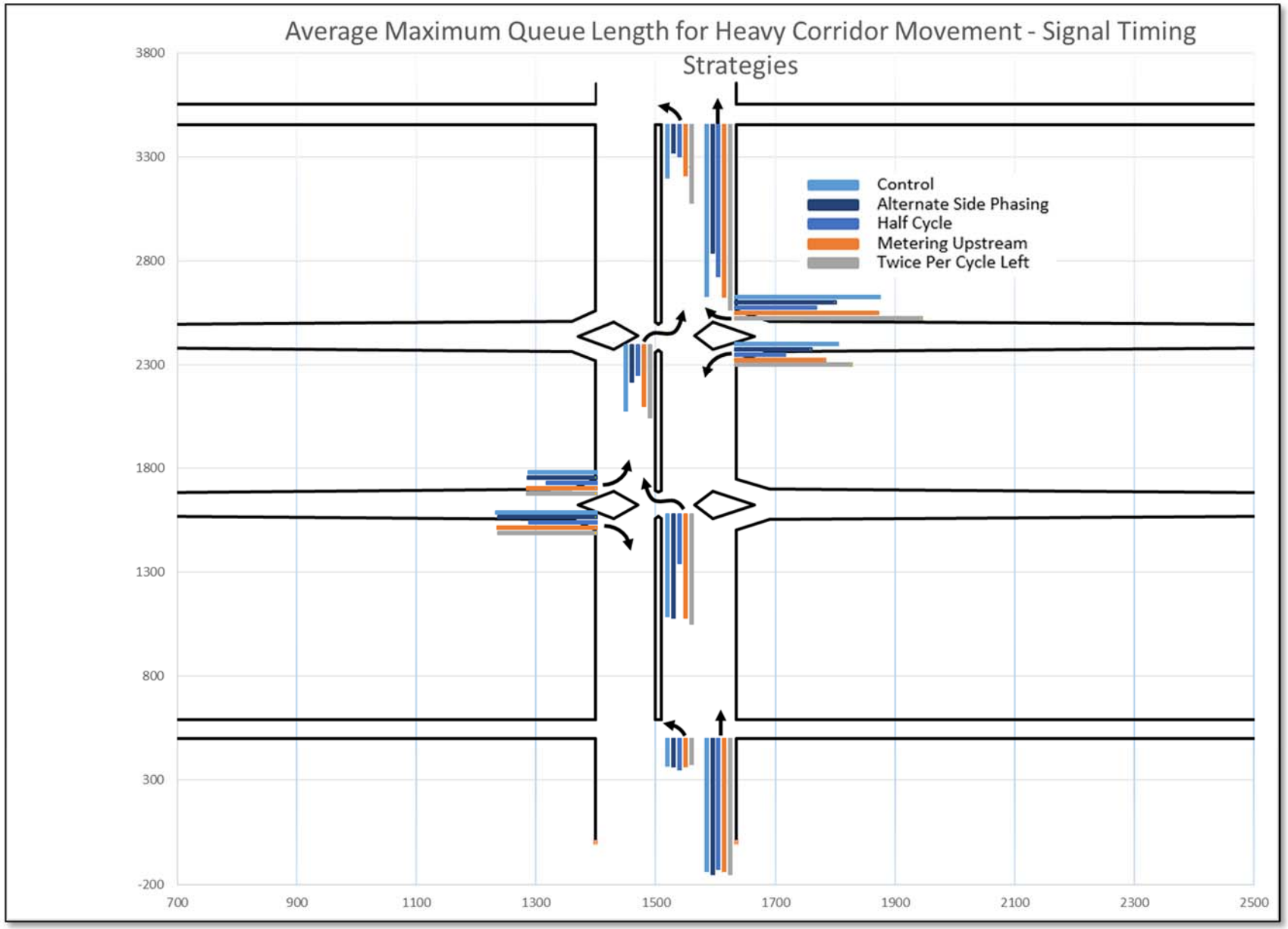
Further Considerations: Below the condensed results, four charts detail the full results for each measure of effectiveness. For the heavy corridor movement, it is important to consider the impact to east- and westbound to northbound turning movements at the crossover. Because the directional split along the corridor is 60/40 to the north, the turning movements headed north are more heavily impacted in the simulation than the turning movements headed south.

Supplemental Effects: Strategies which result in longer queues for the northbound movement at the northern adjacent intersection are showing residual queues for the westbound to northbound right turning movement. Additionally, the strategies tend to favor the northbound corridor movement as opposed to the southbound movement. This is may be due to the directional split which favors the northern movement.



	North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	181 seconds	SSI = 3.5	56%	822 ft	119 seconds	SSI = 2.1	45%
Alternating Side Street	0.81	0.67	0.69	0.89	1.01	0.91	0.64
Half Cycle	0.51	0.70	1.01	0.75	0.70	0.55	0.92
Metered	0.99	0.96	1.00	1.01	0.97	0.97	1.00
MUT	0.91	0.77	1.20	0.55	1.01	0.71	0.98
Rotated MUT	0.83	0.68	0.95	0.60	1.14	0.91	0.90
Stretched Superstreet	0.65	0.34	-	0.54	1.03	0.71	-
Superstreet	0.62	0.42	-	0.56	0.97	0.73	-
Twice Per Cycle Left	1.09	1.22	1.01	1.08	0.96	0.96	0.99


Average Maximum Queue Length for Heavy Corridor Movement - Geometric Strategies
3800
3300
2800
2300
1800
1300
800
300
-200
700
900
1100
1300
1500
1700
1900
2100
2300
2500
Control
Stretched Superstreet
MUT
Superstreet
Rotated MUT


Average Maximum Queue Length for Heavy Corridor Movement - Signal Timing Strategies
3800
3300
2800
2300
1800
1300
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300
-200
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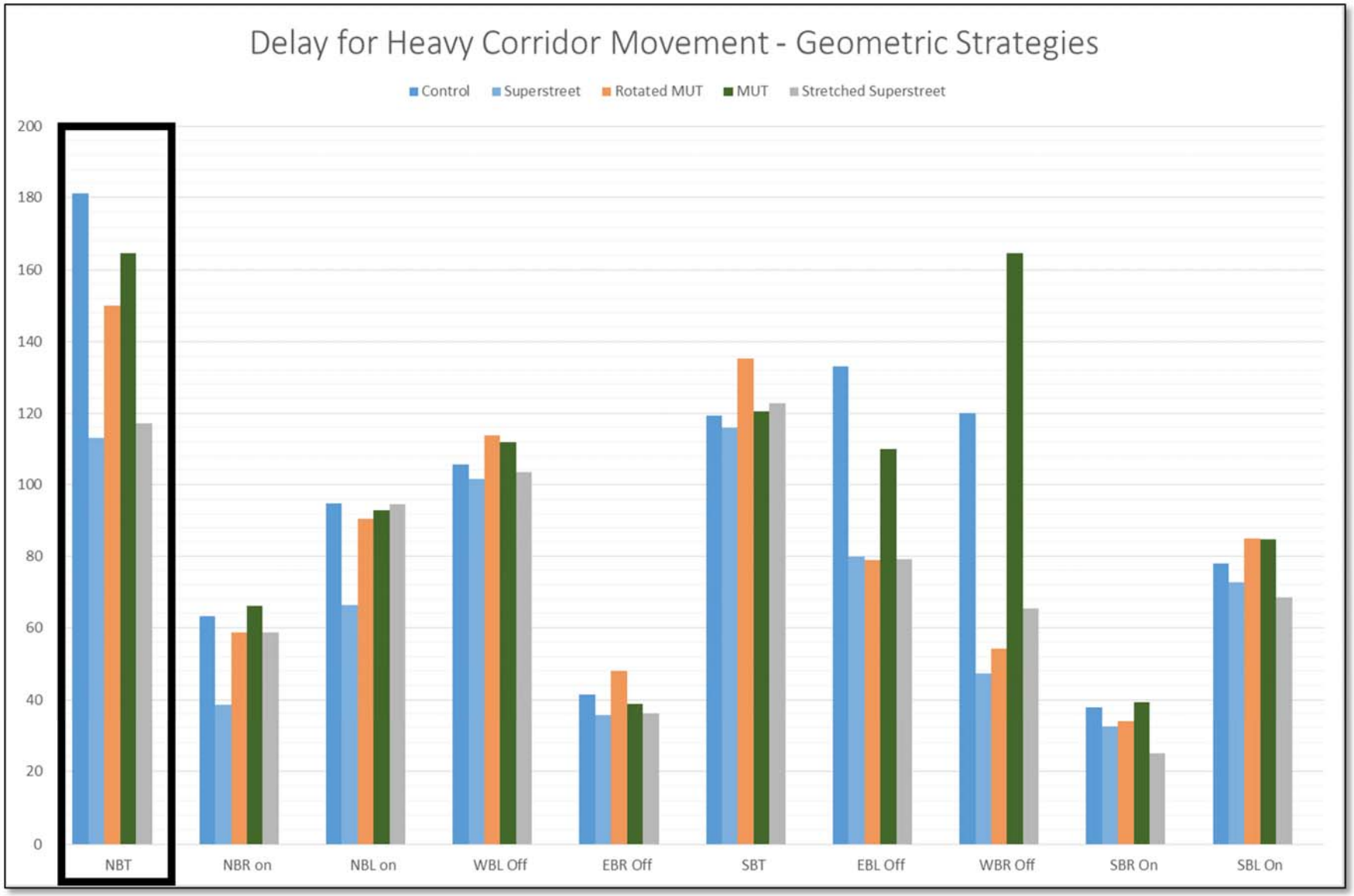
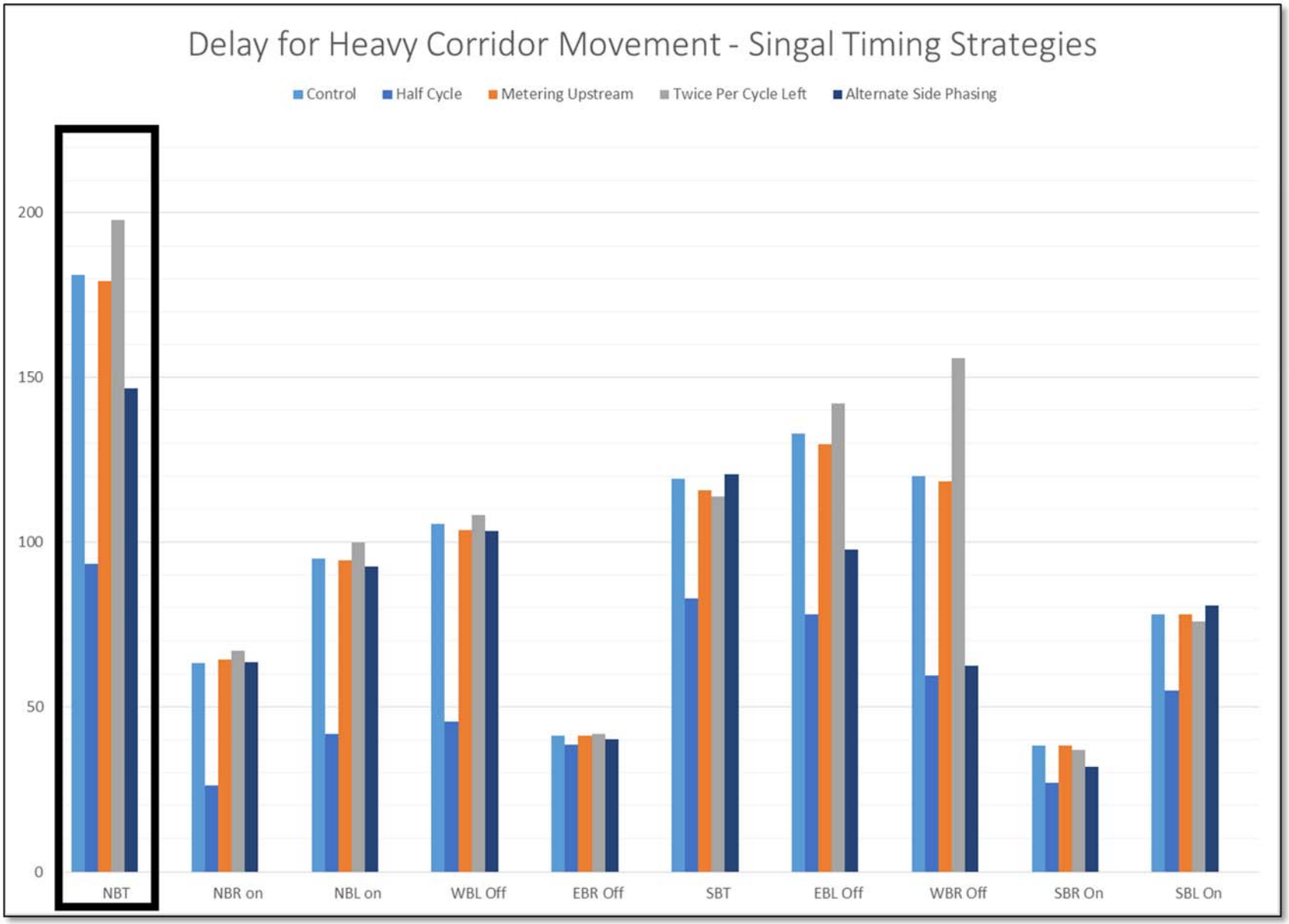
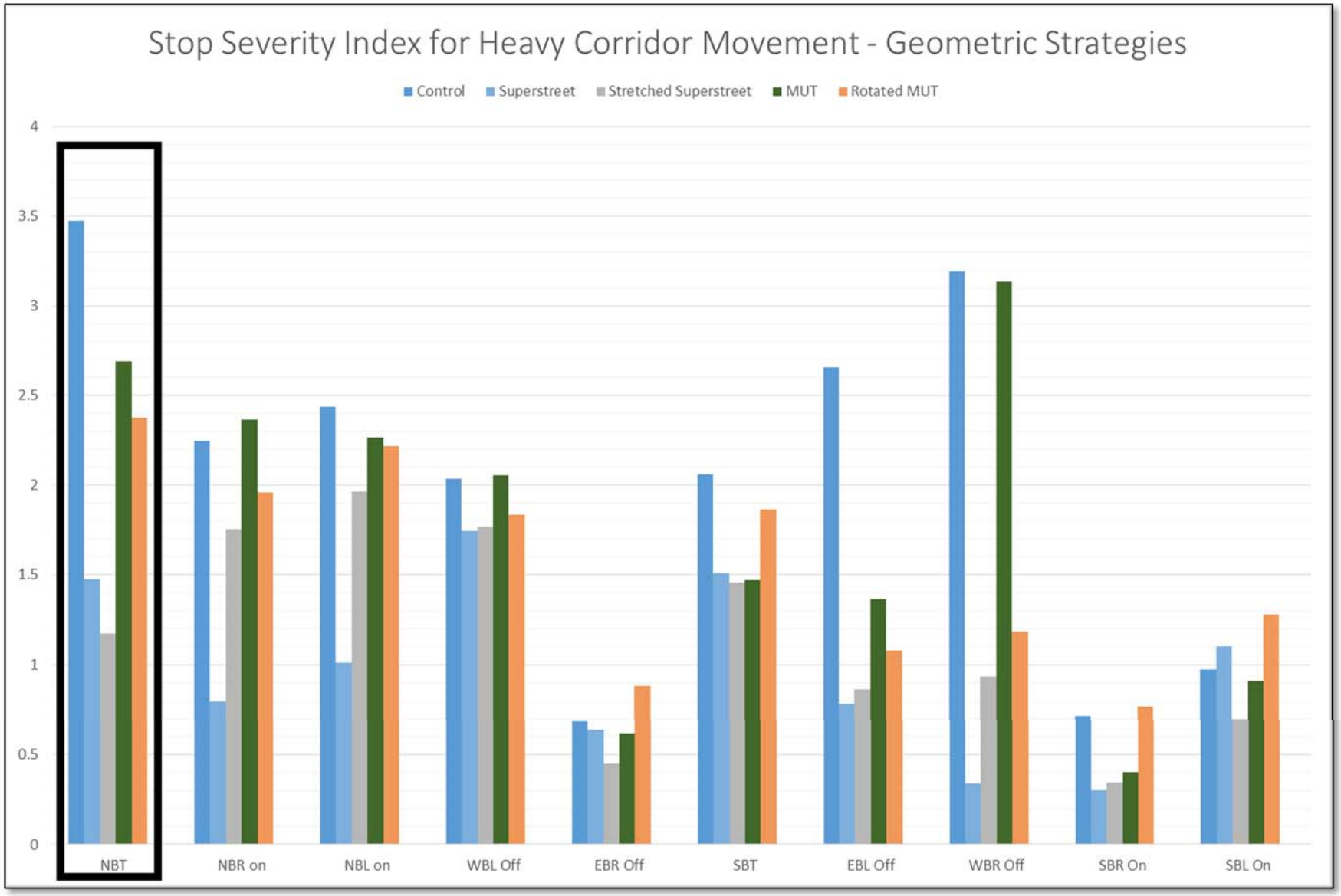
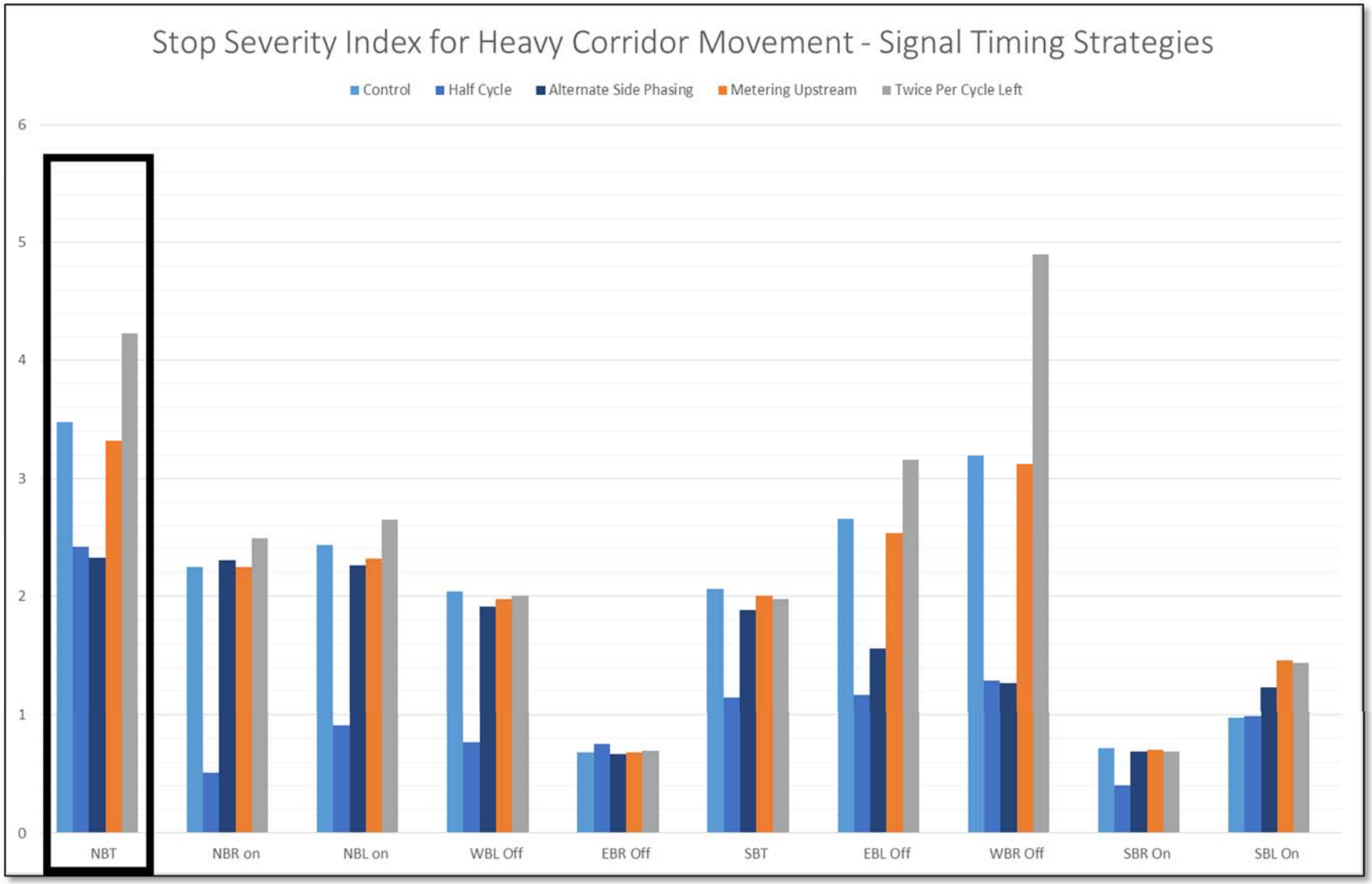
1500
1700
1900
2100
2300
2500
Control
Alternate Side Phasing
Half Cycle
Metering Upstream
Twice Per Cycle Left


Delay for Heavy Corridor Movement - Geometric Strategies
Control
Superstreet
Rotated MUT
MUT
Stretched Superstreet
200
180
160
140
120
100
80
60
40
20
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Delay for Heavy Corridor Movement - Singal Timing Strategies
Control
Half Cycle
Metering Upstream
Twice Per Cycle Left
Alternate Side Phasing
200
150
100
50
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy Corridor Movement - Geometric Strategies
Control
Superstreet
Stretched Superstreet
MUT
Rotated MUT
4
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy Corridor Movement - Signal Timing Strategies
Control
Half Cycle
Alternate Side Phasing
Metering Upstream
Twice Per Cycle Left
6
5
4
3
2
1
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On

	Green Usage for Heavy Corridor Movement															
	Downstream Adjacent				Outbound Crossover				Inbound Crossover				Upstream Adjacent			
	SBT	NBT	NBL	SBL	SBT	NBT	WBR	WBL	SBT	NBT	EBL	EBR	SBT	NBT	NBL	SBL
Control	45%	56%	48%	43%	53%	61%	34%	14%	45%	52%	21%	15%	46%	54%	46%	44%
Alternate Side Phaing	29%	39%	30%	26%	48%	78%	55%	14%	43%	71%	37%	15%	43%	62%	44%	43%
Metering Upstream	45%	56%	47%	43%	53%	62%	36%	14%	44%	51%	20%	15%	45%	53%	46%	44%
Twice Per Cycle Left	45%	56%	47%	44%	53%	66%	39%	14%	46%	54%	21%	15%	46%	54%	45%	44%
Half Cycle	42%	57%	45%	41%	54%	56%	26%	16%	50%	52%	22%	16%	48%	54%	43%	41%
Rotated MUT	41%	53%			52%	52%	21%	14%	45%	50%	20%	15%	44%	53%	45%	46%
MUT	44%	67%	46%	44%	47%	65%	64%	12%	41%	51%	20%	15%	43%	53%	45%	42%
Superstreet					20%	46%	23%	14%	27%	47%	20%	15%	26%	49%	45%	34%
Stretched Superstreet					53%	94%	87%	3%	38%	95%	70%	7%	29%	89%	25%	38%

B.2 Heavy Left Off

Movement of Interest: Eastbound to northbound right turn from the freeway

Volume: 1000 vph

Strategies Tested:

- Alternating side street phasing
- Dedicated phase for the off-ramp movements
- Half cycle at the DDI crossovers
- Metering traffic at the upstream adjacent intersection
- Median U-Turn
- Rotated Median U-Turn
- Stretched superstreet with DDI between the u-turns
- Superstreet
- Twice per cycle left at the downstream adjacent intersection

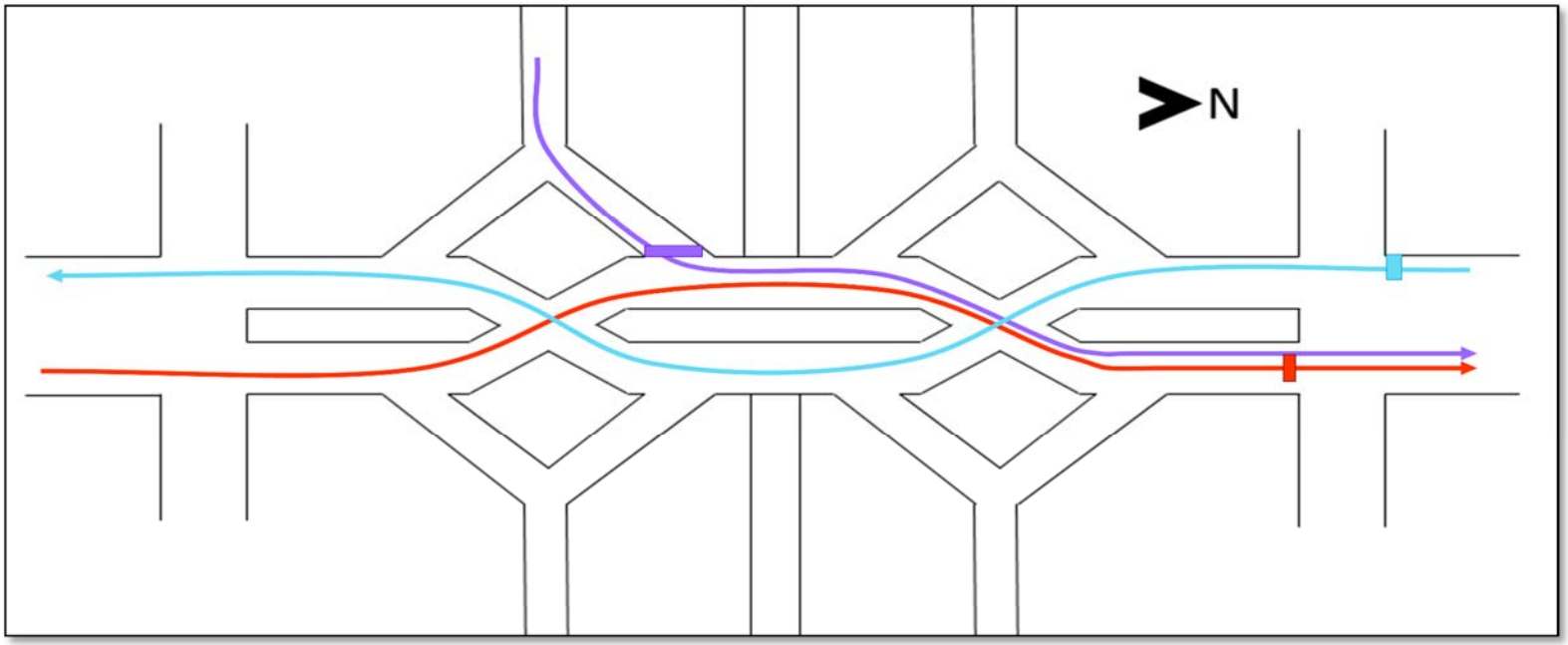
Measures of Effectiveness:

- Average Maximum Queue Length as measured at a movement – the maximum queue length was measured once per cycle and those results were averaged
- Delay as measured over a origin-destination path – this delay includes all times when the vehicle was driving less than the desired speed due to stop, control, and geometric delay
- Stop Severity Index as measured over a path – this measurement considers the number of times a vehicle stopped as well as the number of intersections it traveled through given the following equation $SSI = \frac{\#\ of\ stops^2}{\#\ of\ intersections}$
- Green Usage as measured at a movement – the percentage of time during which the green signal was used as defined by a vehicle passing the stop bar

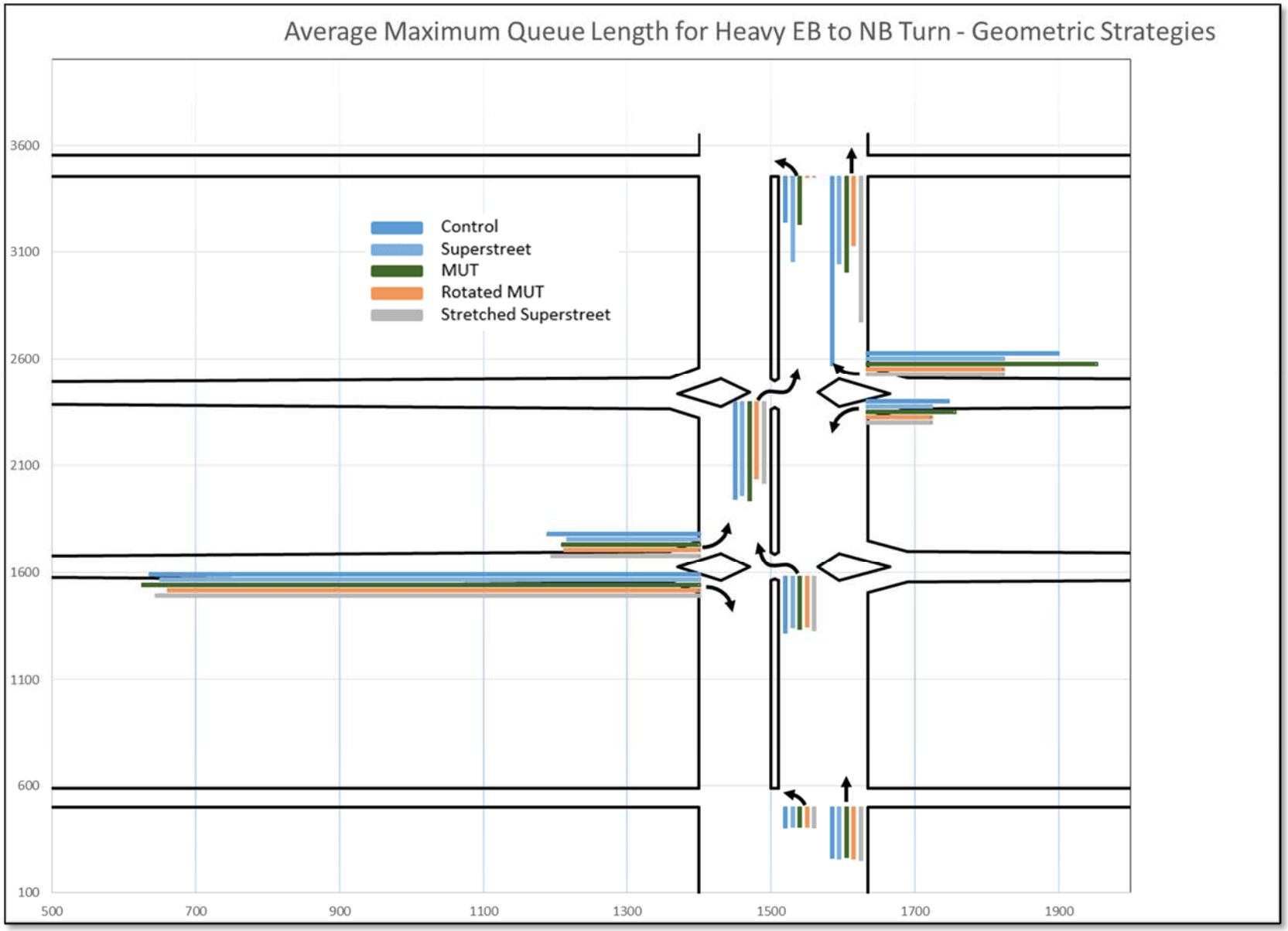
Overview of Results: Presented below is a diagram of a DDI with the movement of interest as well as the two corridor movements. Also included is a table featuring the four measures of effectiveness and their value under the control (no strategy) scenario for each of the movements highlighted in the diagram. Below the control values are the improvements seen in the measure of effectiveness for each strategy relative to the control. An improvement of 50%+ is shaded dark green while an improvement of -50%+ is dark red. The path over which delay and the stop severity index (SSI) is measured is indicated by a line while the movement at which the average maximum queue length and green usage is measured is indicated by a block.

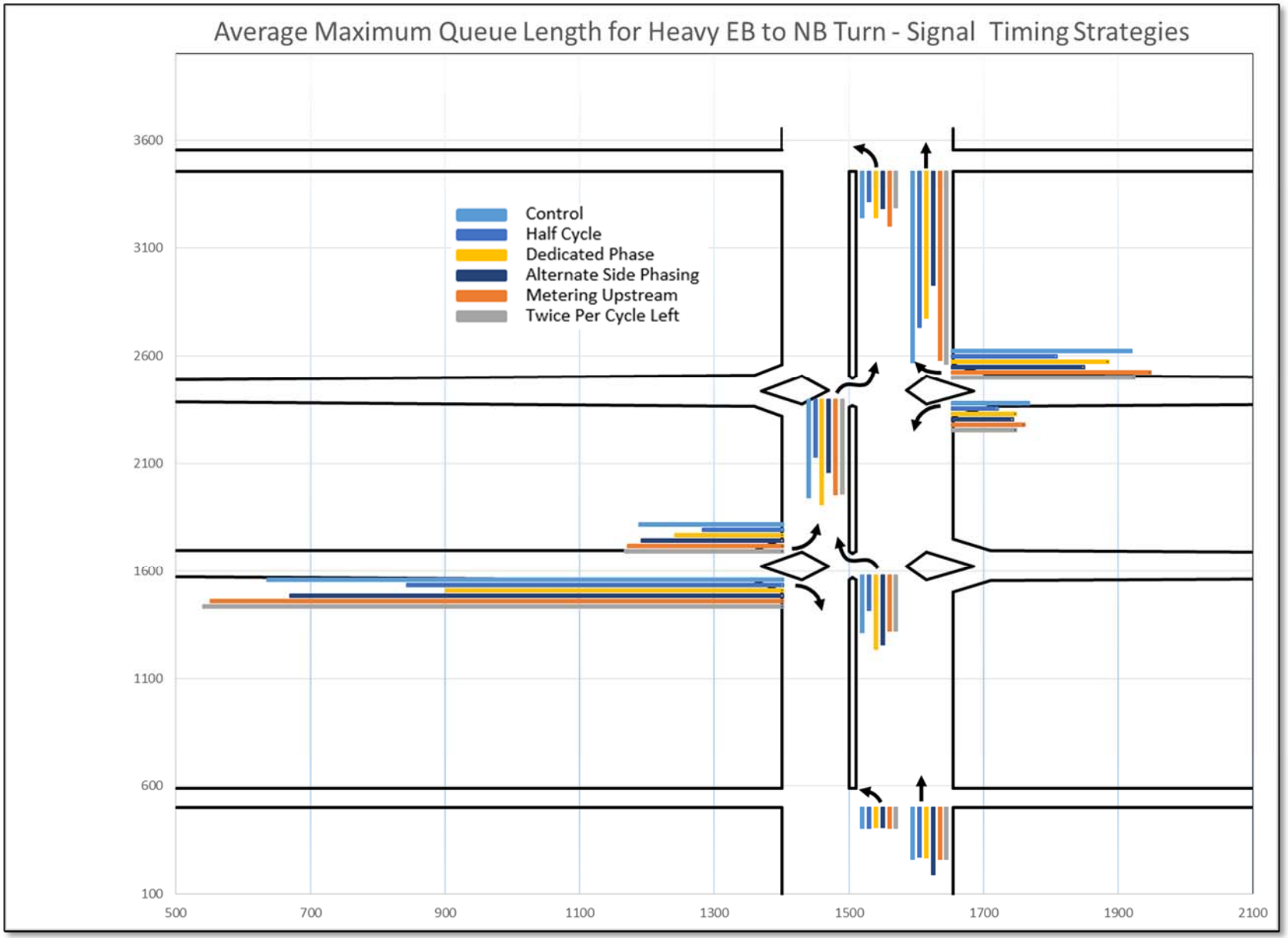
Further Considerations: Below the condensed results, four charts detail the full results for each measure of effectiveness. For the left turn from the freeway it is important to consider the impact to the northbound traffic at the southern crossover. The increased volume at the left turn movement will decrease the amount of green time for the northbound movement. The westbound to northbound right turn from the freeway will be similarly impacted. Additionally, although the movement of interest is left turn, the right turn queue should be considered in conjunction with the left turn. Due to the lane configuration, the queue counter assigns any queue that extends upstream of the left/right turn diverge point to the right turn even if the majority of those vehicles intend to turn left.

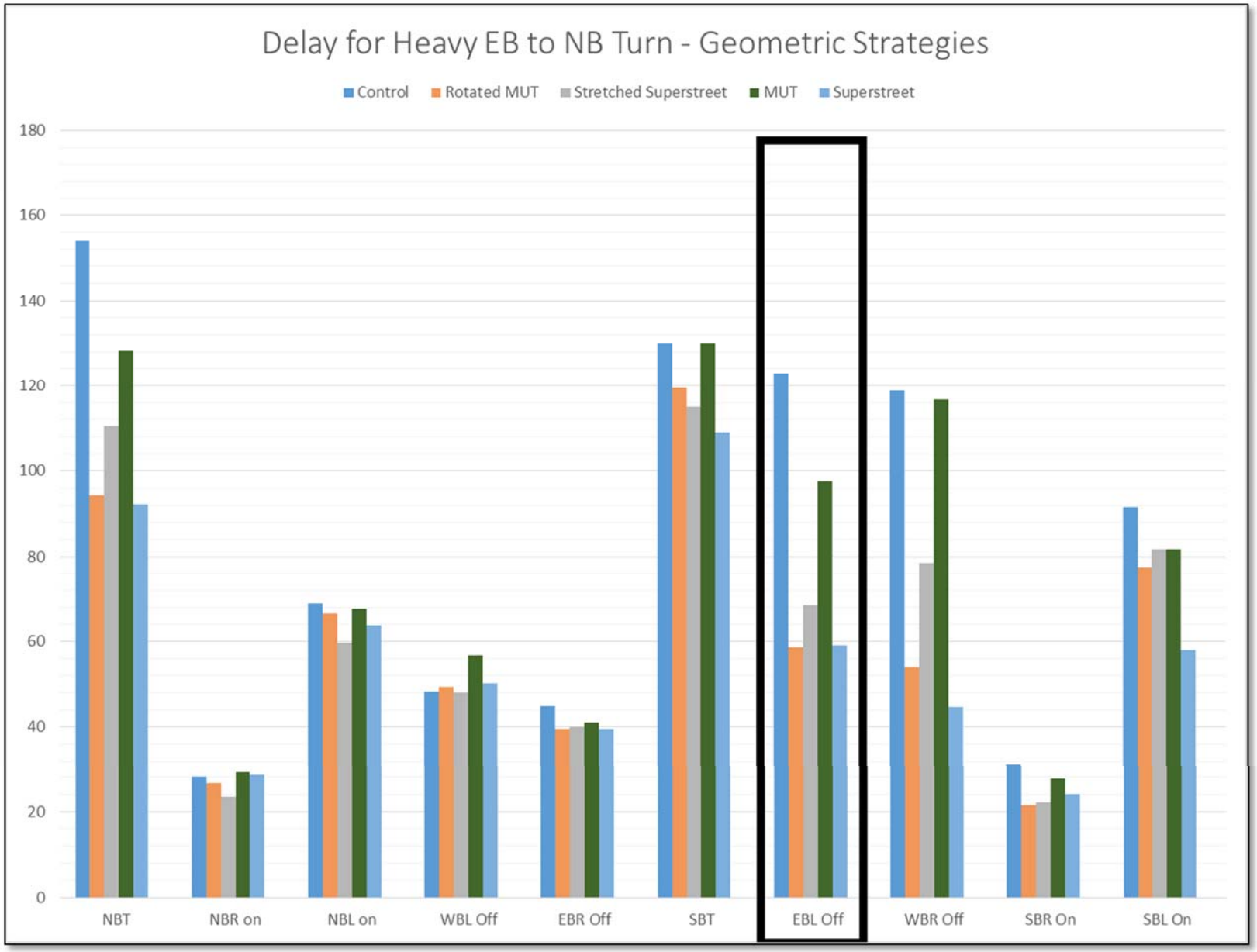
Supplemental Effects: Some of the most effective queue reduction strategies at the off ramp, including the dedicated phasing, result in an increased queue at the northern crossover. The storage capacity on the bridge must be considered when looking at strategies. If not properly coordinated, the reduced delays and stops seen at the off ramp are offset by additional stops and delay at the northern crossover and northern adjacent intersection.

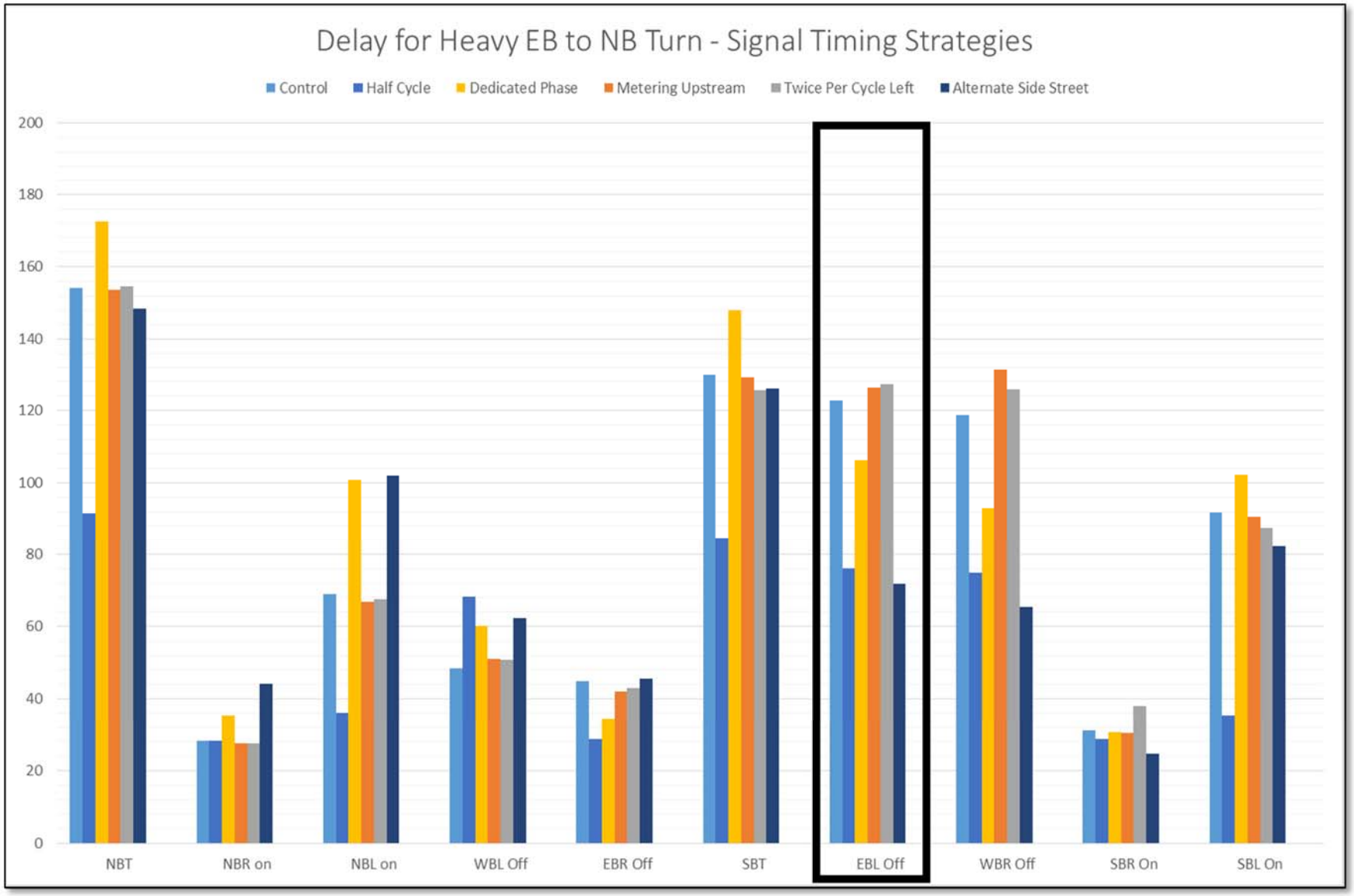


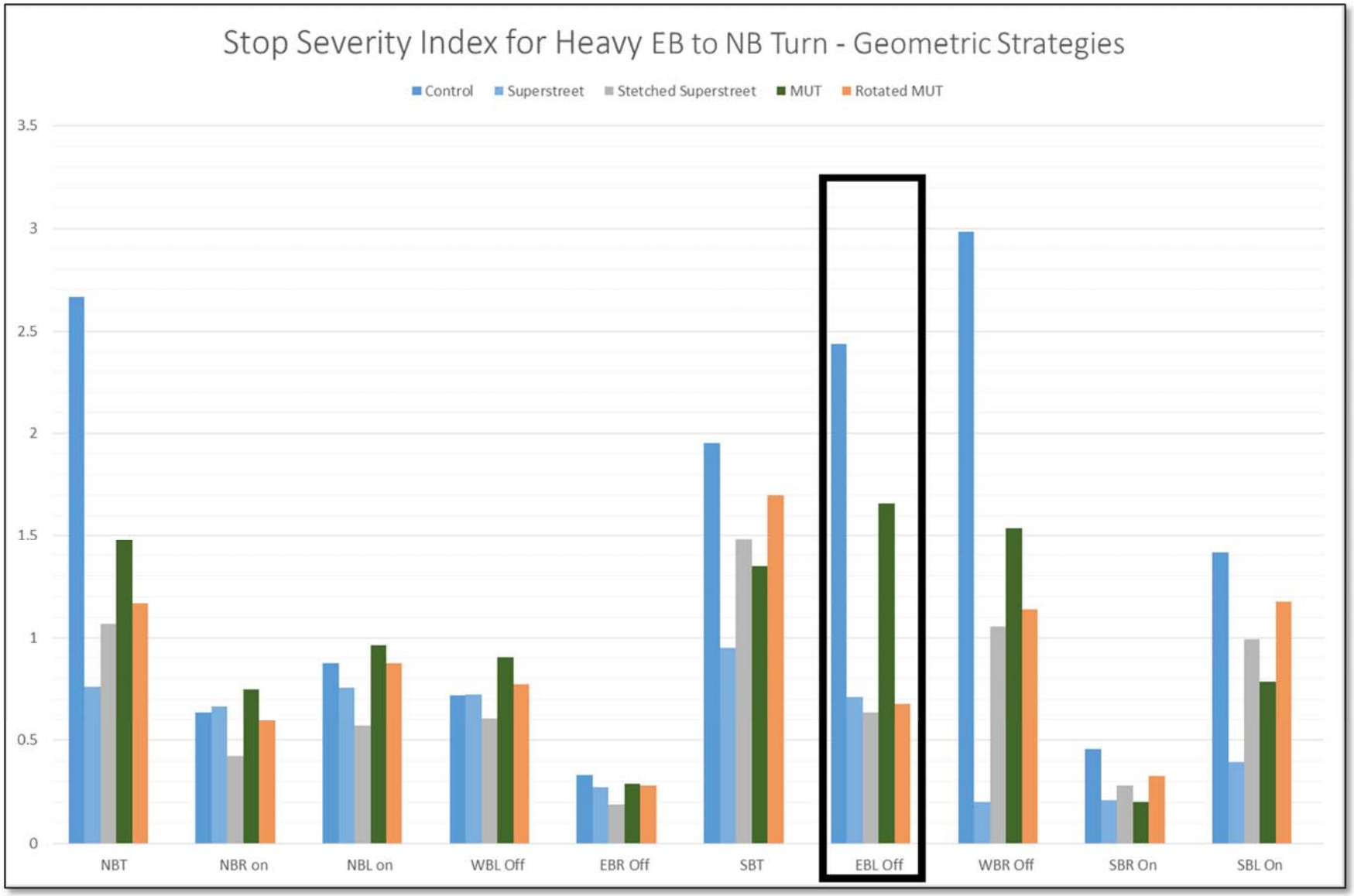
	Left Turn Off of the Freeway				North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	123 seconds	SSI = 2.4	45%	762 ft	154 seconds	SSI = 2.7	36%	882 ft	130 seconds	SSI = 2.0	37%
Alternating Side Street	0.58	0.43	1.14	0.96	0.96	0.77	0.59	0.50	0.97	0.86	0.62
Ded. Off Ramp Phase	0.87	0.75	1.05	0.65	1.12	1.06	0.98	0.77	1.14	1.61	0.69
Half Cycle	0.62	0.55	1.05	0.73	0.59	0.65	0.93	0.81	0.65	0.65	0.90
Metered	1.03	1.09	1.02	1.11	1.00	0.98	1.00	0.99	0.99	0.98	0.99
MUT	0.79	0.68	1.04	1.01	0.83	0.56	1.10	0.50	1.00	0.69	0.96
Rotated MUT	0.48	0.28	0.99	0.97	0.61	0.44	0.94	0.36	0.92	0.49	0.80
Stretched Superstreet	0.56	0.26	1.00	0.99	0.72	0.40	-	0.77	0.89	0.87	-
Superstreet	0.48	0.29	1.12	0.98	0.60	0.29	-	0.46	0.84	0.76	-
Twice Per Cycle Left	1.04	1.11	1.05	1.12	1.00	1.00	0.98	1.01	0.97	1.03	0.87

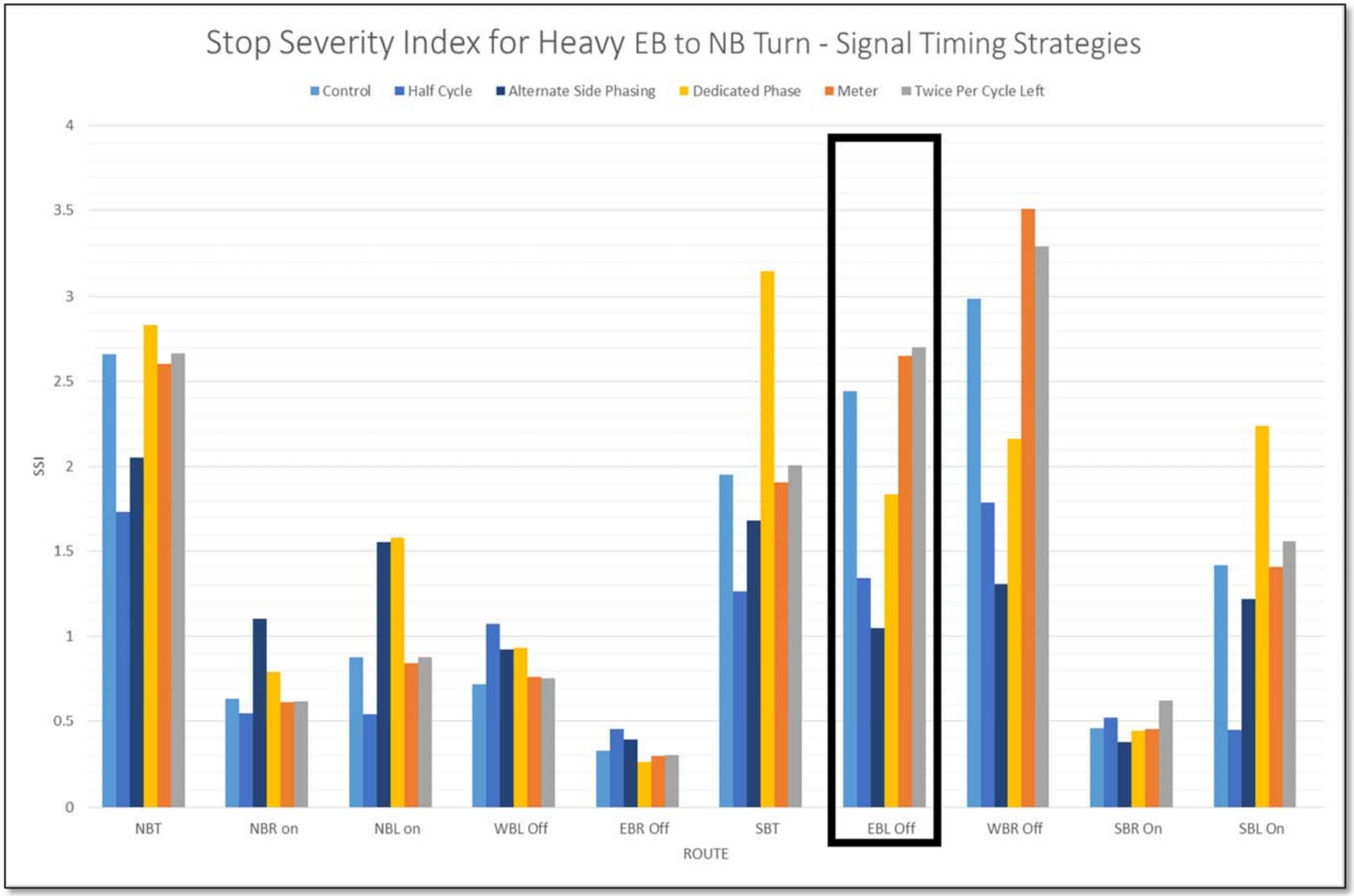

Average Maximum Queue Length for Heavy EB to NB Turn - Geometric Strategies
Control
Superstreet
MUT
Rotated MUT
Stretched Superstreet
3600
3100
2600
2100
1600
1100
600
100
500
700
900
1100
1300
1500
1700
1900


Average Maximum Queue Length for Heavy EB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Dedicated Phase
Alternate Side Phasing
Metering Upstream
Twice Per Cycle Left
3600
3100
2600
2100
1600
1100
600
100
500
700
900
1100
1300
1500
1700
1900
2100


Delay for Heavy EB to NB Turn - Geometric Strategies
Control
Rotated MUT
Stretched Superstreet
MUT
Superstreet
180
160
140
120
100
80
60
40
20
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Delay for Heavy EB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Dedicated Phase
Metering Upstream
Twice Per Cycle Left
Alternate Side Street
200
180
160
140
120
100
80
60
40
20
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy EB to NB Turn - Geometric Strategies
Control
Superstreet
Stetched Superstreet
MUT
Rotated MUT
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy EB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Alternate Side Phasing
Dedicated Phase
Meter
Twice Per Cycle Left
SSI
4
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On
ROUTE

	Green Usage for Heavy EB to NB Turn															
	Downstream Adjacent				Outbound Crossover				Inbound Crossover				Upstream Adjacent			
	SBT	NBT	NBL	SBL	SBT	NBT	WBR	WBL	SBT	NBT	EBL	EBR	SBT	NBT	NBL	SBL
Control	37%	58%	36%	39%	48%	63%	37%	12%	29%	61%	45%	27%	37%	36%	39%	44%
Half Cycle	33%	55%	39%	39%	52%	57%	32%	13%	32%	57%	48%	26%	38%	36%	41%	42%
Dedicated Phase	25%	57%	37%	39%	46%	69%	43%	13%	33%	64%	48%	24%	36%	36%	40%	44%
Twice Per Cycle Left	32%	57%	49%	40%	47%	69%	42%	12%	30%	59%	48%	25%	37%	36%	39%	43%
Metering Upstream	36%	58%	37%	39%	46%	65%	39%	12%	30%	59%	46%	25%	37%	36%	39%	43%
Alternate Side Phasing	23%	43%	30%	24%	49%	71%	53%	12%	30%	57%	52%	24%	46%	46%	39%	41%
MUT	35%	64%	47%	38%	48%	63%	55%	10%	27%	58%	47%	26%	35%	37%	40%	41%
Superstreet					52%	73%	66%	8%	25%	57%	51%	25%	33%	37%	40%	38%
Rotated MUT	29%	55%			50%	49%	26%	12%	31%	58%	45%	26%	37%	37%	39%	43%
Stretched Super					47%	54%	26%	12%	29%	59%	46%	26%	36%	37%	40%	41%

B.3 Heavy Right Off

Movement of Interest: Westbound to northbound right turn from the freeway

Volume: 1000 vph

Strategies Tested:

- Acceleration lane
- Alternating side street phasing
- Dedicated phase for the off-ramp movements
- Dual right turn lanes
- Dual right turn lanes with right-turn-on-red allowed
- Half cycle at the DDI crossovers
- Metering traffic at the upstream adjacent intersection
- Median U-Turn
- Right-turn-on-red allowed
- Rotated Median U-Turn
- Slip lane for westbound-northbound-eastbound right turning traffic
- Stretched superstreet with DDI between the u-turns
- Superstreet
- Twice per cycle left at the downstream adjacent intersection

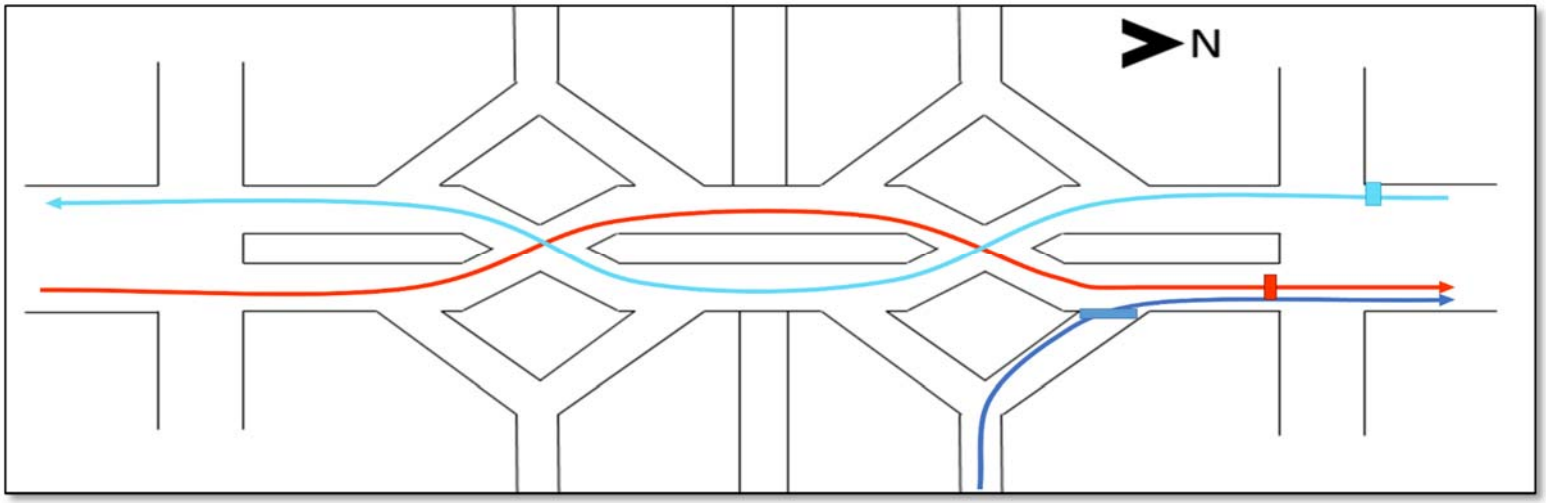
Measures of Effectiveness:

- Average Maximum Queue Length as measured at a movement – the maximum queue length was measured once per cycle and those results were averaged
- Delay as measured over a origin-destination path – this delay includes all times when the vehicle was driving less than the desired speed due to stop, control, and geometric delay
- Stop Severity Index as measured over a path – this measurement considers the number of times a vehicle stopped as well as the number of intersections it traveled through given the following equation $SSI = \frac{\#\ of\ stops^2}{\#\ of\ intersections}$
- Green Usage as measured at a movement – the percentage of time during which the green signal was used as defined by a vehicle passing the stop bar

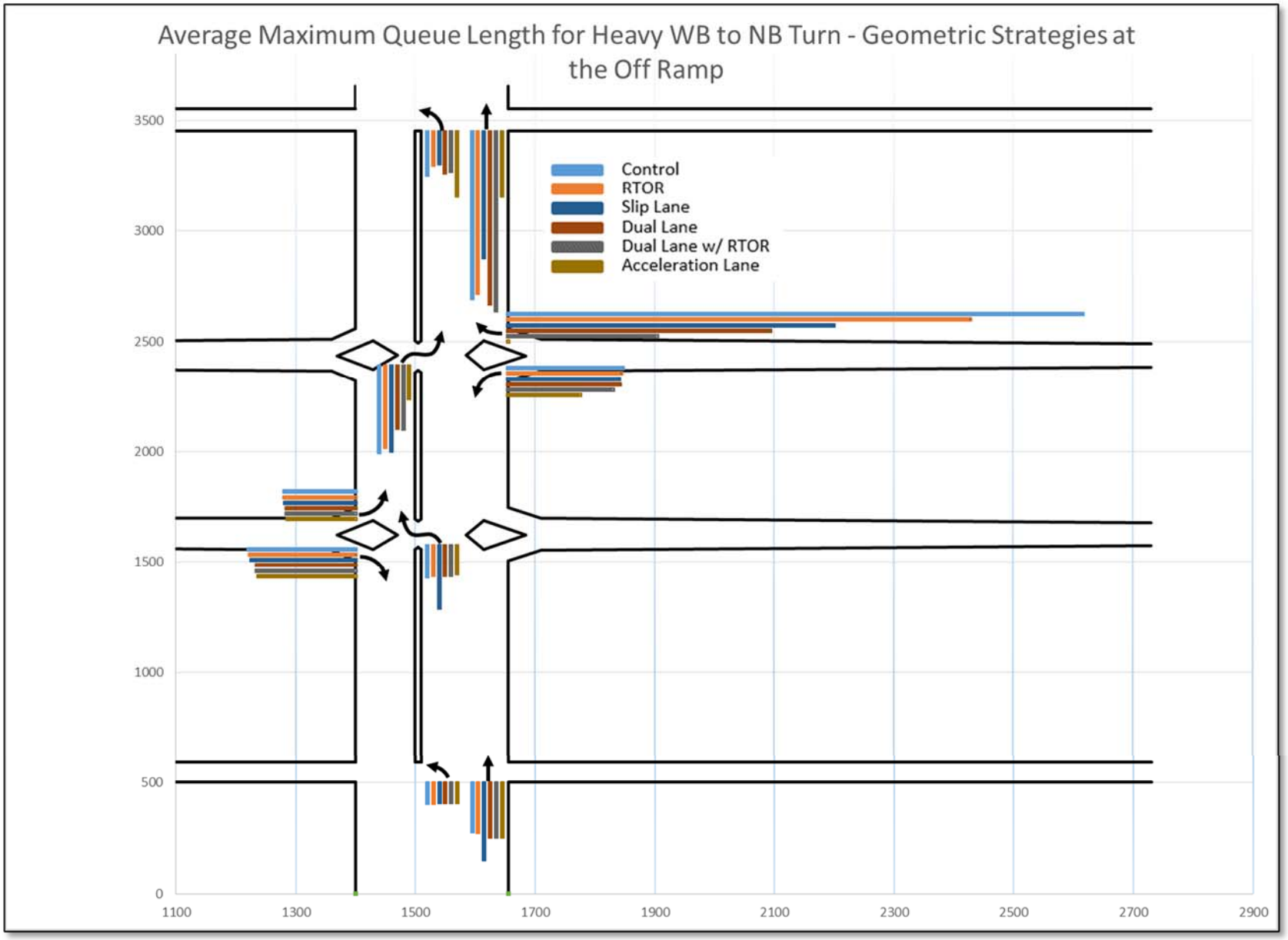
Overview of Results: Presented below is a diagram of a DDI with the movement of interest as well as the two corridor movements. Also included is a table featuring the four measures of effectiveness and their value under the control (no strategy) scenario for each of the movements highlighted in the diagram. Below the control values are the improvements seen in the measure of effectiveness for each strategy relative to the control. An improvement of 50%+ is shaded dark green while an improvement of -50%+ is dark red. The path over which delay and the stop severity index (SSI) is measured is indicated by a line while the movement at which the average maximum queue length and green usage is measured is indicated by a block.

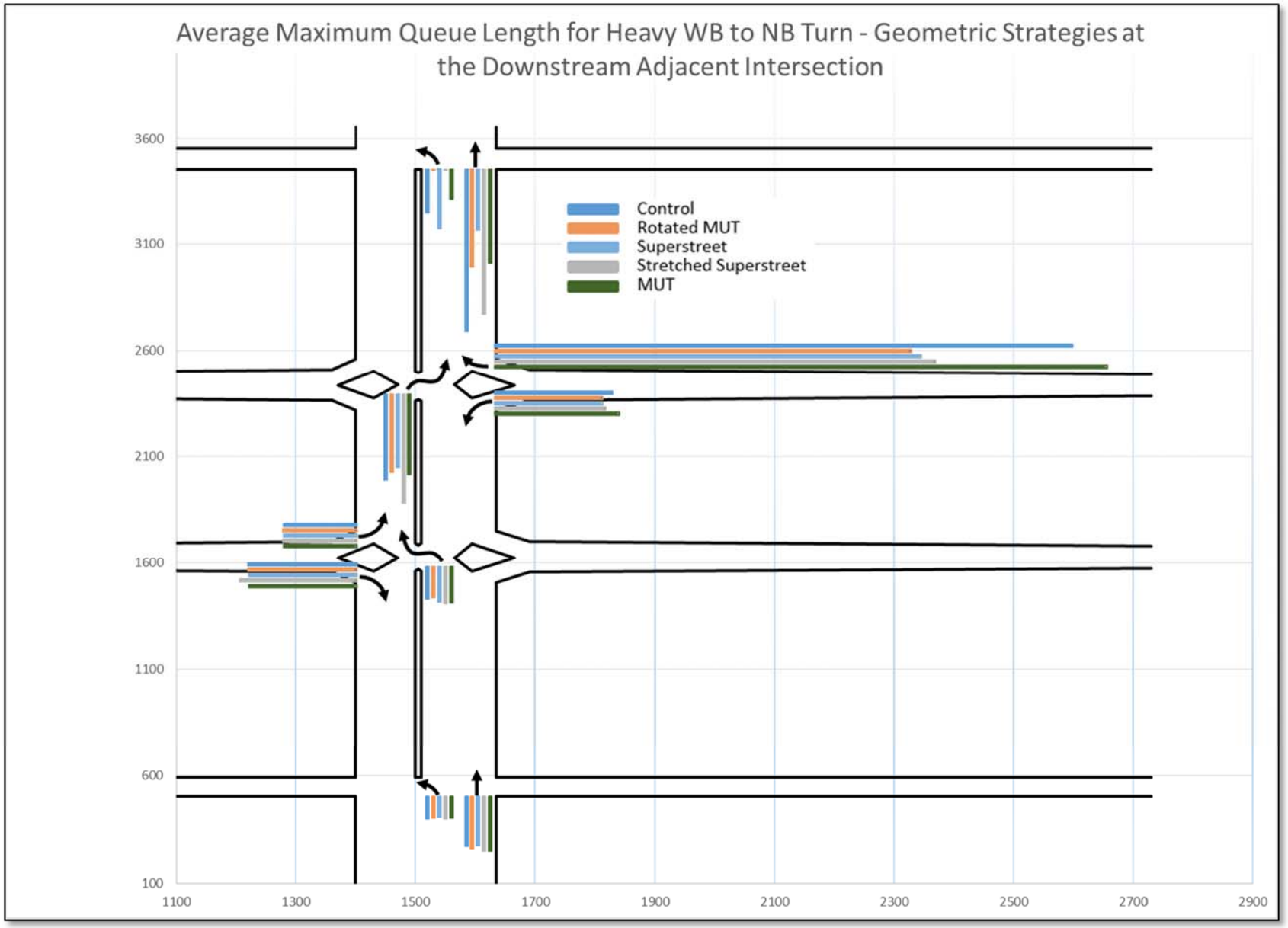
Further Considerations: Below the condensed results, four charts detail the full results for each measure of effectiveness. For the right turn from the freeway, it is important to consider the queue impact to the northbound through movement at the northern crossover because the increase in volume for the right turn will reduce the green time for that crossover movement.

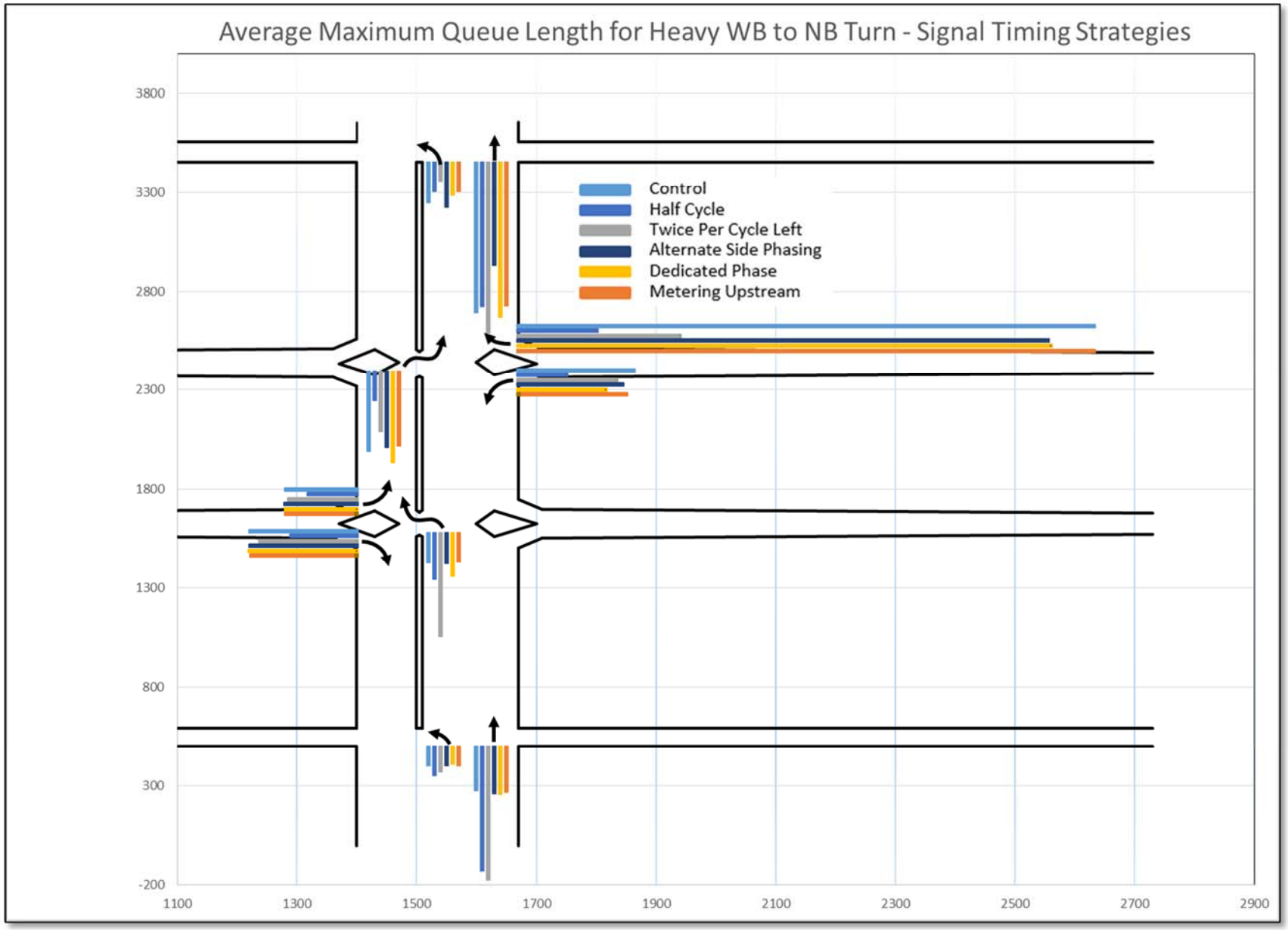
Supplemental Effects: While most of the strategies improved the MOEs for the west bound to northbound turn and the northbound corridor movement, the southbound corridor movement was generally damaged. The acceleration lane strategy was one of the most effective for both the turning movement and northern corridor movement, but this may, at least in part, be due to the additional northbound through lane it provides at the northern intersection. Some strategies, such as the dual turn lanes, or allowing RTOR shows promise for the turning movement, but may tend to funnel vehicles faster to the northern adjacent intersection resulting in increased queue storage problems at that intersection.

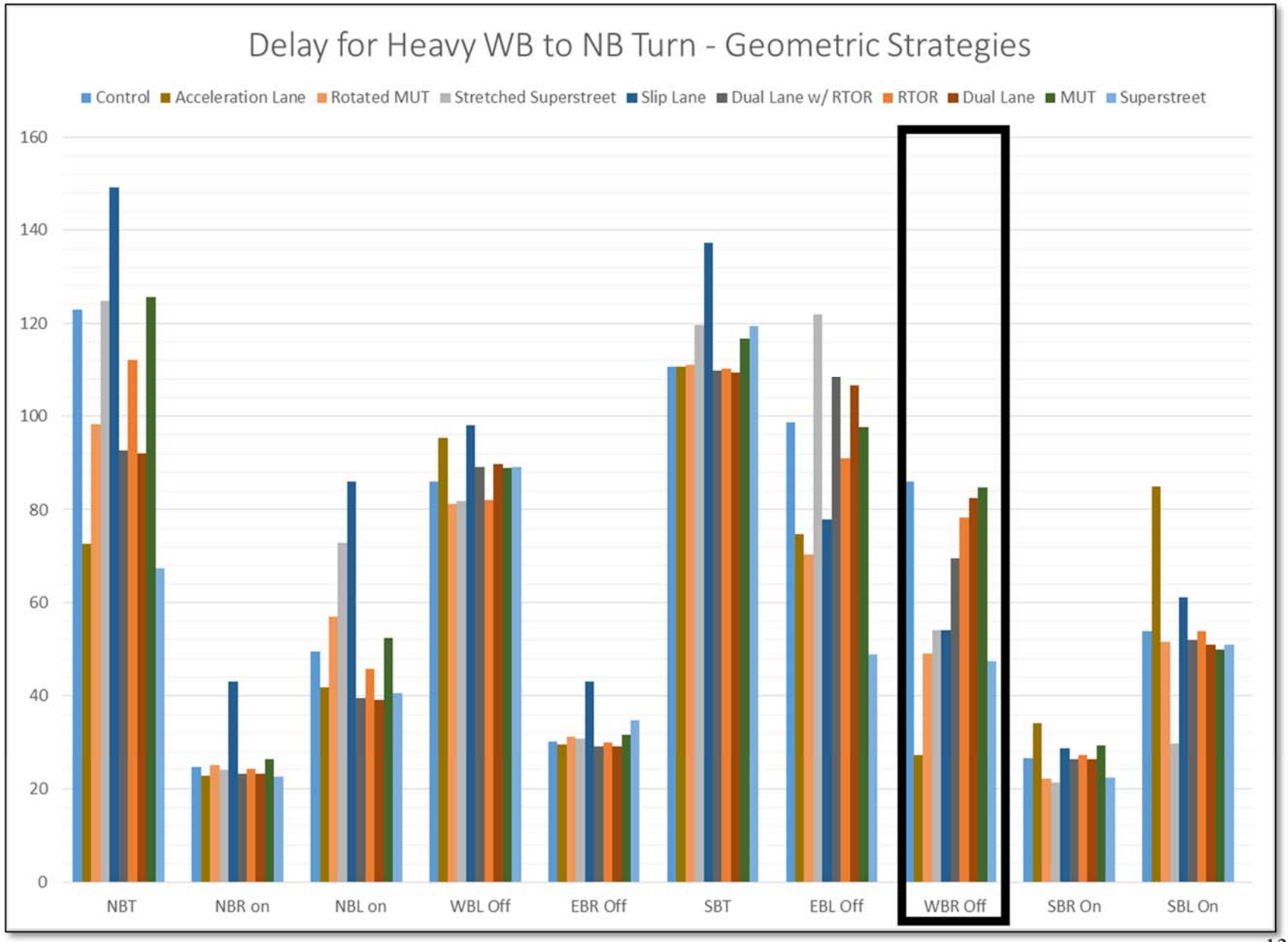


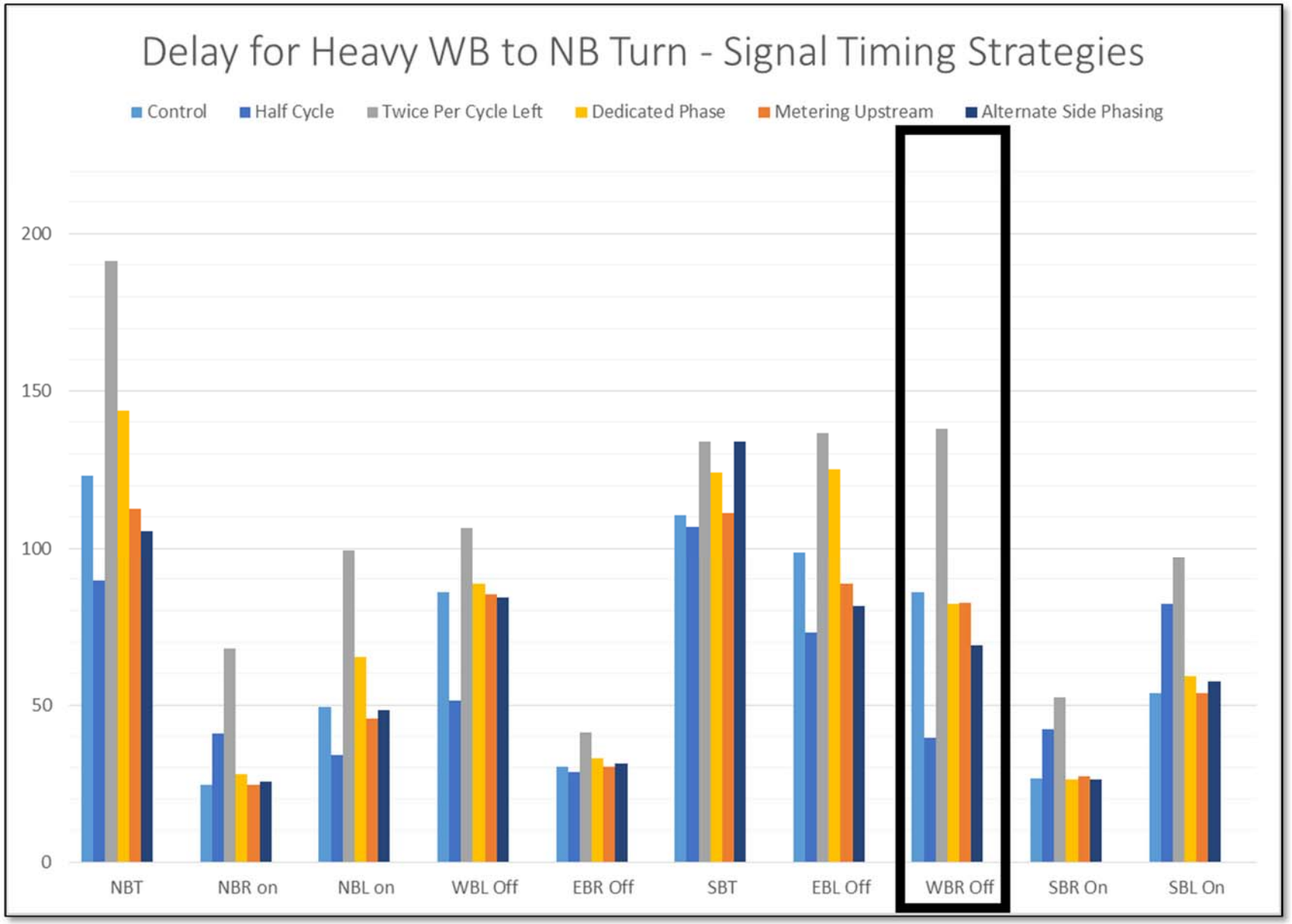
	Right Turn Off of the Freeway				North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	86 seconds	SSI = 1.5	53%	961 ft	123 seconds	SSI = 2.3	40%	759 ft	111 seconds	SSI = 1.9	32%
Accel. Lane	0.32	0.14	0.54	-	0.59	0.34	1.13	0.39	1.00	0.96	1.12
Alternating Side Street	0.80	0.80	0.96	0.92	0.86	0.69	1.35	0.69	1.21	1.15	1.08
Ded. Off Ramp Phase	0.95	0.86	0.97	0.93	1.17	1.24	1.44	1.03	1.12	1.09	0.99
Dual RT Lanes	0.96	1.09	1.10	0.46	0.75	0.66	1.44	1.03	0.99	0.98	0.98
Dual RT Lanes & RTOR	0.81	1.46	1.01	0.26	0.75	0.66	1.45	1.07	0.99	0.98	0.67
Half Cycle	0.46	0.39	1.01	0.14	0.73	0.59	1.50	0.96	0.96	1.06	1.04
Metered	0.96	0.89	0.98	1.00	0.91	0.91	1.46	0.95	1.01	1.01	1.00
MUT	0.99	0.82	1.03	1.06	1.02	0.78	1.68	0.58	1.05	0.79	1.10
Right Turn On Red	0.91	0.92	1.01	0.80	0.91	0.91	1.46	0.97	1.00	1.01	1.01
Rotated MUT	0.57	0.40	0.84	0.72	0.80	0.81	1.47	0.60	1.00	0.95	0.91
Slip Lane	0.63	0.44	0.82	0.57	1.21	1.41	1.43	0.76	1.24	1.11	1.04
Stretched Superstreet	0.63	0.36	0.86	0.76	1.01	0.84	-	0.90	1.08	0.79	-
Superstreet	0.55	0.35	1.56	0.74	0.55	0.25	-	0.38	1.08	0.68	-
Twice Per Cycle Left	1.60	2.64	0.94	0.28	1.56	1.66	1.41	1.14	1.21	1.20	0.97

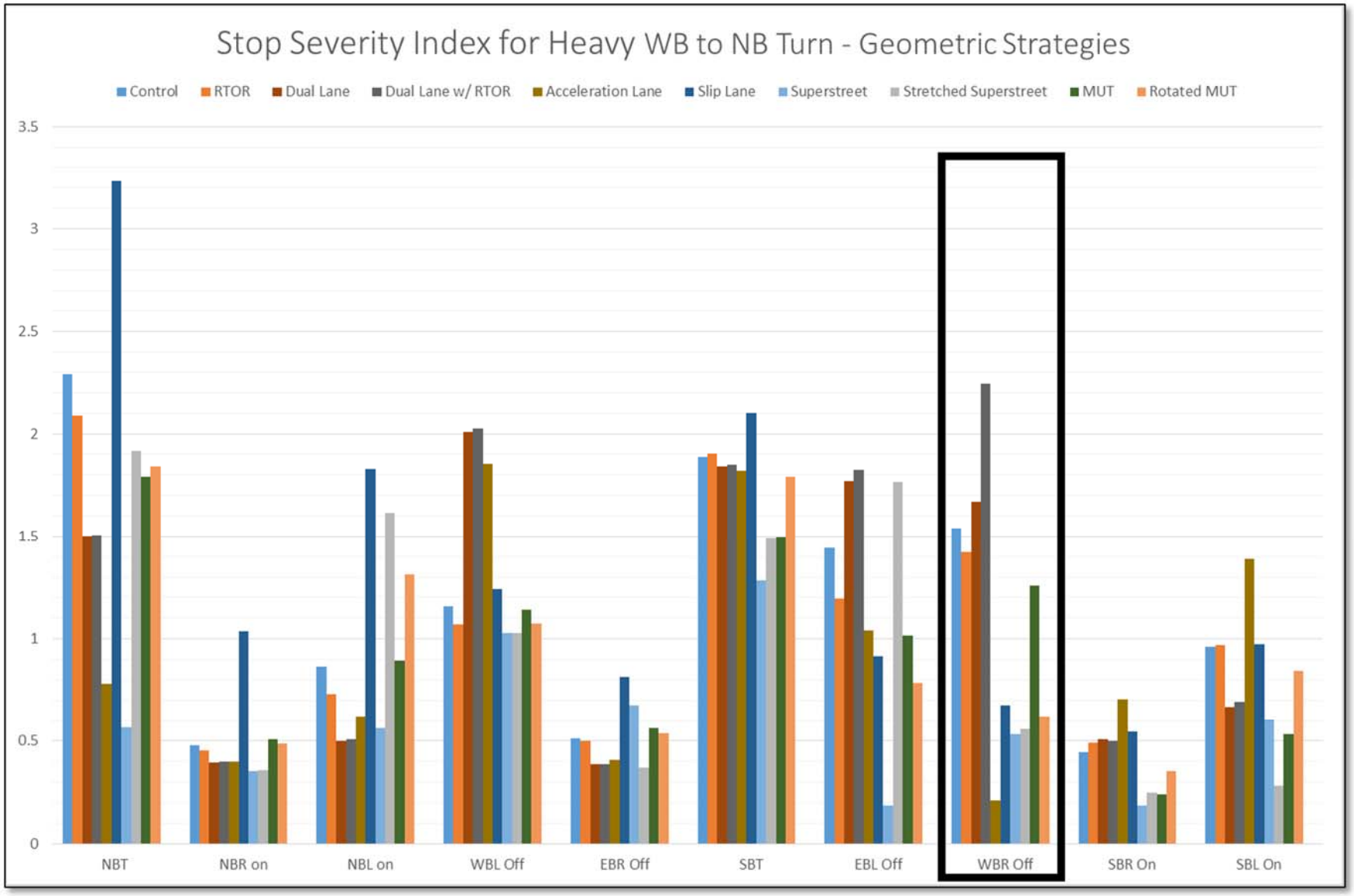

Average Maximum Queue Length for Heavy WB to NB Turn - Geometric Strategies at the Off Ramp
Control
RTOR
Slip Lane
Dual Lane
Dual Lane w/ RTOR
Acceleration Lane
3500
3000
2500
2000
1500
1000
500
0
1100
1300
1500
1700
1900
2100
2300
2500
2700
2900

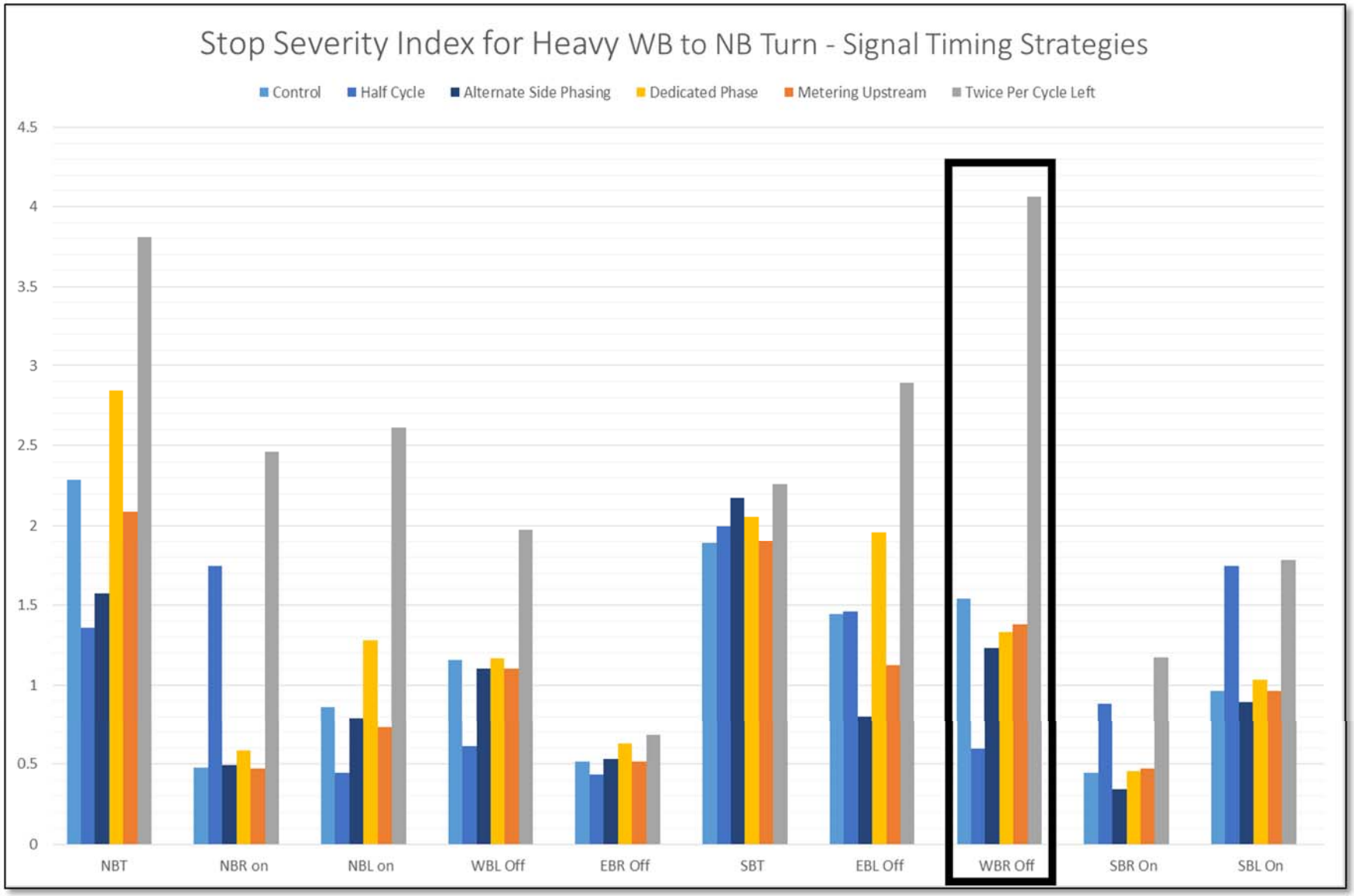

Average Maximum Queue Length for Heavy WB to NB Turn - Geometric Strategies at the Downstream Adjacent Intersection
Control
Rotated MUT
Superstreet
Stretched Superstreet
MUT
3600
3100
2600
2100
1600
1100
600
100
1100
1300
1500
1700
1900
2100
2300
2500
2700
2900


Average Maximum Queue Length for Heavy WB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Twice Per Cycle Left
Alternate Side Phasing
Dedicated Phase
Metering Upstream
3800
3300
2800
2300
1800
1300
800
300
-200
1100
1300
1500
1700
1900
2100
2300
2500
2700
2900


Delay for Heavy WB to NB Turn - Geometric Strategies
Control
Acceleration Lane
Rotated MUT
Stretched Superstreet
Slip Lane
Dual Lane w/ RTOR
RTOR
Dual Lane
MUT
Superstreet
160
140
120
100
80
60
40
20
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Delay for Heavy WB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Twice Per Cycle Left
Dedicated Phase
Metering Upstream
Alternate Side Phasing
200
150
100
50
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy WB to NB Turn - Geometric Strategies
Control
RTOR
Dual Lane
Dual Lane w/ RTOR
Acceleration Lane
Slip Lane
Superstreet
Stretched Superstreet
MUT
Rotated MUT
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy WB to NB Turn - Signal Timing Strategies
Control
Half Cycle
Alternate Side Phasing
Dedicated Phase
Metering Upstream
Twice Per Cycle Left
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On

	Green Usage for Heavy WB to NB Turn															
	North Adjacent				North Crossover				South Crossover				South Adjacent			
	SBT	NBT	NBL	SBL	SBT	NBT	WBR	WBL	SBT	NBT	EBL	EBR	SBT	NBT	NBL	SBL
Control	32%	40%	47%	37%	29%	61%	53%	27%	42%	36%	22%	15%	34%	35%	40%	43%
Dedicated Phase	32%	57%	47%	36%	35%	65%	52%	18%	40%	34%	23%	15%	34%	36%	41%	41%
Twice Per Cycle Left	31%	56%	49%	41%	30%	60%	50%	26%	42%	36%	21%	15%	34%	35%	40%	42%
Alternate Side Phasing	34%	53%	28%	24%	28%	61%	51%	26%	42%	36%	21%	15%	34%	36%	40%	42%
Metering Upstream	32%	58%	47%	37%	30%	60%	52%	25%	42%	36%	21%	15%	33%	35%	40%	42%
RTOR	32%	58%	46%	37%	30%	60%	53%	26%	42%	36%	21%	15%	34%	35%	40%	42%
Acceleration Lane	36%	45%	36%	39%	40%	23%	29%	14%	39%	19%	20%	15%	32%	13%	0%	43%
Half Cycle	33%	59%	48%	37%	34%	56%	54%	25%	42%	39%	23%	16%	37%	37%	40%	41%
Slip Lane	33%	57%	39%	39%	31%	57%	44%	24%	40%	32%	20%	15%	46%	45%	39%	42%
Dual Lane	31%	57%	48%	36%	35%	46%	58%	18%	34%	36%	20%	15%	34%	35%	40%	42%
Dual Lane w/ RTOR	21%	57%	48%	36%	35%	46%	54%	17%	34%	36%	20%	15%	34%	35%	40%	42%
MUT	35%	67%	46%	38%	29%	63%	55%	24%	41%	37%	21%	15%	34%	36%	40%	42%
Superstreet					25%	94%	83%	4%	25%	96%	76%	5%	17%	86%	22%	29%
Rotated MUT	29%	58%			30%	59%	45%	26%	42%	36%	22%	15%	34%	36%	40%	42%
Stretched Super					28%	61%	46%	26%	42%	38%	33%	15%	34%	36%	40%	39%

B.4 Heavy Left Turn to the On-Ramp

Movement of Interest: Southbound to eastbound turn onto the freeway

Volume: 1000 vph

Strategies Tested:

- Half cycle at the DDI crossovers
- Median U-Turn
- Rotated Median U-Turn
- Stretched superstreet with DDI between the u-turns
- Superstreet

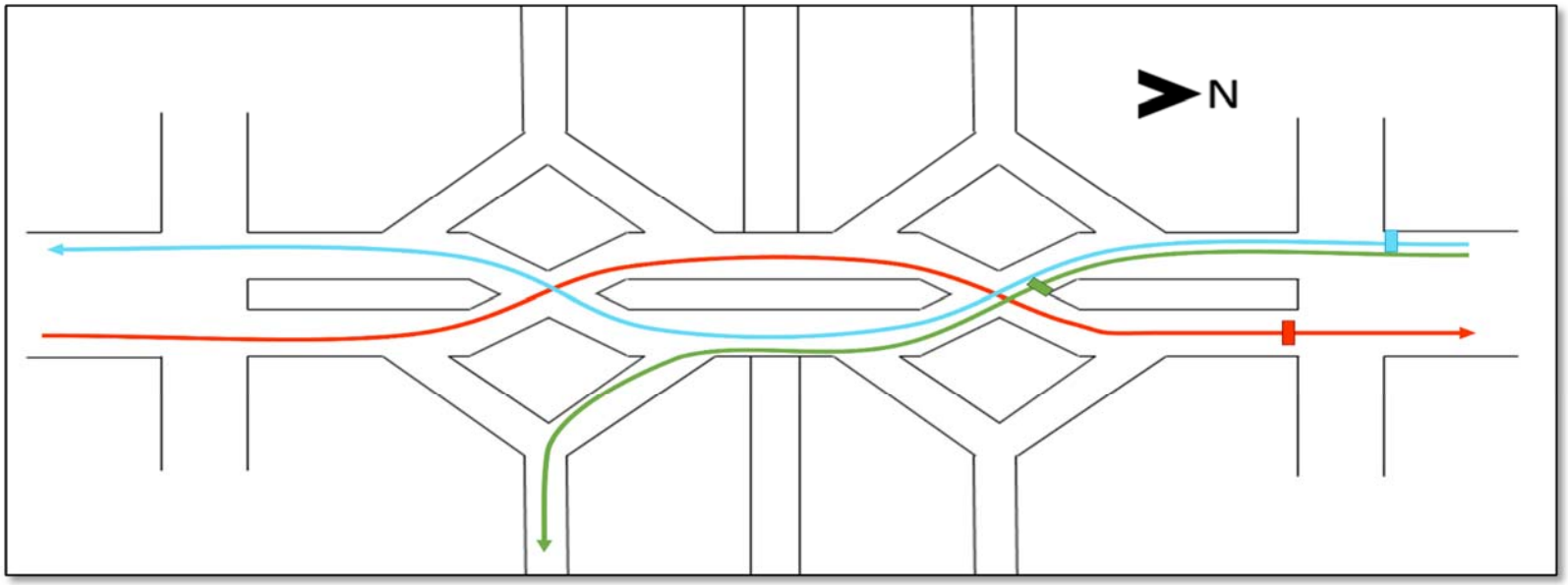
Measures of Effectiveness:

- Average Maximum Queue Length as measured at a movement – the maximum queue length was measured once per cycle and those results were averaged
- Delay as measured over a origin-destination path – this delay includes all times when the vehicle was driving less than the desired speed due to stop, control, and geometric delay
- Stop Severity Index as measured over a path – this measurement considers the number of times a vehicle stopped as well as the number of intersections it traveled through given the following equation $SSI = \frac{\#\ of\ stops^2}{\#\ of\ intersections}$
- Green Usage as measured at a movement – the percentage of time during which the green signal was used as defined by a vehicle passing the stop bar

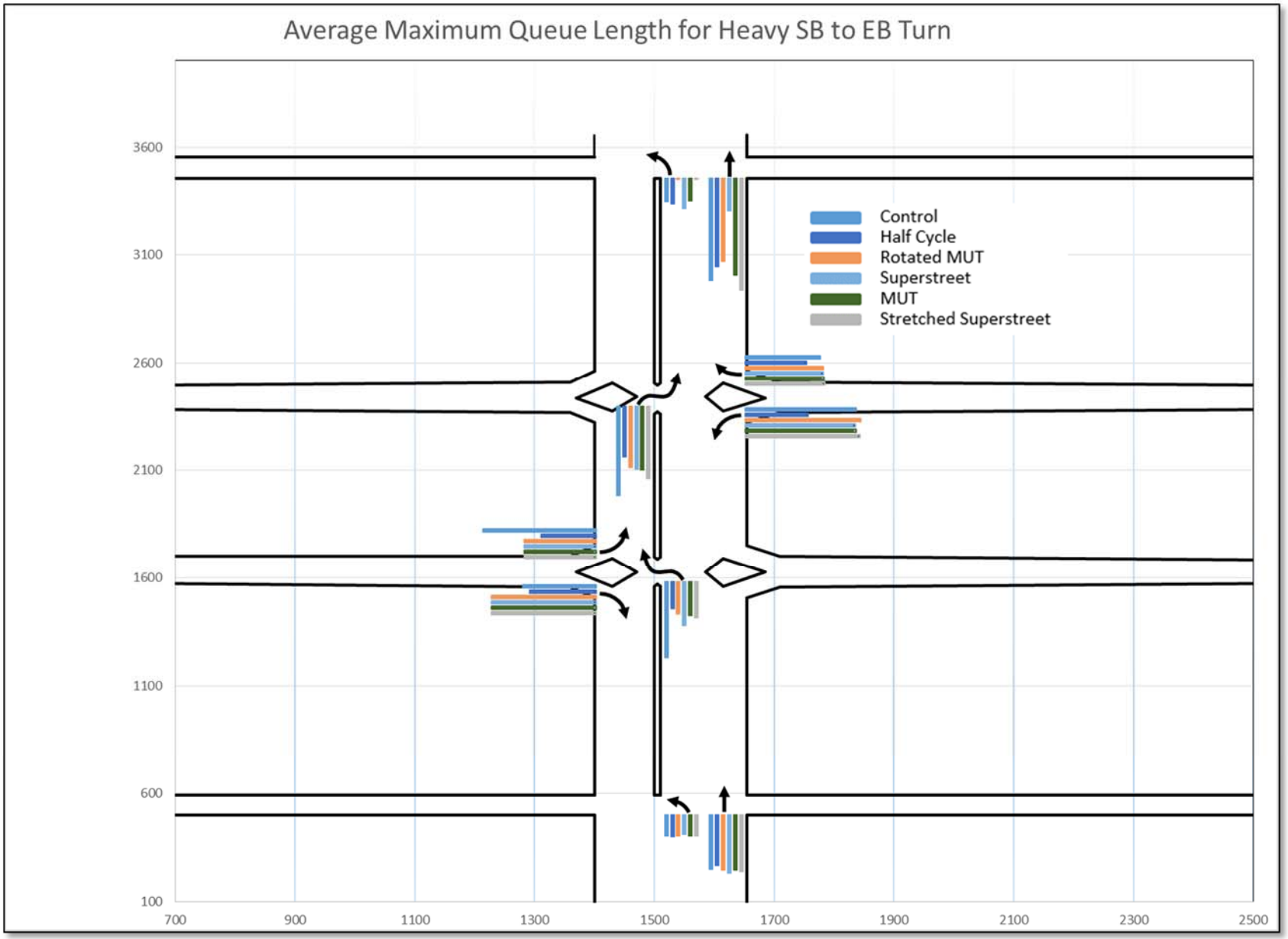
Overview of Results: Presented below is a diagram of a DDI with the movement of interest as well as the two corridor movements. Also included is a table featuring the four measures of effectiveness and their value under the control (no strategy) scenario for each of the movements highlighted in the diagram. Below the control values are the improvements seen in the measure of effectiveness for each strategy relative to the control. An improvement of 50%+ is shaded dark green while an improvement of -50%+ is dark red. The path over which delay and the stop severity index (SSI) is measured is indicated by a line while the movement at which the average maximum queue length and green usage is measured is indicated by a block.

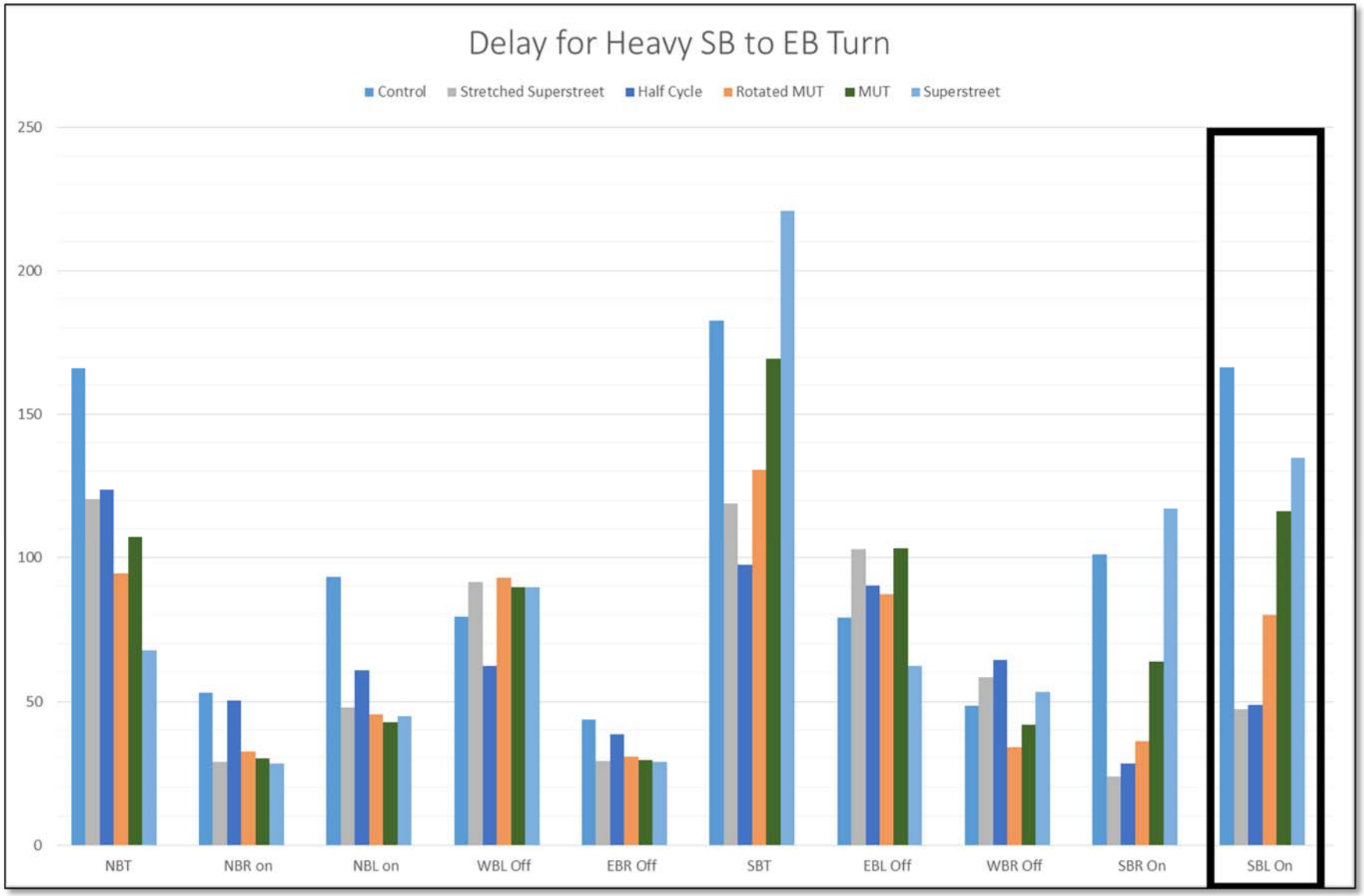
Further Considerations: Below the condensed results, four charts detail the full results for each measure of effectiveness. For the left turn from the freeway, it is important to consider the queue impact for the northbound traffic at the northern crossover. The increased volume at the southbound movement will reduce the green time for the northbound movement. Similarly, the northbound traffic at the northern adjacent intersection will be impacted.

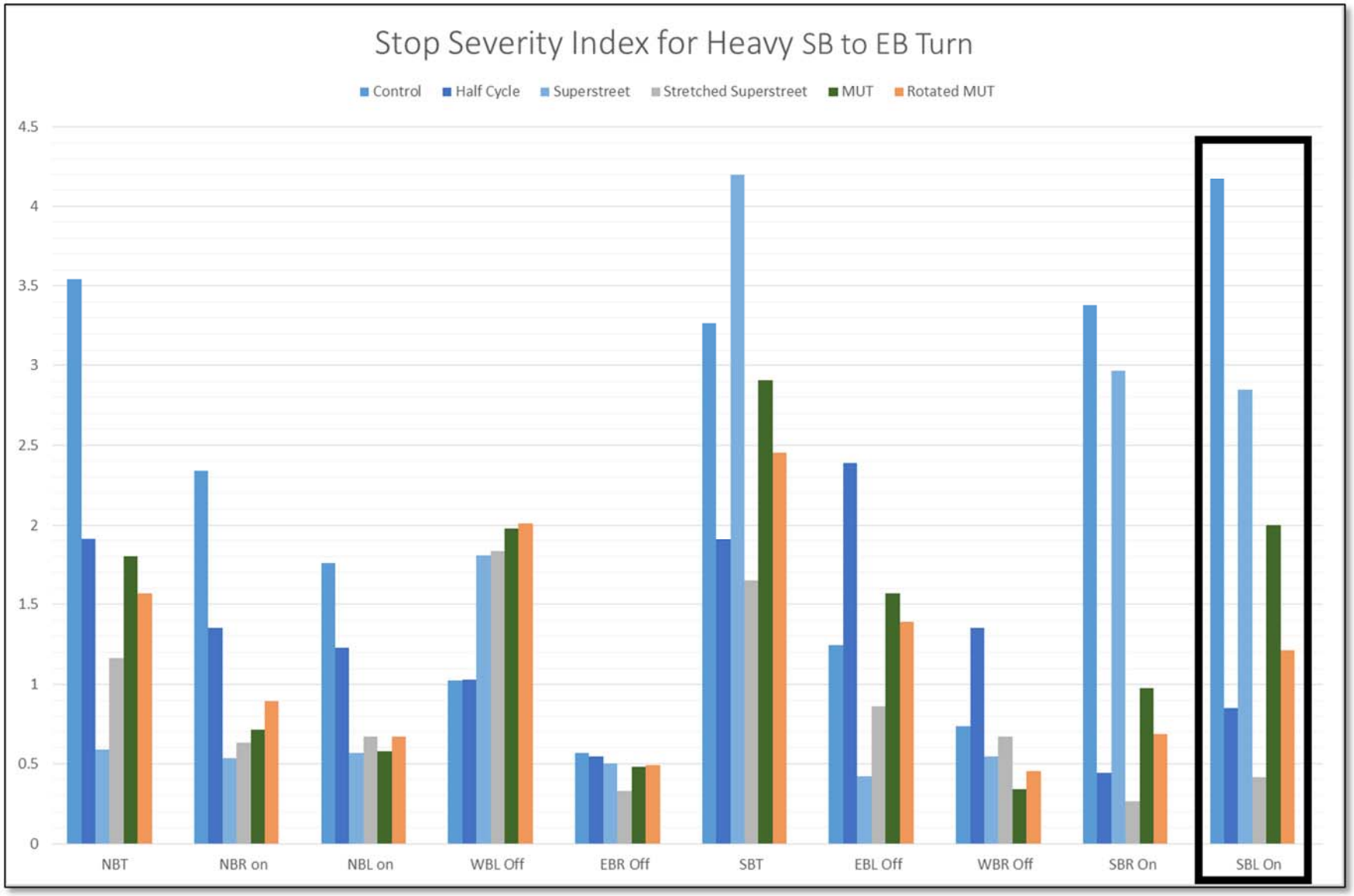
Supplemental Effects: From the summary chart, it can be seen that the superstreet performs well for the movement of interest and the northbound movements through the corridor, but is detrimental to the southbound movement through the corridor. This suggest the northern crossover and northern adjacent intersection are optimized for a southbound movement, but the southern adjacent intersection and southern crossover are optimized for northbound movement (as the heavy movement never reaches these intersections). Under no scenario does the green usage improve at the three main movements as marked on the next page.



	Left Turn Onto the Freway			North Bound Through the Corridor				South Bound Through the Corridor		
MOE	Delay	Stops	Green Usage	Delay	Stops	Green Usage	Queue	Delay	Stops	Green Usage
Measured Over	Path	Path	Mvmt	Path	Path	Mvmt	Mvmt	Path	Path	Mvmt
Control	166 seconds	SSI = 3.4	30%	166 seconds	SSI = 3.5	0.5	473 ft	183 seconds	SSI = 3.3	0.58
Half Cycle	0.29	0.20	0.91	0.75	0.54	0.98	0.86	0.53	0.59	0.91
MUT	0.70	0.48	-	0.65	0.51	-	0.94	0.93	0.89	-
Rotated MUT	0.48	0.29	0.93	0.57	0.44	0.88	0.81	0.71	0.75	0.93
Stretched Superstreet	0.28	0.10	-	0.73	0.33	-	1.08	0.65	0.50	-
Superstreet	0.81	0.68	-	0.41	0.17	-	0.32	1.21	1.29	-


Average Maximum Queue Length for Heavy SB to EB Turn
3600
3100
2600
2100
1600
1100
600
100
700
900
1100
1300
1500
1700
1900
2100
2300
2500
Control
Half Cycle
Rotated MUT
Superstreet
MUT
Stretched Superstreet


Delay for Heavy SB to EB Turn
Control
Stretched Superstreet
Half Cycle
Rotated MUT
MUT
Superstreet
250
200
150
100
50
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On


Stop Severity Index for Heavy SB to EB Turn
Control
Half Cycle
Superstreet
Stretched Superstreet
MUT
Rotated MUT
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
NBT
NBR on
NBL on
WBL Off
EBR Off
SBT
EBL Off
WBR Off
SBR On
SBL On

	Green Usage for Heavy SB to EB Turn															
	Downstream Adjacent				Outbound Crossover				Inbound Crossover				Upstream Adjacent			
	SBT	NBT	NBL	SBL	SBT	NBT	WBR	WBL	SBT	NBT	EBL	EBR	SBT	NBT	NBL	SBL
Control	58%	50%	43%	45%	56%	48%	15%	27%	30%	58%	13%	27%	36%	35%	40%	42%
Half Cycle	53%	49%	46%	44%	64%	46%	16%	21%	49%	39%	22%	16%	39%	36%	41%	43%
MUT					52%	50%	18%	21%	32%	36%	20%	15%	32%	35%	40%	40%
Superstreet					51%	47%	15%	20%	34%	34%	20%	15%	32%	36%	40%	39%
Rotated MUT	54%	44%			58%	47%	15%	24%	35%	36%	20%	15%	34%	35%	40%	42%
Stretched Superstreet					56%	49%	15%	22%	35%	36%	20%	15%	32%	36%	40%	37%