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ABSTRACT

PLANNING METHODOLOGY FOR ALTERNATIVE INTERSECTION DESIGN AND SELECTION

by
Liran Chen

The recent publication of the 6th Edition of the Highway Capacity Manual included a chapter on Ramp Terminals and Alternative Intersections that introduces various alternative intersection designs and assesses the performance of Median U-turn, Restricted crossing U-turn and Displaced left-turn intersections. Missing from the literature is an alternative intersection selection tool for identifying whether an alternative intersection would be successful under local conditions. With limited information of organized alternative intersection research, most planners must rely heavily on their personal judgement while selecting the most suitable intersection designs. As appealing as alternative intersections are, there is no comprehensive methodology for planners to evaluate all possible designs and locate the best option.

Several studies have been performed on identifying the selection of the most appropriate alternative intersection. As straightforward as they are, they failed to accommodate the Highway Capacity Manual (HCM) and are highly dependent on the professional judgment of the planners. This dissertation aims to design a selection methodology that is easy to use and HCM compatible and independent of personal judgments.

The selection procedure is composed of three stages. The goal of the first stage is to clarify the objectives and concerns of planners in the selection of candidate intersections. This stage should identify the treatment objectives (for existing intersections) and

stakeholders' concerns (for new intersections). If more than one objective were identified, the planners should assign a weight for each objective. A questionnaire should be used in collecting this information. The second stage is to filter out some candidate designs before the detailed analysis. This stage tries to generalize the range of application for each Unconventional Alternative Intersection Design (UAID). Any design that cannot satisfy the capacity and Right-of-Way (ROW) requirement is deleted from future analysis. In stage three of the selection process, the alternative intersection designs selected for consideration are ranked and assessed based on the treatment purposes/stakeholders' interests, which may likely include increasing mobility or safety.

By identifying a primary parameter used to score or rank all the considered intersections, the alternative intersection selection tool would assist planners to compare different intersection designs and to describe the intersection performance comprehensively. The primary parameter should account for both mobility and safety at each of the intersections evaluated. For intersection mobility, the evaluation process relies on methodologies provided in the Highway Capacity Manual 2016. For the safety assessment, a safety evaluation procedure is also developed to provide an overall assessment of the safety performance at the evaluated intersection. A selection algorithm is then designed to rank all intersections based the intersection performance.

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Liran Chen

A Dissertation
Submitted to the Faculty of
New Jersey Institute of Technology
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Doctor of Philosophy in Transportation

John A. Reif, Jr. Department of Civil and Environmental Engineering

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APPROVAL PAGE

PLANNING METHODOLOGY FOR ALTERNATIVE INTERSECTION DESIGN AND SELECTION

Liran Chen

Dr. Janice Daniel, Dissertation Advisor — Date
Professor of Civil and Environmental Engineering, NJIT

Dr. Steven I-Jy Chien, Committee Member — Date
Professor of Civil and Environmental Engineering, NJIT

Dr. Lazar Spasovic, Committee Member — Date
Professor of Civil and Environmental Engineering, NJIT

Dr. Branislav Dimitrijevic, Committee Member — Date
Assistant Professor of Civil and Environmental Engineering, NJIT

Dr. Athanassios Bladikas, Committee Member — Date
Associate Professor of Mechanical and Industrial Engineering, NJIT

BIOGRAPHICAL SKETCH

Author: Liran Chen

Degree: Doctor of Philosophy

Date: May 2022

Undergraduate and Graduate Education:

- Doctor of Philosophy in Transportation Engineering,
  New Jersey Institute of Technology, Newark, NJ, 2022

- Master of Science in Construction Management,
  New Jersey Institute of Technology, Newark, NJ, 2014

- Bachelor of Science in Engineering Management,
  Zhengzhou University, Henan, P. R. China, 2012

Major: Transportation Engineering

Presentations and Publications:

Liran Chen and Janice Daniel, “An Analysis of Breakdown Conditions at Alternative Intersections,” The Transportation Research Board 2Annual Meeting, Washington DC, United States, January 2020.

To my dearest family, my husband, and my precious child.

谨以此文，献给我挚爱的父母、丈夫和孩子。

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CHAPTER 1

INTRODUCTION

Traffic congestion has been a serious problem for decades. According to a study conducted by INRIX Research (Reed & Kidd, 2019), America lost about 97 hours in average due to congested conditions in 2018. This amounted to a cost of $1348 per driver annually according to the Federal Department of Transportation time loss valuation. The nationwide cost is $87 billion. Due to a new calculation methodology adopted in 2018, it is hard to compare the numbers with previous years. In 2017, the calculated congestion cost was $305 billion (Cookson, 2018), rising from the $124 billion in 2013 (Centre for Economics and Business Research, 2014).

To mitigate the economic loss and other impacts caused by traffic congestion, many researchers had devoted great effort to reduce traffic congestion. One of the traffic engineering solutions surrounds new forms of intersection designs. The intent of many of these new or alternative intersection designs is to reroute left-turn movements from the primary intersection, thus helping to produce a smoother traffic flow and reduce the delays at intersections. The Highway Capacity Manual 6th edition named those designs collectively as alternative intersections, but the name Unconventional Arterial Intersection Designs (UAIDs) is also very popular among researchers.

Modern roundabouts appeared during the 1960s and represent one type of alternative intersection of those early proposed. All the vehicles entered the roundabout went right along the circular road and exited at designated openings for different directions. Its benefit was significant. Numerous researchers have proved that roundabouts reduce traffic delay at the intersection and provide a safer driving environment. However, as time went by, its disadvantages became evident. Roundabouts were not very friendly to the pedestrians, and they could not handle unbalanced traffic flow very well (Rodegerdts, 2010). Many other designs were proposed afterwards to adapt various traffic situations.

In 1998, Hummer (Hummer, 1998a, 1998b) presented seven alternative intersection designs to address arterial congestion. These intersections were Median U-turn Intersections (MUT, also known as Michigan Left), Jughandles, Continuous Green Intersections, Bowtie intersections, Superstreets (also named Restricted Crossing U-turn Intersection), Continuous Flow Intersections (CFI, also named Displaced Left-turn Intersection), and Paired Intersections. The first three of the seven has been adopted in at least one state for decades and has proved to be successful in reducing congestion. The other four are variations of the existing unconventional designs. Additional unconventional intersection designs have emerged during recent decades. As of 2018, the Virginia Department of Transportation (VDOT) identified 26 unconventional intersection designs in their planning tool VDOT Junction Screening Tool (VJuST) version 1.02 (Lahiri, 2018; VDOT, 2017).

Despite the long history of unconventional intersection designs, their importance has been underestimated. Hummer's papers (Hummer, 1998a, 1998b) were the first to summarize preceding unconventional intersection designs and explored their advantages and disadvantages. Inspired by his work, others began to not only focus on intersection design, but also explored the suitability of each design. By comparing the performance of one or more unconventional intersection design with the conventional intersection under similar condition, it had been proved that these intersections are capable of reducing traffic delay and increasing traffic safety at the same time (El Esawey & Sayed, 2013).

Previous work had proved that alternative intersection designs were competitive when it comes to intersection transportation efficiency and safety. However, they also pointed out that those designs had their own advantages and disadvantages. For this reason, careful planning is necessary before any decisions made about the type of intersection design to be used. With limited research performed on alternative intersections analysis, and with a scarcity of data sources on the operation, performance and known difficulties of these intersections, there is still no widely accepted quantitative tool to help planners assess all the possible intersections. This dissertation research aims to design a practical tool for selecting the most appropriate intersection design. By following this proposed procedure, planners will gain deeper understanding of all the current intersection designs and be more confident when helping the local government making their decision during the planning process.

1.1 Problem Statement

The recent publication of the 6th Edition of the Highway Capacity Manual introduced a chapter on ramp terminals and alternative intersections that assesses the performance of Median U-turn Intersections, Restricted Crossing U-turn Intersections and Displaced

Left-turn Intersections. Missing from the literature is an alternative intersection selection tool for identifying whether an alternative intersection would be successful under local conditions. With limited information of organized alternative intersection research, most planners must rely heavily on their personal judgement while selecting the most suitable intersection designs. As appealing as alternative intersections are, there is no comprehensive methodology for planners to evaluate all possible designs and locate the best option.

Several studies have been performed on identifying the selection of the most appropriate alternative intersection. The most popular selection procedure was proposed by Warren Hughes (Hughes et al., 2010). In his report, he organized the selection methodology into six steps including: (1) establish objectives for projects and relative importance of factors; (2) assess the level of expected pedestrian activity and conflicts; (3) assess availability of right-of-way; (4) assess local site needs; (5) determine level of service at sketch planning level; and (6) conduct simulation analysis of viable alternatives. In Warren's theory, it is very time consuming or unnecessary to assess all alternatives. Therefore, he designed four more steps before traffic analysis of all feasible alternative intersections to filter out improper intersection designs in advance. This filtering process was based on judgement without any detailed analysis. In step (6) of Warren's methodology which involved the traffic analysis part, Critical Lane Volume (CLV) was considered, and Level of Service (LOS) was used as the decisive factor in selecting the most appropriate intersection. Any option with a summation of CLV over 1600 was rejected. Warren failed to provide a universally applicable quantitative tool to perform the assessment, and the evaluation of the selected intersection using a simulation analysis was not fully explained.

Asokan et al. (Asokan et al., 2010) incorporated more details to the Critical Lane Volume method in the selection of the best alternative intersection. This enhanced methodology uses capacity as the only parameter in the selection of the best intersection design and cannot provide effective guidance when more than one factors is being considered.

Engineers at the Indiana Department of Transportation introduced a decision tree algorithm into the intersection selection process (Bowen et al., 2014). This approach expands the candidate intersection design to include existing conventional and alternative intersections. The intersection selection methodology to be explored in this research will utilize information about the project Right of Way (ROW), intersection mobility, safety, construction cost and many other parameters in the section of candidate intersection designs. In this way, the methodology will rule out UAIDs that will not be useable, giving final selection based on the quantitative analysis of potential designs.

1.2 Research Objectives

This research aims at developing an intersection selection tool that is capable of ranking and assessing conventional, as well as major alternative intersection designs. This tool should be easy to use, Highway Capacity Manual compatible and with high accuracy. The selection procedure is composed of three stages. The goal of first stage is to clarify the objectives and concerns of planners in the selection of candidate intersections. This stage should identify the treatment objectives (for existing intersections) and stakeholders' concerns (for new intersections). If more than one objective were identified, the planners should assign a weight for each objective. A questionnaire should be used in collecting this information. The second stage is to filter out some candidate designs before the detailed analysis. This stage tries to generalize the range of application for each UAID. Any design that cannot satisfy the capacity and ROW requirement is deleted from future analysis. In Stage 3 of the selection process, the alternative intersection designs selected for consideration are ranked and assessed based on the treatment purposes/stakeholders' interests, which may likely include increasing mobility or safety.

By identifying a primary parameter used to score or rank all the considered intersections, the alternative intersection selection tool would assist planners to compare different intersection designs and to describe the intersection performance comprehensively. The primary parameter should account for both mobility and safety at each of the intersections evaluated. For intersection mobility, the evaluation process will rely on methodologies provided in the Highway Capacity Manual 2016. For the safety assessment, a safety evaluation procedure will also be developed to provide an overall assessment of the safety performance at the evaluated intersection. A selection algorithm will then be designed to rank all intersections based the intersection performance.

CHAPTER 2

LITERATURE REVIEW

This chapter is organized into six sections. Section 2.1 introduces the general information about current alternative intersection designs; Section 2.2 discusses the benefits and challenges of UAIDs. Section 2.3 examines assessments of UAIDs including a brief introduction of the HCM alternative intersection analysis method. Section 2.4 concludes with a discussion of alternative intersection selection methods used by DOTs. Section 2.5 introduces the related studies of UAID service volumes. And Section 2.6 covers the safety analysis of UAIDs.

2.1 General Information of Alternative Intersections

Previous research has been performed providing comprehensive reviews on the state of art of alternative intersections. One of the primary documents has been *Alternative Intersections/ Interchanges : Information Report (AIIR)*, by Hughes (Hughes et al., 2010). In this report, four alternative intersections and two alternative interchanges are discussed in depth including: displaced left-turn (DLF) intersections; restricted crossing U-Turn (RCUT) intersections; median U-turn (MUT) intersections; quadrant left-turn (QR) intersections; double crossover diamond (DCD) interchanges; and displaced left-turn interchanges. Two years later, Esawey and Sayed (El Esawey & Sayed, 2013) expanded the categories of alternative intersections into 11 designs by adding the unconventional MUTs, bowtie intersections, Jughandles, split intersections, upstream signalized crossover intersections, double crossover intersections, and parallel flow intersections. The roundabout design was excluded from their summarization because the authors believed that it has been excessively common to be referred to as an unconventional arterial intersection design. In 2016, a total of 26 alternative intersections and their variants were included in the alternative intersection selection tool - Virginia Junction Screening Tool (VJuST), developed by the VDOT (VDOT, 2017). A complete table will be developed to conclude all the alternative intersection designs have been mentioned so far. Among all these well discussed intersection designs, only six of them have been implemented in the United States. In the subsequent sections, each of these intersection designs will be introduced in detail. Information about all the other alternative designs will be summarized in the appendix.

2.1.1 Median U-turn Intersection

The concept of Median U-turn (MUT) Intersection can date back to 1960s and has been widely used in Michigan for more than half a century (Hummer, 1998b). Its major purpose was to reroute the left turn at primary intersections to avoid conflicts of left turn vehicles and the opposing through traffic. To achieve this goal, crossovers were placed at the medians of the arterial to accommodate the U-turn movement (Figure 2.1. (a)). In this design, arterial left-turning vehicles must go straight at the primary intersection and make a U-turn at the downstream crossover and merge on to the right turn lane at the opposite direction before reaching the primary intersection again and turn right. Left turn vehicles of the cross street will have to turn right at the major intersection and make a U-turn at the downstream crossover then go straight at the primary intersection.

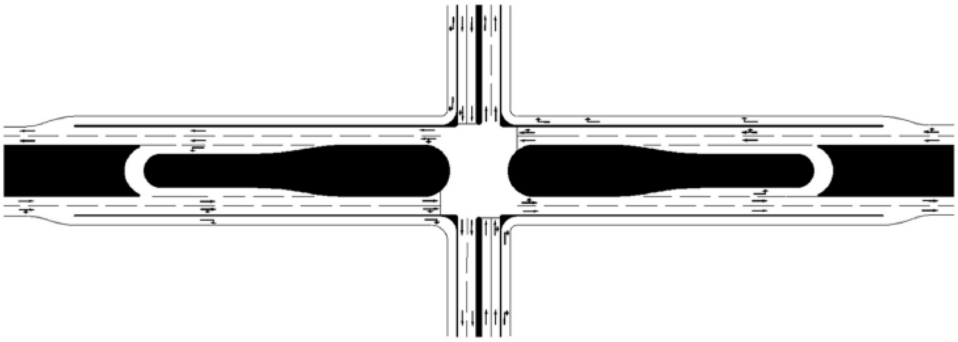


Figure 2.1 (a) Conventional MUT intersection.

As the traffic demand increases, some variants of the conventional MUT became popular. Some designs included crossovers on both the major and minor cross streets to remove left turns at the primary intersection. Shai and Choupani (Shahi & Choupani, 2009) proposed an “unconventional median U-turn” by building a non-traversable median at the primary intersection to prohibit any cross street vehicles from crossing this primary intersection (Figure 2.1 (b)). Unlike a common MUT, the primary intersection and crossovers of this unconventional MUT are typically controlled by a “STOP” or “YIELD” sign instead of signals. All the other movements remain the same with conventional MUT designs except that through movement from the cross street is rerouted to the downstream crossover located at the arterial and require these vehicles make a U-turn and then a right-turn at the primary intersection. Some papers also refer to these kind of designs as RTUT (Right Turn followed by U-turn) (Lu et al., 2001). No report has shown that this unconventional MUT has been built within the U.S., but it has been a common treatment of signalized intersections in Cairo, Egypt for more than two decades (Elazzony et al. 2011, El Esawey and Sayed 2011b). Iran adopted this design several years ago (Shahi & Choupani, 2009). However, the US seems to favor the common MUT over its variant and has applied it in states such as Michigan, Florida, Maryland, and New Jersey (Esawey and Sayed, 2012).

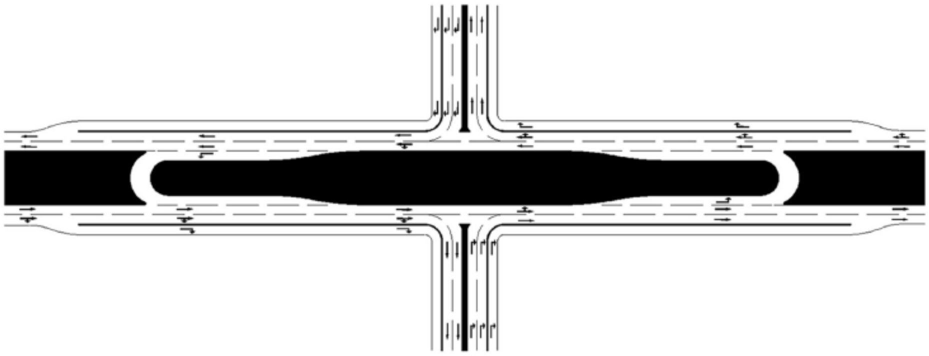


Figure 2.1 (b) Unconventional MUT intersection.

2.1.2 Restricted Crossing U-turn Intersection

Restricted Crossing U-turn Intersections (RCUT) are also known as superstreets and Reduced Conflict Intersections (RCI) (Eyler, 2011; Hummer, 1998b). This alternative intersection design has become one of the most promising treatments for traditional signalized intersections ever since it was proposed by Kramer in 1987 (Kramer, 1987). It can be viewed as extended development of the unconventional MUT. A RCUT resembles an unconventional MUT in many ways except that the primary intersection and crossovers of RCUTS are controlled by signals and left turns from the arterial are sometimes allowed at the primary intersection. Unsignalized RCUTs are more often referred to as J-turn Intersections (Edara et al., 2013; El Esawey & Sayed, 2013; Hughes et al., 2010). Detailed movement information and geometry design will be illustrated by Figure 2.2. Constructions of RCUT can be found in Indiana, Michigan, Minnesota, Maryland, North Carolina, and Texas.

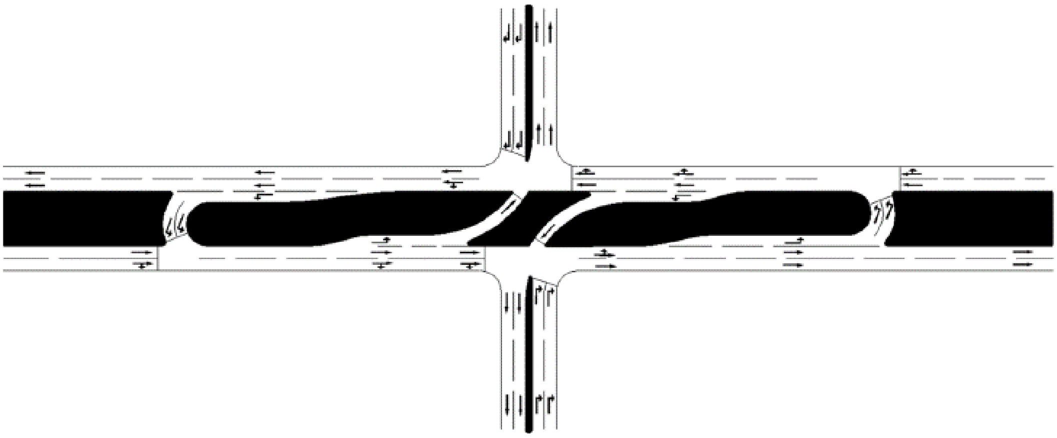


Figure 2.2 RCUT intersection.

2.1.3 Displaced Left-turn Intersection

Resembling the other members in the alternative intersection family, Displaced Left-turn Intersections (DLT) also have many appellations. Many researchers referred to it as Continuous Flow Intersections (CFI) (Goldblatt et al., 1994) and Crossover Displaced Left-turn Intersections (XDL) (El Esawey & Sayed, 2013). This concept was first proposed by Mier in the late 1980s (El Esawey & Sayed, 2013). By placing a crossover a few hundred feet before the primary intersection to allow left turn movements on the main arterial to be routed to a left turn lane placed on the left side of the opposing flow lane, this alternative intersection design allows both through and left turn movements to simultaneously utilize the intersection without creating a conflict with each other. Right turn movements are channelized to bypass the primary intersection (Figure 2.3). In general, a DLT intersection can be viewed as one primary intersection with four secondary intersections. Since it allows both through and left turn movements at the same time, the primary intersection can be operated under a two-phase signal (Reid & Hummer, 2001), reducing the traffic delay and vehicular accident rate at the same time. Due to its unique advantages, DLT intersection and its variations have been successfully implemented in New York, Louisiana, Utah, Oakland and Maryland in the United States (El Esawey & Sayed, 2013).

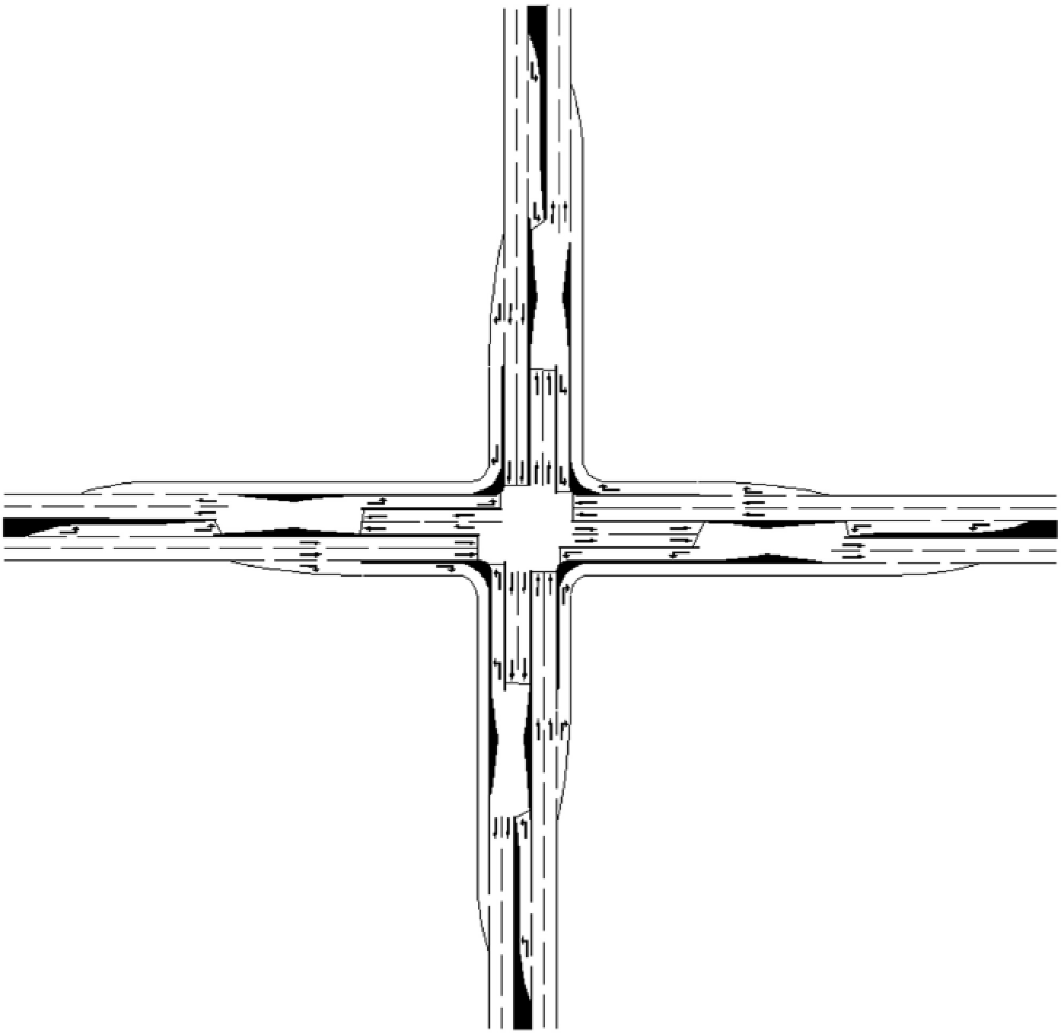


Figure 2.3 DLT intersection.

2.1.4 Jughandle

Jughandles, also known as New Jersey Left, have been in service in New Jersey for many decades (Hummer, 1998a). New Jersey Department of Transportation (NJDOT) has categorized current Jughandle designs into three types including: type A-Forward Ramps, type B-U-turn Ramps, and type C-Reverse Ramp based on the direction and location of the ramps. A typical type A Jughandle will consist of a four-approach intersection and two one-way ramps (Figure 2.4). The ramp usually starts a few hundred feet before the primary intersection from the cross street to a downstream crossover located several hundred feet away from the primary intersection. In this case, all the movements from the cross street will remain the same as the conventional intersections but the turn movements from the arterial will have to be rerouted to the ramp before making any turns. The ramp can be either stop controlled or yield controlled. Description of the other two variants can be found in Signalized Intersection: Information Guide by Rodegerdts et al.(Rodegerdts et al., 2004).

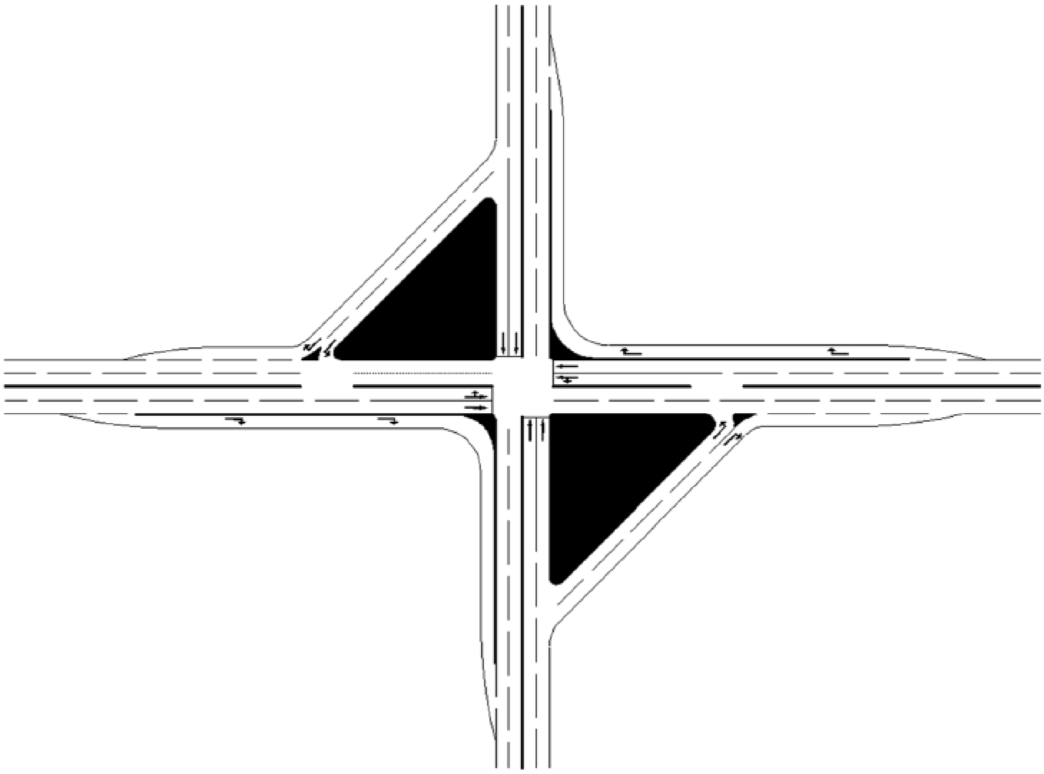


Figure 2.4 Jughandle intersection.

2.1.5 Quadrant Roadway

The concept of Quadrant Roadway (QR) design was first proposed by Reid in 2000 (Reid, 2000). The QR design removed left-turns from the primary intersection by adding a two-way “quadrant roadway” between two adjacent approaches. In this case, a quadrant roadway intersection can be viewed as one primary intersection surrounded by two secondary three-approach intersections. All the left-turns are rerouted to the quadrant roadways before making any turns (Figure 2.5). Currently, the QR design has been implemented in many states especially Michigan and New York.

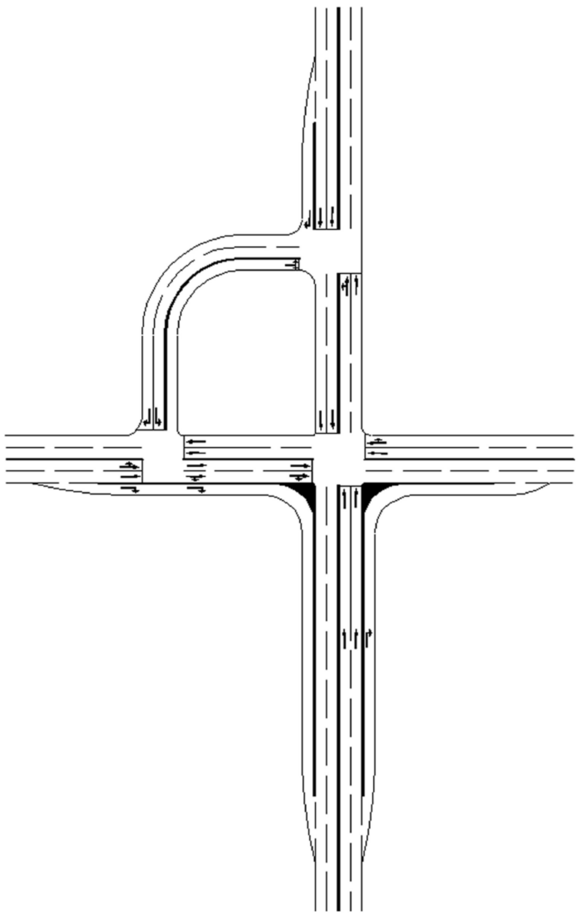


Figure 2.5 Quadrant roadways intersection.

2.1.6 Split Intersection

A split intersection splits a two-way arterial into two adjacent one-way streets, changing the one big intersection into two smaller intersections on the cross street (Polus & Cohen, 1997). Figure 2.6 demonstrates the geometry and vehicle movements in detail. Unlike most of the alternative intersection designs, the split intersection does not reduce many of the intersection conflict points. However, by separating one big intersection into two smaller intersections, the split intersection is capable of increase the intersection capacity and travel efficiency at the same time (Bared & Kaisar, 2000). Two adjacent smaller intersections on the cross street enables more storage lanes for left-turning vehicles. Also, the split intersection helps convert a four-phase signal intersection into two two-phase or three-phase intersection, thus reducing the total travel time. The split intersections have currently been constructed in Texas and Utah.

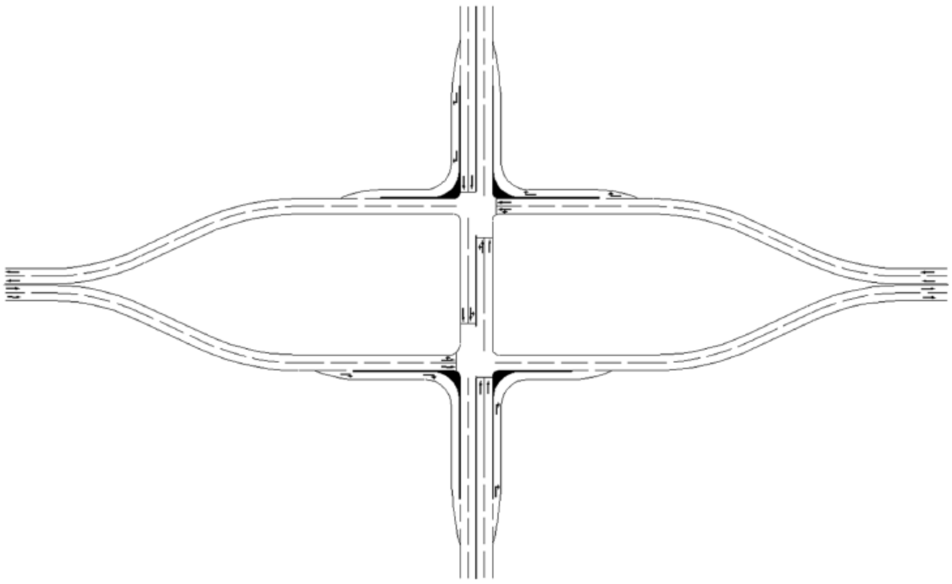


Figure 2.6 Split intersection.

2.2 Advantages and Disadvantages of Alternative Intersection Designs

As alternative intersection designs get more and more attention, a growing number of states have some interest in implementing these alternative intersection designs. However, it will easily lead to a wrong decision if planners do not have good information about all the advantages and disadvantages of constructed alternative intersections. To avoid this situation, many researchers have tested the performance of UAIDs against conventional intersections. This subchapter will summarize the advantages and disadvantages of selected alternative intersection designs.

The performance of UAIDs have been determined using a similar approach including selecting the target UAIDs, identifying measurable parameters for assessment, building models, and collecting data (from either simulation or field), running simulation, comparing the results, and drawing conclusions. This standard approach allowed for the results to be compared between UAIDs and conventional intersections.

Based on those experimental results (Hummer & Reid, 2000), the UAIDs share some similarities while each particular design outperform the others under some predefined circumstances. Table 2.1 summarized the general advantages and disadvantages of UAIDs, and Table 2.2 summarized their special strength and weakness.

Table 2.1 General Advantages and Disadvantages of UAIDs

Advantages	Disadvantages
• Reducing stops for through arterial traffic. • Reducing traffic delay for through arterial traffic. • Increasing capacity of the primary intersection for most UAIDs. • Theoretically safer for vehicles and drivers than conventional intersection with fewer and separate conflict points.	• Increasing confusion for new drivers. • Increasing delay and travel distance for left-turn vehicles. • Increasing the number of stops for left-turn vehicles. • Larger right of way needed than conventional intersections. • Drivers may ignore the left turn prohibition at the primary intersections.

Table 2.2 Special Advantages and Disadvantages of UAIDs

UAID	Advantages	Disadvantages
MUT	• Higher capacity and throughput at the primary intersection. • Easier progression for through arterial traffic. • Lower waiting time and congestion length for left turn vehicles. • Theoretically safer for crossing pedestrians.	• Increasing delays for all movements from cross street. • Unable to accommodate high left turn volumes and high approach volumes. • May harm roadside business due to the large median required.
RCUT	• Reducing traffic delay for left turn vehicles from either of arterials or cross street (usually arterials). • Perfect two-way progression with any signal spacing for through arterial traffic. • Theoretically safer to pedestrians. • Performing slightly better than MUT with low cross street volume.	• Usually over performed by MUT. • May cause confusion and extra travel time for pedestrians. • Increasing delays, stops, and travel distance for cross street through traffic. • Increasing delays, stops, and travel distance for one pair of left-turn movements (usually left turns from cross street).
DLT	• Increasing the primary intersection capacity. • Reducing delays for the whole intersection with properly designed intersection spacing. • Reducing stops for through arterial traffic. • Less ROW needed along the arterial. • Better performance with heavy left turns and through traffic.	• Increasing stops for left turn vehicles. • Restricting the U-turn possibilities. • Increasing confusion and travel time for pedestrians. • Increasing construction, maintenance, and operation cost for ramps. • May affect roadside business due to limited accessibility.
Jughandle	• Reducing stops for through arterial traffic. • Easier progression for through arterial traffic. • Narrower right-of-way needed along the arterial.	• Drivers may ignore the left turn prohibition at the primary intersection. • Increasing delay, stops and travel distance for left-turn vehicles from the arterial. • Additional construction and maintenance cost for ramps. • Lack of access to arterial for parcels next to ramps.

Not many studies have compared the advantages and disadvantages of QR and Split Intersections, thus no common information is available for comparing these intersections to other designs. However, in an unconventional intersection travel time investigation conducted by Reid (Reid & Hummer, 2001), QR will produce the least travel time for some specific intersections and the Split Intersection has a lowest total travel time during off-peak hours.

2.3 Current Operational Analysis Methodologies of Alternative Intersection

Current operational analysis methodologies for alternative intersections generally follows two patterns: simulation-based analysis and the Highway Capacity Manual method analysis.

The Florida DOT adopted the simulation based analysis to evaluate various alternative intersection designs in *Evaluating Transportation Systems Management & Operations (TSM&O) Benefit to Alternative Intersection Treatment* (referred to as *TSM&O Alternative Intersection* below)(Abou-Senna et al., 2015). Its methodology is quite straight forward: (1) conducted meta-analysis to evaluate DLT, MUT, RCUT, QR, Roundabout, Diverging Diamond Interchange (DDI) and double crossover intersection (DXI) in terms of area type and road conditions, right of way, pedestrian & bicyclist interaction, wayfinding, signalization, benefit-to-cost ratio, and performance measures; and (2) simulate the UAIDs in VISSIM to verify the conclusions from the previous step. The evaluation results from step 1 are summarized in a comprehensive table. Table 2.3 shows the evaluation of design criteria for DLT, as an example of the summary table.

For the complete information, please refer from the report. For each design, a case study was conducted to verify the conclusions. The researchers picked one existing intersection for each UAID and compared its performance under both the conventional intersection and the UAID. The simulation results matched with the summary conclusion: UAIDs outperform most of the conventional intersections and enhance the traffic mobility and safety.

Table 2.3 Evaluation of Design Criteria for DLT

<table>
<tr><th>Design Criteria</th><th>Treatment: DLT</th><th>Design Criteria</th><th>Treatment: DLT</th></tr>
<tr><td>Area type</td><td>Urban & Suburban areas</td><td rowspan="2">Operational</td><td rowspan="2">Capacity along corridors increase by 20-50%.
Average speed increases by 13-30%.
Energy savings of 5-11%.
HC, CO, and NOx emissions decreased by 1-6%.
Fewer and less severe crashes.
Improved level of service.</td></tr>
<tr><td>Roadway conditions</td><td>Heavy Lefts; V/C > 0.8.
LT*Opp vol>150,000.
LT > 250 vphpl & opp vol > 500 vphpl
Many signal phase failures.
LT spill beyond storage length.</td></tr>
<tr><td>ROW</td><td>Smaller footprint & cheaper than interchange.
Larger footprint than conventional intersection.
Crossover radii (150-200 ft).
DLT can have 4 or 2 displaced lefts.
Adjacent land use access is affected.
300-600 ft from crossover to primary intersection.
Wider medians & lane widths (15 ft) at crossover.</td><td>Signalization</td><td>Up to 5 signals for full DLT with single controller.
Signals are usually coordinated.
Offset length determines signal phase.
Crossover lefts and minor street move together.
No RTOR is recommended but depends on the design.
No U-turn signs for thru movements
Issues with flashing signals or loss of power.</td></tr>
<tr><td>Pedestrian Interactions</td><td>Crossing distance increase.
1-stage or 2-stage crossings.
Need wider medians.
Refugee island between LT & Thru lanes.
Special consideration to pedestrians with disability.
Need signals at channelized right turns.</td><td>Wayfinding</td><td>Position signal heads above crossover lanes.
Signs placed 0.25 miles & 200ft in advance.
Provide wrong way signage and pavement markings.
Consider overhead & post mounted signage.
Provide lighting at conflict points.
Potential for wrong way movement</td></tr>
<tr><td>Bicycle Interactions</td><td>Use traffic lanes as vehicles.
Use bicycle ramps on sidewalks.
Use shared paths on crosswalks.
Use bicycle box on far side of refugee islands.</td><td>Benefit to cost Ratio</td><td>High benefit to cost ratio (can reach up to 11:1).
Cost ranges from $4-8 million.
Grade separation range from $10-30 million.</td></tr>
</table>

LT: left turn; Thru: through; Opp: Opposite; Vol: Volume; vphpl: volume per hour per lane

Source:(Abou-Senna et al., 2015)

The *Highway Capacity Manual* (2016) adopted the parameter Experienced Travel Time (ETT), as a more quantitative performance measure to assess UAIDs. The ETT formula was modified from the traditional equation to calculate control delay. For each O-D movement, the ETT composes the control delay at each junction and extra travel time due to extended travel distance required by the intersection design. Control delay at each junction was calculated by viewing each junction as an independent component and calculated the control delay as a traditional intersection, respectively.

The estimation of extra distance travel time was obtained by dividing estimated speed from extra travel distance. Extra delay caused by weaving maneuver is also an important component of the experienced travel time, but due to limited research available, the HCM 2016 left this part in blank. Figure 2.7 explains the steps taken in the HCM 2016 to estimate the ETT.

This framework of HCM 2016 provided planners numerical and consistent parameters to evaluate the performance of candidate UAIDs and conventional intersections.

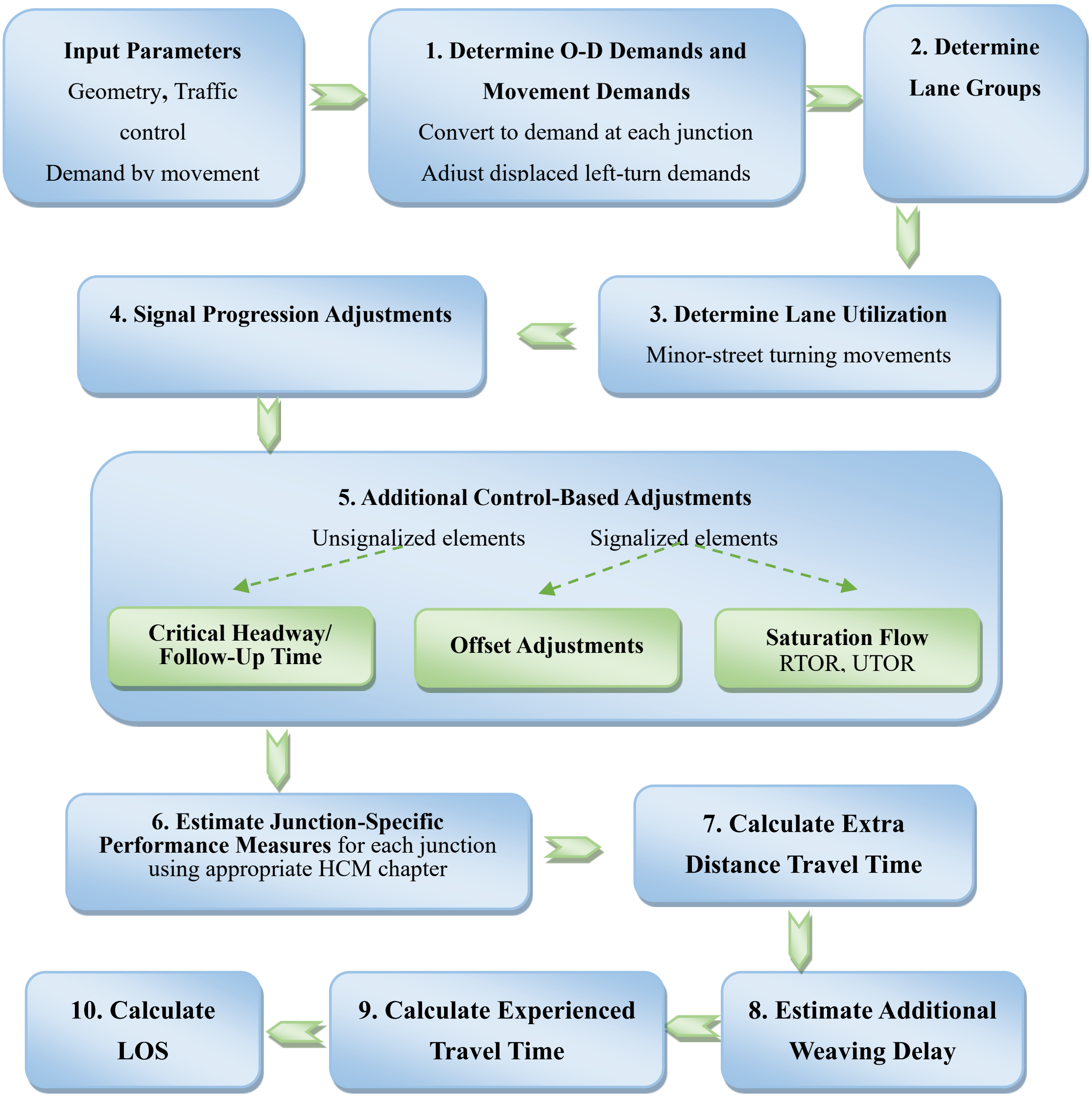


Figure 2.7 Highway Capacity Manual operational analysis procedure.

2.4 Current Selection Methodology of Alternative Intersection

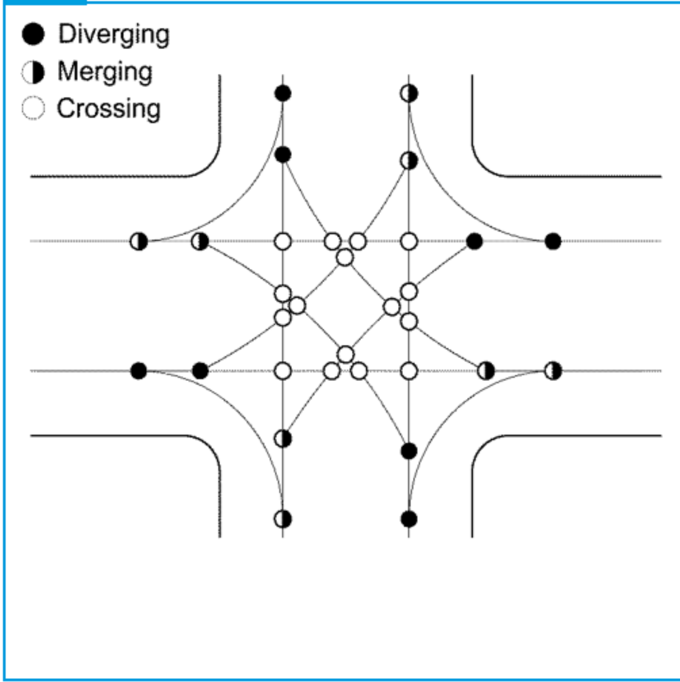
While HCM sets up a framework for determining the operational performance of alternative intersection, it is still not enough for the planners to make a final decision on the intersection design to be used because there are much more factors to be considered when it comes to selecting the most appropriate UAID or conventional intersection design.

As previously mentioned, one of the first alternative intersection selection methods to be developed was the one that proposed by Warren Hughes in the *Alternative Intersections/Interchanges: Informational Report* (2010). In this report, the author organized proposed selection methodology into six steps: (1) establish objectives for projects and relative importance of factors; (2) assess level of expected pedestrian activity and conflicts; (3) assess availability of right-of-way; (4) assess local site needs; (5) determine level of service at sketch planning level; and (6) conduct simulation analysis of viable alternatives. In Warren's theory, it is very time consuming or unnecessary to assess all alternatives. Therefore, he designed four additional steps before traffic analysis of all feasible alternative intersections to filter out improper intersection designs. This filtering is done based on judgement without a detailed analysis. In the traffic analysis part, Critical Lane Volume (CLV) was considered, and Level of Service (LOS) was selected as the primary factor in selecting the most appropriate intersection design. Any option with a summation of CLV over 1600 is rejected.

The premise of CLV method was to assume the intersection was dominated by a sequential of conflicting movements. For example, for east-west movements, the westbound left turn movement cannot proceed simultaneously with the eastbound through movement. In this case, whichever movement had the higher volume is the critical movement and its volume is the CLV for the east-west movement. Similarly, for north-south movements, the movement with higher volume in the conflicting movement set is the CLV for the north-south. The CLV of the intersection is the summation of north-south movement CLV and east-west movement CLV. This method was proposed by Asokan and his coworkers in 2010 (Asokan et al., 2010) and developed a set of excel worksheets to calculate CLV for various UAIDs. The Federal Highway Administration adopted this idea and expanded the UAIDs that can be selected (Sangster & Rakha, 2014). The developed excel workbook was open to the public and can be find by the name Capacity Analysis for Planning of Junctions (CAP-X) tool.

Virginia DOT expanded the CAP-X tool into the VJuST by incorporating more UAID selections and pedestrians into consideration, as well as the safety analysis (VDOT, 2017). The methodology of operational analysis stayed the same as CAP-X. In the safety analysis, the weighted total conflicting points is the performance measure. More weighted total conflicting points indicating higher crash risks. After identifying the conflict points of each UAID, a weight is assigned before each conflict point type: crossing conflict, merging conflict, and diverging conflict. Based on the Highway Safety Manual crash cost and the average crash unit cost by crash type in Virginia, crossing conflict costs twice as much as the merging/diverging conflict cost. Therefore, the given weight is two for crossing conflict points and one for merging/diverging conflict points. Figure 2.8 showed the conflict point diagram for conventional intersection.

Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	16
Merging	8
Diverging	8
Total	**32**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

48

Figure 2.8 Conventional intersection conflict point diagram.

Source: (VDOT, 2017)

Indiana DOT took an alternative direction to selecting alternative intersections (Bowen et al., 2014). Engineers at Indiana DOT introduced the decision tree algorithm into the intersection selection process. The whole procedure is divided into two stages: Stage 1 for initial feasibility screening and Stage 2 for expanded performance assessment. Stage 1 incorporated four screening questions regarding ROW feasibility, ability to solve essential project intent, ability to improve or maintain current state of performance, and other concerns like capital cost and environmental impacts. For each question, if the answer is yes then the candidates moved to the sequential question. If the answer is no, then the candidates were discarded. Stage 2 contains four performance questions with respected to intersection mobility, safety, capital cost to benefit, and other performance measures including stakeholder concerns, environmental impact, and additional factors. Figure 2.9 (a) and (b) summarized the detailed procedures.

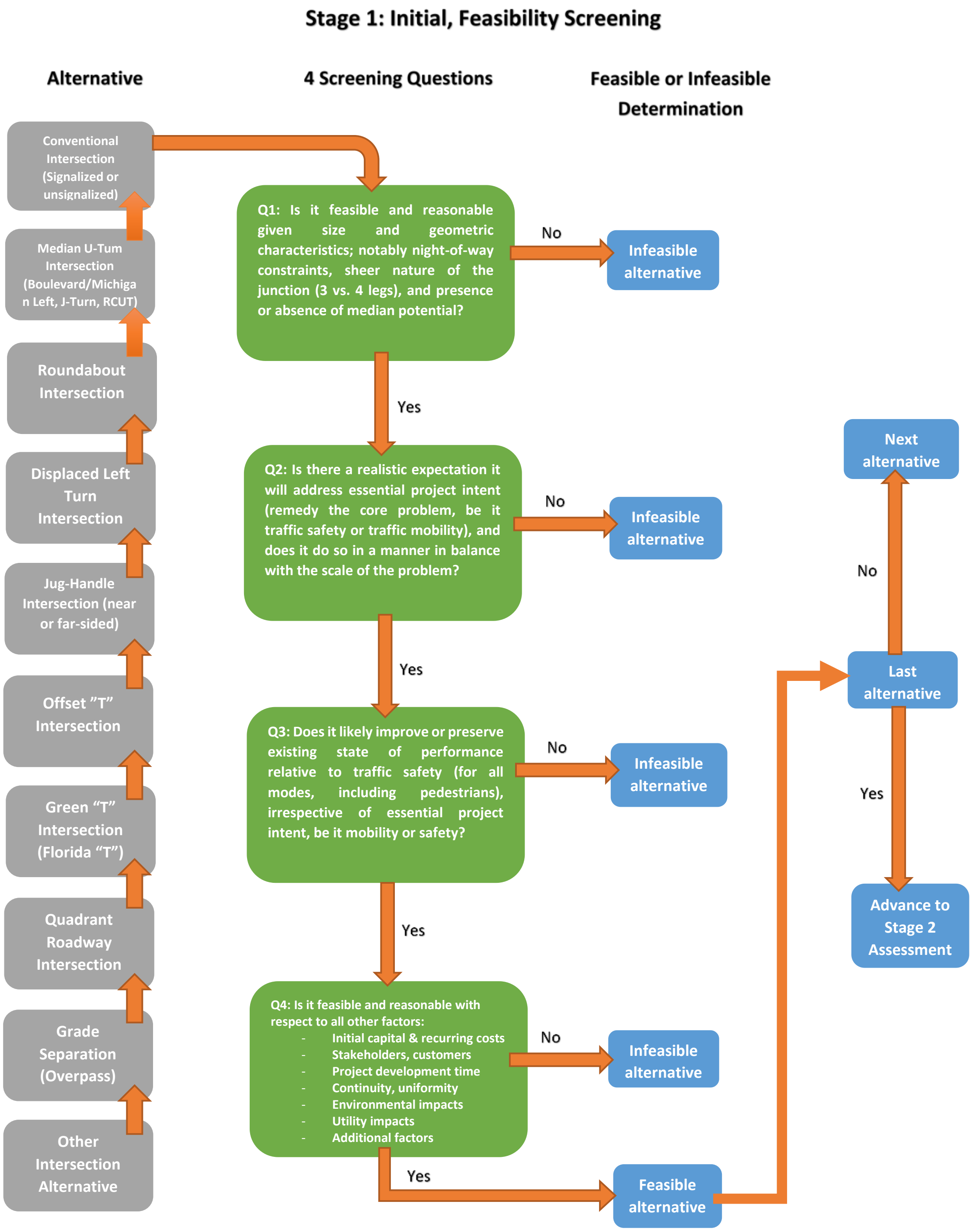


Figure 2.9 (a) Stage 1 of Indiana DOT's decision tree.

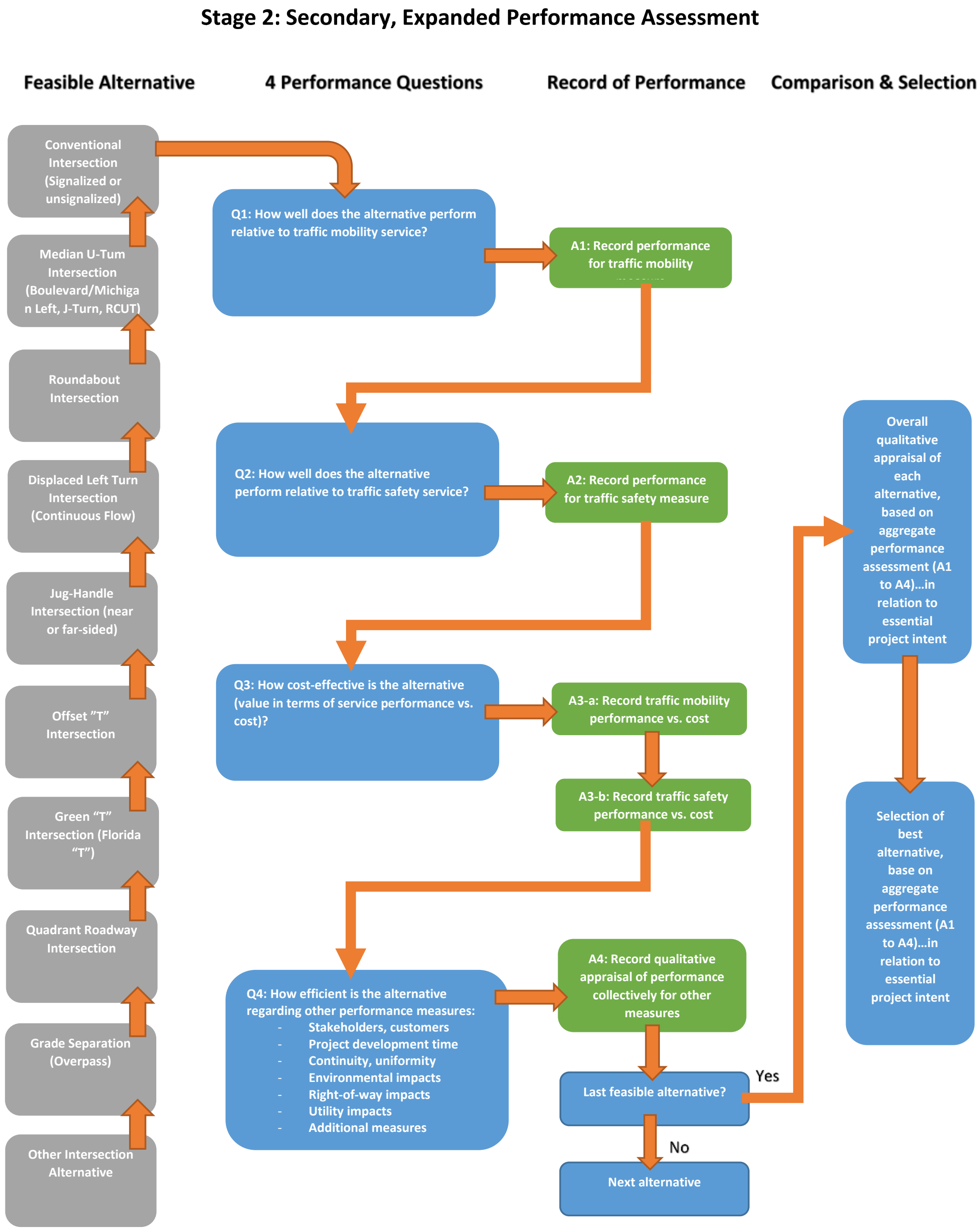


Figure 2.9 (b) Stage 2 of Indiana DOT's decision tree.

2.5 Service Volumes of Alternative Intersections

Limited research has studies service volumes of alternative intersections. Hummer 20010 (Joseph E. Hummer, 2010) developed a series of service volume tables for RCUT's main street in one of his research. In this study, Hummer simulated RCUT in North Carolina Level-of-Service (NCLOS) software. In his simulation, three intersection grade conditions were considered: level, rolling and mountainous. Two traffic lane conditions were considered: 4-lane RCUT Highway and 6-lane RCUT Highway. Table 2.4 shows the details.

Table 2.4 AADT Capacity for LOS Boundary Thresholds of RCUTs

	AADT (veh/day)			**AADT (veh/day)**		
	4-lane Superstreet Highway (Isolated Location)			**6-lane Superstreet Highway (Isolated Location)**		
LOS	**Level**	**Rolling**	**Mountain**	**Level**	**Rolling**	**Mountain**
A	32,300	30,800	28,200	48,400	46,200	42,300
B	43,300	41,300	37,800	64,900	61,900	56,600
C	48,000	45,800	41,900	72,000	68,600	62,800
D	50,300	47,900	43,900	75,400	71,900	65,800
E	51,500	49,100	45,000	77,300	73,700	67,400

Default inputs: Partial adjusted sat. flow = 1700 pcphpl; cycle length = 90 sec; g/C = 0.70; number of lanes = 2; PHF = 0.90; % of trucks = 5; K = 0.10; D = 0.55.
Source:(Joseph E. Hummer, 2010)

The major disadvantage of Hummer's study is that it ignores the impact of vehicles from minor streets. Since all vehicles from minor streets are rerouted to a downstream crossover on the main street, vehicles from minor streets contribute a substantial portion of the RCUT intersection delay. Only considering the vehicle delays from main streets while developing service volumes for RCUT makes the numbers less convincing. Also, the delay calculation in this study is not compatible with the HCM, therefore, its range of application is limited and requires local adjustment when applies to a local intersection. Finally, the cycle length and g/c ratio are constant in the simulation and not optimized. The RCUT will not reaches its optimal performance under a fixed signal plan, therefore the obtained service volumes are not optimal.

In conclusion, the service volumes presented in Table 2.4 provide guidance while analyzing the performance of RCUT but require future research and needs extra caution when applied to local conditions.

2.6 Safety Analysis

The safety performance of alternative intersections is another major concern of transportation planners; therefore, many researchers have investigated the safety performance of alternative intersections. A large portion of research concluded that alternative intersections reduce crashes frequencies when comparing to the conventional intersection (Al-Omari et al., 2020; Q. Sun, 2019; Wolfgram, 2018; Zlatkovic & Kergaye, 2018). However, some recent research showed that alternative intersections do not always improve traffic safety under all geometric and volume conditions (Abdelrahman et al., 2020; Azizi & Sheikholeslami, 2013). The most chosen safety performance parameter in previous studies is the crash modification factor (CMFs). Safety performance functions (SPFs) are also mentioned in some research (X. Sun, 2019). Therefore, this chapter investigates the prevailing safety analysis methodologies adopted in the previous studies including various methodologies to develop CMFs and presents the crash modification factors that have been developed for alternative intersections in the end.

2.6.1 Prevailing Safety Analysis Methodology

In practice, the complexity of safety analysis varies as the research objection changes. Various safety analysis methods have been developed to fulfill different research demands. The following summarizes the prevailing safety analysis methods fulfilling different research demand.

Conflict Point Analysis

There exist some circumstances where only the simplest safety analysis is needed or very limited data available. In this case, conflict point analysis is a widespread practice. Conflict point analysis utilizes an abstracted highly idealized environment when compared to reality where the crash frequency of an intersection is only affected by the number of conflict points within itself, while other influential factors such as road conditions, driver’s familiarity are ignored. In this analysis, a conflict point is defined as the cross point of two conflicting movements. Conflict points are classified as merging conflict points, diverging conflict points, and crossing conflict points. Each type of conflict point is assigned with a specific weight, and the intersection with least weighted total conflict points is identified as the safest intersection. The Virginia Department of Transportation (VDOT) adopted this safety analysis approach and incorporated it into their alternative intersection/interchange screening tool Virginia

Junction Screening Tool (VJuST). Figure 2.8 in Section 2.4 is a good example of the conflict point analysis of a four-legged conventional intersection.

Safety Analysis Methodologies Using CMFs and SPFs

Agencies with a more comprehensive safety analysis need tend to use Crash Modification Factors (CMFs) and Safety Performance Functions (SPFs). Although both concepts are used to estimate the number of crashes, they are different to each other. The Highway Safety Manual defines a crash modification factor as a multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site, while a safety performance function is an equation used to predict the average number of crashes per year at a location as a function of exposure and, in some cases, roadway or intersection characteristics (e.g., number of lanes, traffic control, or median type). In general, CMFs focus on the safety impact of a geometric design or traffic control device, such as the construction of an alternative intersection in this context, and a safety performance function describes the relationship between crash frequency and traffic volume for a roadway or intersection. (Bonneson et al., 2021)

The CMF developing methodologies for alternative intersections follows the same procedure as conventional intersection. Frank Gross (Frank Gross, 2010) introduced six methodologies in developing the values of CMFs including: using before and after crashes with a comparison group analysis; empirical Bayes before-after analysis; full Bayes analysis; cross-sectional analysis; case-control analysis; and Cohort analysis. Due to the data limitations of alternative intersections, before and after with comparison group analysis, empirical Bayes before-after studies and cross-sectional analysis are most common in developing CMFs for alternative intersections. The following introduces the detailed information, advantages, and disadvantages of these three methodologies.

Before-after analysis with comparison group

In the before and after analysis, traffic volume and crash data before and after the construction of an alternative intersection and the comparison group selection are the keys of a reliable analysis. Hauer (Hauer, 1997) proposed the use of sample odds ratios to evaluate the suitability of a chosen comparison group. The sample odds ratios must be calculated for each before-after pair before the construction of the proposed alternative intersection. Equation (2.1) shows the calculation details for determining the sample odds ratio. The closer the ratio is to 1.0, the better chosen the comparable group.

$$sample\ odds\ ratio = \frac{(Treat_{before}Comp._{after})/(Treat._{after}Comp._{before})}{1+\frac{1}{Treat._{after}}+\frac{1}{Comp._{before}}} \quad (2.1)$$

Where,

$Treat._{before}$ = total crashes for the treatment group in year i.

$Treat._{after}$ = total crashes for the treatment group in year j.

$Comp._{before}$ = total crashes for the comparison group in year i.

$Comp._{after}$ = total crashes for the comparison group in year j.

With the proper comparison group chosen, the CMF can be calculated by the following equations.

$$N_{expected,T,A} = N_{observed,T,B}\frac{N_{observed,C,A}}{N_{observed,C,B}} \quad (2.2)$$

$$CMF = \frac{N_{observed,T,A}/N_{expected,T,A}}{1 + var(\frac{N_{expected,T,A}}{N_{expected,T,A}^2})} \quad (2.3)$$

Where,

$N_{observed,T,B}$ = the observed number of crashes in the before period for the treatment group.

$N_{observed,T,A}$ = the observed number of crashes in the after period for the treatment group.

$N_{observed,C,B}$ = the observed number of crashes in the before period in the comparison group.

$N_{observed,C,A}$ = the observed number of crashes in the after period in the comparison group.

Empirical Bayes before-after analysis

Empirical Bayes before-after analysis take the before-after analysis with comparison groups one step further by taking regression-to-mean into consideration. Resembling the comparison group method, the empirical Bayes method also requires a set of comparison groups (sometimes referred to as reference groups in other research studies). A SPF is developed from the comparison group to predict the number of crashes in the treatment group. While calibrating the SPF, a SPF weight is also obtained from the over-dispersion parameters. Thus, the CMF can be obtained through Equation (2.3) with slight modification in calculating the value of $N_{expected,T,A}$ and $Var(N_{expected,T,A})$:

$$N_{expected,T,B} = \text{SPF weight} \times \mathrm{N_{predicted,T,B}} + (1 - \text{SPF weight})\, N_{observed,T,B} \quad (2.4)$$

$$N_{expected,T,A} = N_{expected,T,B} \frac{N_{predicted,T,A}}{N_{predicted,T,B}} \quad (2.5)$$

$$Var(N_{expected,T,A}) = N_{expected,T,A} \frac{N_{predicted,T,A}}{N_{predicted,T,B}} (1 - SPF\ weight) \quad (2.6)$$

Where,

$N_{expected,T,B}$ = the unadjusted empirical Bayes estimate for number of crashes in the before period for the treatment group.

$N_{predicted,T,A}$ = the predicted number of crashes estimated by the SPF in the after period for the treatment group.

$N_{predicted,T,B}$ = the predicted number of crashes estimated by the SPF in the before period in the treatment group.

For both before-after analysis with comparison group and empirical bayes before-after analysis, there is a need to pay special attention to factors may produce potential bias. Gross identified these factors as follows:

1. Traffic volume changes due to general trends or to the alternative intersection design itself.

2. Changes in reported crash experience due to changes in crash reporting practice, weather, driver behavior, effects of safety programs, etc.

3. Improper selected comparison group.

Cross-sectional analysis

The cross-sectional analysis is another useful method to estimate CMFs when the number of instances is limited to perform a before-after analysis.

In the cross-sectional analysis, a safety performance function is developed to quantify the relationship between number of crashes and all variables that affect safety (i.e., intersection types, annual average daily traffic (AADT)). Linear distribution, Poisson distribution, and negative binomial distributions are most used to model the SPF. The CMF can be then inferred by exponentiating the parameter of the variable related to the proposed change (i.e., intersection type, AADT, etc.). Equations (2.7) and (2.8) show a simple example of obtaining a CMF from a SPF.

$$\text{Predicted Crashes} = \exp[\alpha + \beta \cdot \ln(\text{AADT})] \quad (2.7)$$

$$CMF_{AADT} = \exp(\beta) \quad (2.8)$$

where,

α = model intercept,

β = coefficient of the independent variable AADT,

However, the CMFs developed from a cross-sectional analysis should be used cautiously since the crash rate change may be caused by other factors than those that have been identified in the SPF model. In fact, it is difficult to properly identify and measure all the safety influential factors of alternative intersections. It is highly likely that the derived CMFs are inaccurate if a function is improperly selected, some influential variables omitted, or the selected variables are correlated.

Surrogate Safety Assessment Model

Surrogate Safety Assessment Model (SSAM) is a simulation tool to analyze the safety of roadways when real data is not available(*Surrogate Safety Assessment Model and Validation: Final Report*). By importing the vehicle trajectories files generated by simulation software like VISSIM, SSAM is capable of analyze the vehicle trajectories and predicts the number of crashes and crash severities. Wolfgram (Wolfgram, 2018) adopted SSAM in his research to investigate the safety impact of DLTs. SSAM is a powerful tool when field data is inadequate or unavailable. At the same time, its disadvantage is also clear. In many cases, a simulation model cannot 100% duplicate the actual driving behavior. To obtain a reliable result, the SSAM analysis results should not be used without a rigorous calibration.

Comprehensive Safety Analysis with Highway Safety Manual

Most safety analysis of alternative intersections stopped at deriving CMFs. Currently there is no comparable analysis method for alternative intersections as those developed for conventional intersection. The first edition of Highway Safety Manual and its supplements did not mention the safety analysis method for alternative intersections. The released outline for upcoming second edition of Highway Safety Manual also did not mention alternative intersections. However, the researchers still may be inspired by the proposed safety analysis method for conventional intersections and tailor it to fit for alternative intersections. Figure 2.10 showed the framework of the Highway Safety Manual analysis method for conventional intersections.

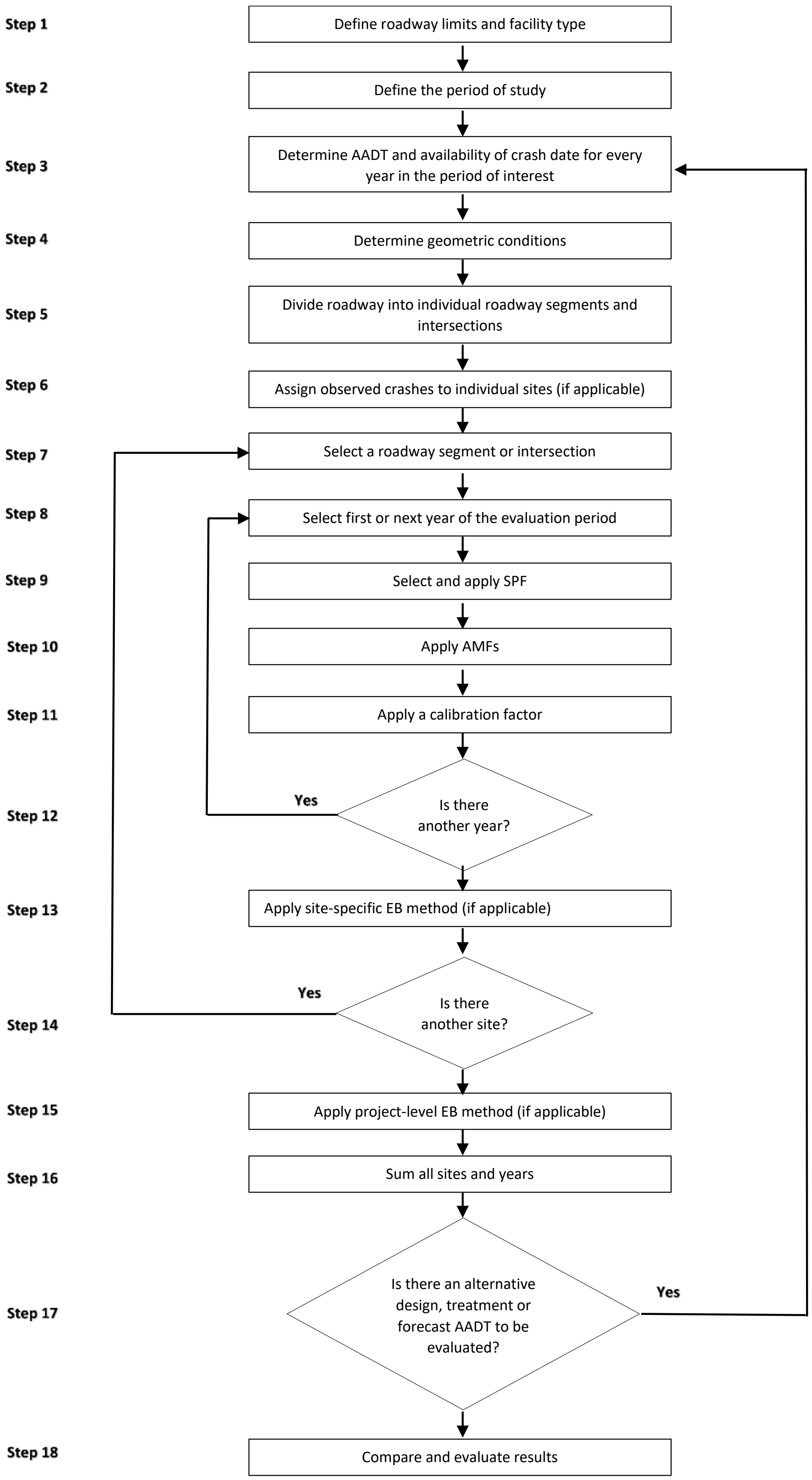


Figure 2.10 Highway Safety Manual analysis method.

2.6.2 Crash Modification Factors Developed for Alternative Intersections

More developments of CMFs for alternative intersections have been done in the past few years. Extremely limited resources can be found before the year 2015 due to the lack of available crash data. In this section, a total of seven papers have been reviewed. Four of them studied the CMFs for DLTs, and three of them calculated CMFs for MUTs. Seven research developed CMFs for RCUTs, and the clearinghouse website has analyzed and ranked the studies based on the data quality and research methodology. The CMFs with a high quality-score will be presented below. Please note, the papers using the same source of data and that get the same CMF values are considered as one paper; the paper with earliest publication is listed in the reference below.

CMFs for DLTs

Table 2.5 summarizes the previous work that has been done to estimate the CMF values for DLTs. Since most constructed DLTs are partial DLTs with only two approaches rerouted the left-turn movements, all the papers listed below derived their CMFs based on the partial DLT. They should be used with caution when calibrating the safety impact of a full DLT.

Table 2.5 Previous Studies Deriving CMFs for DLT

Study	Safety Analysis Method	DLT Type
Abdelrahman et al. 2020	Before-after with comparison group, Cross sectional analysis	Partial
Wolfgram 2018	SSAM	Partial
Zlatkovic and Kergaye 2018	Empirical Bayes	Partial
Zlatkovic et al. 2015	Empirical Bayes	Partial

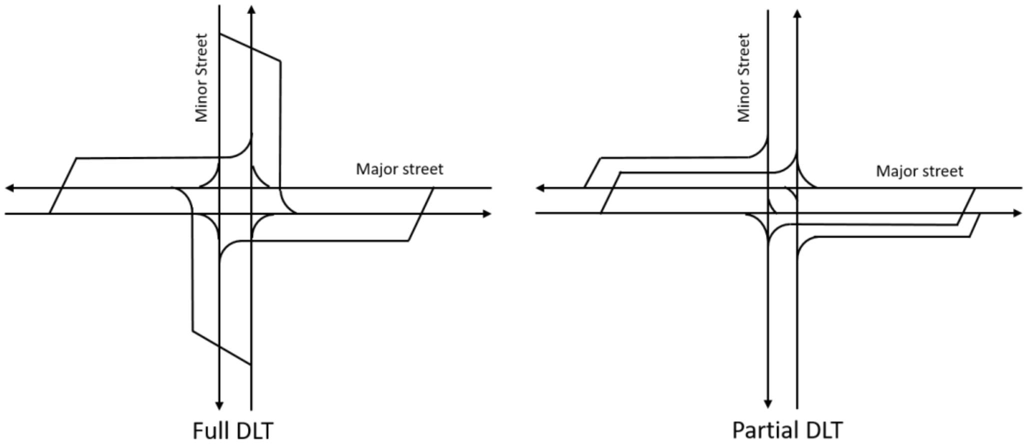


Figure 2.11 Geometry of DLTs.

Table 2.6 summarizes the CMFs values calculated by recent research studies. VDOT proposed a fatal-and-injury CMF of 0.81 for converting traditional intersections to DLTs in the brochure Virginia State Preferred CMF List, but the information source cannot be verified. Therefore, the recommended CMF value of VDOT was not included in the Table 2.6.

Table 2.6 Current CMFs Derived for a Partial DLT

Crash Type	Before-after With Comparison Group	Cross-sectional Analysis			Empirical Bayes
		Low traffic volumes (DVMT = 3000)	Moderate traffic volumes (DVMT = 6000)	High traffic volumes (DVMT = 9000)	
Total crashes	1.11**	1.492***	1.545***	1.577***	0.88
Fatal-and-Injury	1.22**	1.377***	1.416***	1.439***	
PDO	1.07**	1.71***	1.791***	1.84 ***	
Single vehicle	1.52**	1.669***	1.745***	1.791***	
Non-motorized	0.612	0.609*	0.583*	0.569*	
Angle	1.244	1.366*	1.404*	1.426*	
Rear-end	0.946	1.504***	1.558***	1.591***	
Head on	0.713	1.751	1.839	1.891	
Sideswipe same direction	1.11**	1.492***	1.545***	1.577***	

*** significant at 99% confidence level, **significant at 95% confidence level, and * significant at 90 % confidence

CMFs for MUTs

A total of three types of MUT have been studied by researchers. Type A is a MUT with crossovers constructed at both directions downstream the main intersection. Type B MUT has two additional reverse U-turn lanes near the main intersection on top of the downstream crossovers. Type C MUT resembles a type B MUT except that the U-turn lanes were constructed close to the crossovers instead of main intersection. Figure 2.12 showed the layouts of the three types of MUT. No CMFs have been developed for a full MUT yet.

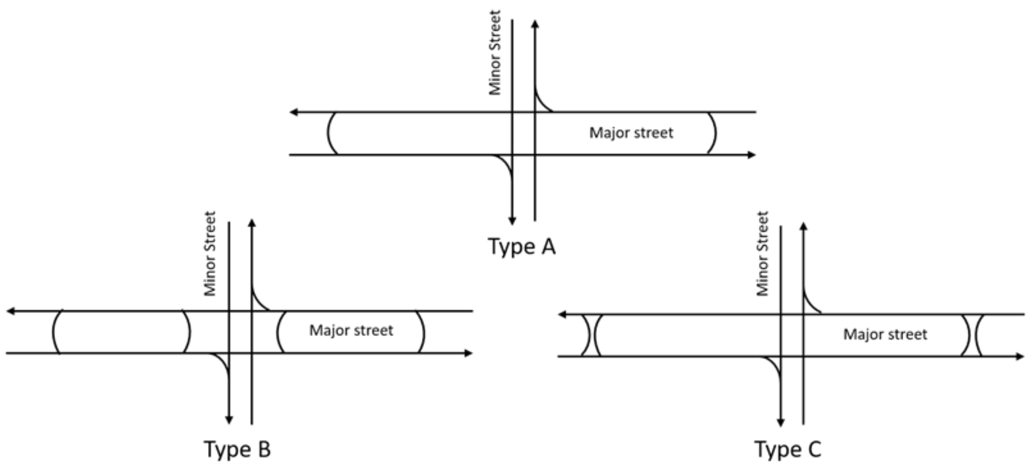


Figure 2.12 Geometry of MUTs.

Many papers have discussed the safety impact of a MUT. Only those directly discussed the CMF value of MUTs were considered in this paper. A total of 3 documents have been found deriving the CMFs of MUT. Hummer 2020 and VDOT also discussed the CMF of MUT, but their value was derived from the source provided by Reid et al. 2014 in the Median U-turn intersection: informational guide, thus these three documents were considered as in one. Table 2.7 concluded the detailed information of reviewed documents. And Table 2.8 summarized the current CMF developed for two types of MUTs.

Table 2.7 Previous Studies Deriving CMFs for MUT

Study	Safety Analysis Method	MUT Type
Al-Omari et al. 2020	Before-after with comparison group, Cross sectional analysis	Type A, Type B
Azizi and Sheikholeslami 2013	Empirical Bayes	Type C
Reid et al. 2014	Non-specify	Non-specify

Table 2.8 Current CMFs Derived for the MUTs

Crash Type	Type A	Type B	Type C	Non-specified
Total	0.6330***	0.7175***	1.132***	0.844
Fatal-and-injury	0.7732***	0.7029***		
Injury	0.7548***	0.6296***		0.702
PDO	0.5984***	1.4447**		0.912
Single vehicle	1.3800**	0.6108***		
Angle	0.6835***	0.3342***		
Head-on	0.2559***	0.1788***		
Head-on left-turn	0.1719***	0.5158***		
Rear-end	0.5258***	0.3940***		
Rear-end left-turn	0.3942***	1.2337		
Rear-end right-turn	0.9361	1.1316		
SD sideswipe	0.9155	0.1269***		
OD sideswipe	0.2167***	1.9576***		
Non-motorized	2.2432***	1.3877		

*** significant at 99% confidence level, **significant at 95% confidence level, and * significant at 90 % confidence

CMFs for RCUTs

Seven studies that estimated the CMFs for RCUTs were recognized by the FHWA founded website Crash Modification Factor Clearinghouse. Table 2.9 presented the CMF developed for RCUTs with highest quality score.

Table 2.9 Current CMFs Derived for the RCUTs

Crash Type	CMF
Total	0.7632***
Fatal-and-injury	0.5669***
Injury	0.5726***
PDO	0.8414*
Single vehicle	1.3079
Angle	0.5854***
Head-on	0.0667***
Rear-end	0.7511**
SD sideswipe	0.9291
OD sideswipe	0.3299***

*** significant at 99% confidence level, **significant at 95% confidence level, and * significant at 90 % confidence

CHAPTER 3

METHODOLOGY

This chapter explains the details of the three-stage selection model proposed in this dissertation. The structure of the model is arranged as follows. Stage 1, the initial stage, is developed to help stakeholders and planners identify: (1) project objectives; (2) budget and right of way restrictions; and (3) other constraints. Stage 2, the filtering stage, establishes standards for identifying UAIDs that meet the criteria stated in Stage 1. Stage 2 also helps to screen out those UAIDs that do not meet the criteria collected from the questionnaire. Stage 3 is the analysis stage, and the last stage, where a detailed operational and safety analysis is performed. Together with the cost-benefit analysis and performance index scoring, the final selection is based on those candidate UAIDs meeting the criteria stated in Stage 1 and Stage 2. The resulting final UAID selections will assist transportation planners in making final decisions.

The following provides a more detailed discussion of each stage.

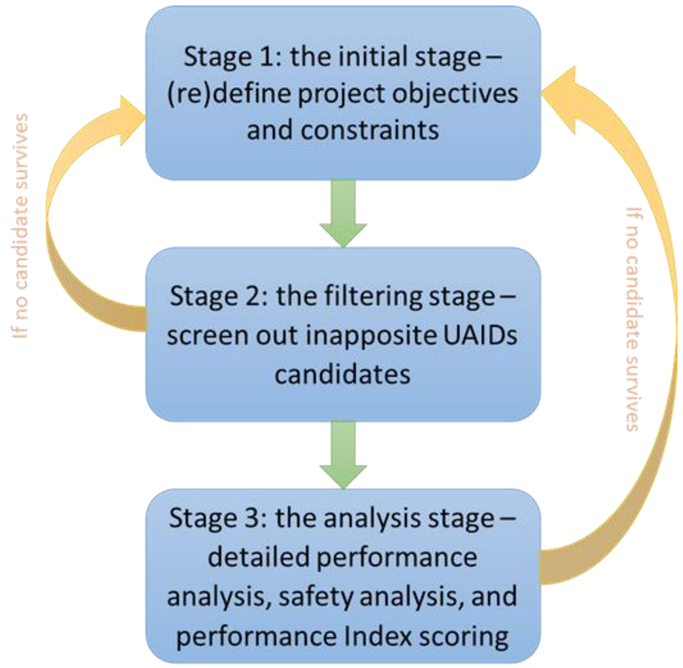


Figure 3.1 The structure of UAIDs selection model.

3.1 Stage 1: Define Project Objectives and Constraints

To capture the objectives and constraints of the intersection to be designed, the research sought to identify project categories that could be used to characterize the project type. Two sources of project categories were identified. In the report *Performance-based Analysis of Geometric Design of Highway and Streets* (Ray et al., 2014), the authors classified the stakeholders' project objectives into four categories: economic competitiveness, livable communities, safety, and state of good repair. Economic competitiveness pursues maximum economic returns on policies and investments; livable communities focus on coordinating polices and investments that increase transportation choices and access to public transportations; safety aims at reducing transportation related crashes; and state of good repair target at improving transportation infrastructure conditions. A second source of project categories is the approach used in the 2012 surface transportation bill (also known as MAP-21). MAP-21 classified project performance concerns into four categories: congestion reduction, environmental sustainability, freight movement, and system reliability ("Moving Ahead for Progress in the 21st Century," 2012). Other types of categories from different sources varied slightly according to the nature of the projects. Based on the combined sources identified, the needs of stakeholders fell under the following five categories: transportation mobility, transportation safety, economic feasibility, environmental sustainability, and community livability.

Due to lack of information and related literature, it is very difficult to quantify the impact of environmental sustainability and community livability. Therefore, the information collection in this research focused on project objectives related to transportation mobility, safety, and economic feasibility.

The collected information composed of two parts. Part 1 identifies the project objectives. If more than one project objective was identified, stakeholders are required to assign a specific weight of each objective. The weights of objectives represent the importance of this objective in the mind of stakeholders. Same project objective can be assigned a different weight representing different focus of stakeholders' concerns. An example is shown in Table 3.1.

Table 3.1 Example of Project Objectives

Project Objectives	Objective Weight
Improve intersection safety	0.4
Reduce delays	0.5
Financially feasible (cost/benefit <1)	0.1

Part 2 of the collected information composed of the project constraints. Table 3.2 summarizes the general concerns should be considered in the information collection process.

Table 3.2 Project Constraints and General Information Summary

Project Constraints
• ROW constraints
• Budget constraints
• Current vehicle volumes, especially for the left-turn movement (Peak and Non-peak)
• Current intersection geometries
• Desired intersection LOS
• Intersection accessibility
• Friendly to pedestrians and bicycles
• Friendly to public transportations
• All the other concerns (please specify)

3.2 Stage 2: Preliminary Screen of the UAIDs Candidates

As mentioned in the previous chapter, approximately 30 UAIDs have been identified in the literature. As the cost of construction for these designs can be very costly, it is therefore essential that an analysis of each design be performed before selecting the most appropriate UAID to be used. Therefore, to save time, energy and money, Stage 2 will help to screen out UAIDs that will not prove effective before moving to the detailed analysis.

Two parameters were chosen for screening UAID candidates: ROW constraints and intersection capacity. The following provides a procedure for checking the ROW constraints before analyzing intersection capacity. The *Alternative Intersections/ Interchanges : Information Report (AIIR)*, by Hughes (Hughes et al., 2010) proposed Table 3.3 as a way to qualitatively analyze the ROW requirement of UAIDs. In this table, Hughes classified the existing median width of the current intersection as "sufficient" or "insufficient" and classified the affordability of additional ROW required as "affordable" or "costly". Each candidate UAID identified after the screening process in Stage 1 would then be evaluated based on an empirical judgement and placed under the four categories (sufficient, insufficient, affordable, and costly). However, the definition of "Sufficient" and "Costly" is not clearly defined. There exists a considerable probability that a well-performing UAID candidate design could be ruled out because of this cursory analysis. In this section, a more detailed analysis is provided that will check the capacity of each UAID before checking ROW constraints.

Table 3.3 Qualitative Assessment Accommodation, Affordability, and Availability

Adequacy of Median Widths to Accommodate U-Turns	Affordability of Additional Right-of-Way Required	Viable Alternative Intersection Design to Consider Further
Sufficient	Affordable	MUT
		RCUT
		DLT
		Roundabout
		QR
Sufficient	Very costly	MUT
		RCUT
		DLT
Insufficient	Affordable	MUT
		RCUT
		DLT
		Roundabout
		QR
Insufficient	Very costly	MUT with loons
		RCUT with loons

Source: (Hughes et al., 2010)

In Stage 2, a carefully designed "break-down" test was developed to determine the capacity range of each UAID candidate. The break down test is aimed to determine when and where a UAID will break down. As a definition, "break-down" is defined that at least one movement of the candidate UAID reaches LOS E. By understanding when and where the intersection will breakdown helps planners understand the operating mechanism of each UAID and will therefore allow the ROW to be selected appropriately.

Identifying where break down occurs was performed using the simulation software VISSIM. Three UAIDs were studied: Displaced left-turns (DLT), Median U-Turn and Restricted Crossing U-Turn intersections. Volumes for the main arterial, cross-street and turning volumes were simulated under increasing volume conditions.

The analysis was also performed using equal volumes on the major and minor street, as well as under the case when the volumes were not balanced and there was a higher volume on the major street. The performance of DLT and MUT is similar under balanced volume and unbalanced volume. For illustration purposes only the balanced volume will be discussed in detail for the DLT and MUT in this research. Table 3.4 shows the volume conditions for the DLT and MUT and Table 3.5 shows the volume conditions for the RCUT.

Table 3.4 Vehicle Volume and Left-Turn Percentage Combination for DLT and MUT - Balanced Volume

Major Street		Minor Street	
Volume	Left-turn Percentage	Volume	Left-turn Percentage
1000	10%	1000	10%
	20%		
	30%		
1500	10%	1500	10%
	20%		
	30%		
2000	10%	2000	10%
	20%		
	30%		

Table 3.5 Vehicle Volume and Left-Turn Percentage Combination for RCUT - Unbalanced Volume

Major Street		Minor Street	
Volume	Left-turn Percentage	Volume	Left-turn Percentage
1000	10%, 20%, 30%	500	10%
1500	10%, 20%, 30%		
2000	10%, 20%, 30%		
1000	10%, 20%, 30%	1000	10%
1500	10%, 20%, 30%		
2000	10%, 20%, 30%		

By increasing the vehicle volume and the left-turn movement percentage, the LOS of each UAID and its movements were recorded. A total of seventy-five simulation runs were performed. The detailed test results will be discussed in Chapter 4.

To demonstrate how the results from these simulation runs will be used to identify appropriate UAID suitable for a location, the following example is provided. Table 3.6 provides a portion of the breakdown test table developed for DLT. The table shows the DLT's LOS for each movement at primary intersection and crossovers under 10% left turn volume condition. No LOS E, which is what we have defined as breakdown, is observed with volume conditions 1000 vph and 1500 vph. However, at the highest volume conditions analyzed, 2000 vph, LOS E conditions do occur for the through movement at the primary intersection. Therefore, the DLT design is feasible for volumes at 1000 vph and 1500 vph with 10% left turns. The DLT would not be appropriate for use for volume conditions in 2000 vph and 10% left turns. If one intersection has a volume of 2000 vph and 10% left turn volume, the DLT design will be excluded from future consideration.

Table 3.6 LOS for DLT Movements with Two Through Lanes, 100m Intersection Spacing and Balanced Volume (Example)

Left Turn %	*10%*		
Volume:	*1000-1000*	*1500-1500*	*2000-2000*
MOVEMENT	***LOS***	***LOS***	***LOS***
Primary intersection: NB Left	A	A	A
Primary intersection: NB Through	B	C	E
Primary intersection: NB Right	A	A	B
Primary intersection: SB Left	A	A	A
Primary intersection: SB Through	B	C	E
Primary intersection: SB Right	A	A	A
Primary intersection: EB Left	B	B	B
Primary intersection: EB Through	C	C	D
Primary intersection: EB Right	A	A	A
Primary intersection: WB Left	B	B	B
Primary intersection: WB Through	C	C	D
Primary intersection: WB Right	A	A	A
Primary intersection	B	C	D
NB Crossover Through	A	A	A
NB Crossover Left	C	C	D
SB Crossover Through	A	A	A
SB Crossover Left	C	C	D
EB Crossover Through	A	A	A
EB Crossover Left	C	C	D
WB Crossover Through	A	A	A
WB Crossover Left	C	C	D

The complete breakdown test tables will cover a wider range of volume and left-turn percentages for more alternative intersection designs. By utilizing the breakdown tables, transportation planners will exclude alternative intersection designs that would not adequately handle volume projected volume conditions. If the traffic volume of the target intersection is below the lowest threshold, 1000 vph, it will be treated as 1000 vph during the breakdown test. If the traffic volume of the target intersection exceeds the highest threshold, 2000 vph, it will be treated as 2000 vph during the breakdown test. For traffic volumes between 1000 vph and 2000 vph, they will be rounded up to the nearest threshold (i.e., 1000 vph, 1500 vph, 2000 vph).

The left-turn percentage follows the same rule.

The identification of break down conditions will also be performed looking at the weakest location of each UAID. By knowing where the UAID will breakdown first, planners will be able to evaluate the ROW constraints accordingly. Summary tables were also developed to show the movements of each UAID and their LOS. Table 3.7 shows an example.

Table 3.7 LOS for Critical Movement of UAIDs With Two Through Lanes, 100m Intersection Spacing and Balanced Volume (example)

Left Turn%	*UAID*	*Critical movement*	*Vehicle Volume*		
			1000	*1500*	*2000*
10%	DLT	Left-turn movements at the crossover	C	C	D
	MUT	Through movement at the median openings	A	A	E
	RCUT	U-turn movement at the median openings	B	C	C
20%	DLT	Left-turn movements at the crossover	C	C	D
	MUT	Through movement at the median openings	A	B	F
	RCUT	U-turn movement at the median openings	B	B	C
30%	DLT	Left-turn movements at the crossover	E	E	E
	MUT	Through movement at the median openings	A	B	F
	RCUT	U-turn movement at the median openings	B	B	C

From the Table 3.7, the bottleneck of a DLT can be identified as associated with the crossover. The conclusion from the review of this table is if one wishes to improve the performance of a DLT, a wider median to accommodate the crossover would be more efficient than many other treatments. These results are the same for the RCUT. For the MUT, there is no requirement for a wider median width because the bottleneck area is the through lanes at the median openings. A relatively narrow median with more through lanes would be better for heavy volume conditions.

With the bottleneck area of each UAID identified and Hughes' qualitative assessment table, planners will be able to evaluate their ROW constraints accordingly and screen out those designs that cannot be designed within the constraints of the intended location.

3.3 Stage 3: Detailed Mobility, Safety and Economic Analysis of UAIDs Candidates

In Stage 3, all the UAIDs found to be suitable for use in Stage 2 will undergo a detailed mobility, safety, and cost analysis. Control delay is the chosen performance measure for the mobility analysis. Crash rate is used for the safety analysis. After the three analyses are completed, two approaches will be used to simultaneously rank the candidate UAIDs. The first approach is a scoring system. The UAIDs will be scored for their mobility, safety, and cost and then a performance index will be calculated. The UAID with the lowest performance index will be the optimal solution. The second ranking system is a cost-benefit analysis. The reductions of control delay and crash rate compared to a traditional intersection will be converted to monetary benefits; the construction cost and other negative impacts (i.e., the increase of control delay and crash rate increase) will be counted as monetary cost. The UAID with lowest cost-benefit ratio will be the optimal solution.

3.3.1 Mobility Analysis

In this study, control delay is the chosen performance measure for estimating mobility of the UAID. While average speed and capacity are also popular measures for mobility, these measures were not considered as they do not fully capture mobility at UAIDs. As this study focused on UAIDs, average speed does not capture the extra distance traveled by vehicles in the UAID when compared to a conventional intersection. This extra distance travel time is an important component of the UAID's controlled delay. Capacity is accounted for in the control delay calculations and therefore capacity as a separate measure is not warranted. Considering the above facts, control delay is the final choice of performance measure.

The HCM 6th edition proposed a methodology to calculate control delays for DLTs, MUTs, and RCUTs. The control delays for other UAIDs have not yet been developed in the HCM. Therefore, only DLT, MUT, and RCUT are considered in this research.

As mentioned previously in the literature review, the UAID LOS calculation methodology was based on the approach used in the HCM for determining LOS for conventional signalized intersections. By modifying the adjustment factors of saturation flow rate and by including a calculation of Experienced Travel Time, a ten-step LOS computing method is developed. The ten steps included the following: (1) Determine O-D demands and movement demands, (2) Determine lane groups, (3) Determine lane utilization, (4) Signal progression adjustments, (5) Additional control-based adjustments, (6) Estimate junction-specific performance measures, (7) Calculate Extra Distance Travel Time, (8) Estimate additional weaving delay, (9) Calculate Experienced Travel Time, and (10) Calculate LOS.

Two critical steps of this methodology are the calculation of the UAID's saturation flow rate and the calculation of the experienced travel time for each movement. The calculation of the saturation flow rate is performed using Equations

(3.1) and (3.2).

$$S = S_0 * N * (f_{adj}) \tag{3.1}$$

Where: S = saturation flow rate,

S_0 = base saturation flow rate for ideal conditions,

N = number of lanes in lane group,

f_{adj} = adjustment for non-ideal conditions.

$$f_{adj} = f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \tag{3.2}$$

Where: f_w = adjustment for lane width,

f_{HV}= adjustment for heavy vehicles,

f_g= adjustment for approach grade,

f_p= adjustment for existence of parking adjacent to lane group,

f_{bb}= adjustment for bus stops within the intersection area,

f_a= adjustment for area type,

f_{LU}= adjustment for lane utilization,

f_{LT}= adjustment for left turns in lane group,

f_{RT}= adjustment for right turns in lane group,

f_{Lpb}= pedestrian/bicycle adjustment for left turn group, and

f_{Rpb}= pedestrian/bicycle adjustment for left turn group.

Equations (3.1) and (3.2) hold true for both conventional intersections and alternative intersections. Most of the adjustment factor calculations stayed the same between the UAID and a conventional signalized intersection except for the lane utilization adjustment f_{LU}. Due to the unique geometry of alternative intersections, most drivers tend to place their vehicles in the target lane in advance to avoid multiple weaving maneuvers. In this case, field data is preferred. If no field data is available, the f_{LU} is calculated using an estimation procedure identified by HCM. The same tactic applied to the estimation of signal progression adjustments and additional control-based adjustments.

The experienced travel time for the intersection is estimated by the summation of control delay and extra distance travel time for each O-D movement:

$$\mathrm{ETT} = \sum d_i + \sum EDTT \tag{3.3}$$

Where: d_i = the control delay at junction i.

EDTT = the extra distance travel time experienced for each movement.

Extra distance travel time for the DLT is considered negligible. For MUTs and RCUTs, Equation (3.4) gives an estimation of MUTs and RCUTs operated under STOP signs and signals. For RCUTs operated under merges, extra consideration of delay associated with deceleration into a turn and acceleration from the turns is added to the ETT calculation, as Equation (3.5) indicates.

For RCUTs and MUTs with STOP signs and signals, the EDTT is determined as follows:

$$\mathrm{EDTT} = \frac{D_t + D_f}{1.47 \times S_f} \tag{3.4}$$

For RCUTs operated under merges, the EDTT is determined as follows:

$$\mathrm{EDTT} = \frac{D_t + D_f}{1.47 \times S_f} + a \tag{3.5}$$

where: EDTT = extra distance travel time (s),

D_t = distance from primary intersection to the U-turn crossover (ft),

D_f = distance from the U-turn crossover to the primary intersection (ft),

1.47 = conversion factor from mi/h to ft/s

S_f = major-street free-flow seed (mi/h), and

a = delay associated with deceleration into a turn and acceleration from the turn (s).

To efficiently identify the LOS of UAIDs, a series of service volume tables are also developed to assist in screening out inappropriate UAIDs. By utilizing the service volume tables, transportation planners can approximate the UAID's LOS. Any design that results in a LOS that exceeds the target LOS will be removed from further consideration. Detailed information about service volume tables will be included in Chapter 5.

3.3.2 Safety Analysis

To estimate the number of crashes at the UAID, we will utilize existing crash modification factors (CMFs). CMFs allow us to estimate the expected number of crashes for a geometric or operational change on a roadway. For example, if the CMF of a countermeasure for widening medians, for example,) is 0.8, and the average number of crashes before the countermeasure is 10, then the expected number of crashes after the countermeasure is given by 10×0.8 = 8. The crash reduction is 10-8 = 2. Equation (3.6) gives the general estimation of crash estimation after geometric or operational change, and Equation (3.7) gives the estimation for difference of crash rates due to UAIDs. The positive crash difference means the crash reduction rate and the negative crash difference indicates the crash increase rate.

$$Avg.Crashes\ after\ change\ = Avg.Crashes\ before\ change\ \times CMF \quad (3.6)$$

$$Crash\ difference = Avg.Crashes\ before\ change\ \times (1 - CMF) \quad (3.7)$$

The strategy to be used is to determine the change in crash frequency between a conventional intersection and a UAID. To do so, the CMF for transferring a conventional intersection to a UAID will be determined. The literature review in Chapter 2 listed all CMFs that have been developed for DLT, MUT, and RCUTs. The methodologies for developing CMFs are also reviewed. Since developing CMFs are not the focus of this research, the developed CMFs from previous studies will be adopted in this research. To make the adopted CMFs as realistic as possible, the researchers sought to adopt CMFs that were developed under similar conditions as the selected to-be-reconfigured intersection. If there were no CMFs developed under similar intersections as the intersection under study, a localized factor for CMFs f_{local} would then be applied to calibrate crash prediction. In this case, Equation (3.7) needs to be modified as shown in Equation (3.8). f_{local} is the ratio of estimated number of crashes and observed number of crashes for an intersection that is located next to the selected intersection and has same geometry designs as the selected intersection. If such an intersection is unavailable, the value of f_{local} is viewed as 1.0.

$$Crash\ difference = Avg.Crashes\ before\ change\ \times (1 - CMF \times f_{local}) \quad (3.8)$$

Where f_{local} = localization factor for CMFs.

Crash cost for various crash severities and crash types will also be investigated. By combining the reduced/increased crash rate with the crash cost, transportation planners can easily calculate the safety benefits/cost of a UAID. Detailed information is included in Chapter 6. The UAIDs with negative safety benefits will be removed from future consideration.

3.3.3 Construction Cost Estimation

The construction costs of UAIDs are also considered. The actual construction cost of a UAID highly depends on the market price construction material cost, and local labor cost. A detailed construction cost estimation during the planning stage is unfeasible, therefore, the posted construction cost of a similar project will be used as an approximate cost in this research. The selected intersection construction cost should be adjusted to local material cost and labor cost and should also be adjusted for the same dollar value for the year of construction.

3.3.4 Cost-benefit Ratio Analysis

The cost-benefit ratio is the chosen indicator used in this research to help planners evaluate the candidate UAIDs in a monetary approach. The lower the cost-benefit ratio, the more attractive the candidate UAID.

Cost-benefit ratio is a ratio obtained by the total benefits (expressed in dollars) of the project divided by the total cost (expressed in dollars) of the project. In this case, the benefit of the UAID is the delay and crash reduction, and the cost of the UAID is the construction cost. Equation 3.9 shows the calculation of cost-benefit ratio in this analysis. Annual delay increase cost and annual delay reduction cost cannot be both positive in one equation. In other words, if the annual delay reduction benefit is positive, the annual delay increase cost should be zero, and vice versa. The annual crash increase cost and annual crash reduction benefit follow the same rule.

$$\text{Cost} - \text{benefit ratio} = \frac{annual\ construction\ cost\ (\$)}{annual\ delay\ reduction\ benefit(\$) + annual\ crash\ reduction\ benefit(\$)} \tag{3.9}$$

USDOT initiated a study on Value of Travel Time (VTT) estimation to assist all DOTs in using cost-benefit analysis (need to state authors and year). This research was initiated in 1997 and the latest version was released in 2016. Equations (3.10) and (3.11) give the estimation of annual benefit due to the reduction in delay. To be noted, the number of vehicle volumes used in Equation (3.11) are the average vehicle volumes for this alternative intersection during its designed service life cycle.

$$\text{Annual delay reduction benefit} = \frac{Delay\ reduction}{C} \times VTT \times Persons\ Volume \times 8760 \tag{3.10}$$

$$\text{Persons Volume} = \text{Vehicle volumes} \times \text{occupation rate} \tag{3.11}$$

Where:

Delay reduction = delay reduced by UAID in a circle length (s),

C = circle time (s),

Persons volume= total number of drivers and passengers entering the intersection per hour,

Vehicles volume = the average total number of vehicles entering the intersection per hour during the designed service life of UAID, Occupation rate = average number of persons in a vehicle, and

8760 = number of hours in a year.

The annual benefit of crash reduction can be derived by total number of crashes per year times average cost of crash. Equation (3.12) shows the calculation.

$$\text{Annual crash reduction benefit} \qquad (3.12)$$
$$= \text{annual number of crashes } \times \text{average crash cost}$$

Unlike the delay increase cost and crash increase cost, the construction cost is a non-recurring cost that happens at the construction stage of a UAID. In general, the designed service life of UAIDs are 30 years. To better compare the construction costs with other costs in the cost-benefit analysis, the construction cost should also be converted to an annual value. For simplification, the annual construction cost mentioned in this research is the summary of annualized construction cost and annual maintenance cost. The equation is listed below.

$$\text{Annual construction cost} \qquad (3.13)$$
$$= \text{ construction cost } \times \left[\frac{i}{1-(1+i)^{-n}}\right]$$
$$+ \, annual\ maintenance\ cost$$

Where:

Construction cost = the present value of the cost for construct UAID,

i = interest rate, and

n = designed service life of UAID.

With cost-benefit ratio available, the remaining candidates will be ranked in the ascending order and the candidate with smallest cost-benefit ratio should be the best option.

3.3.5 Performance Index Scoring System

The ranking of the candidate UAIDs will be dependent on the scoring system. A Performance Index (PI) is designed to score each UAID. The calculation of the PI is shown in the following expression.

$$PI = W_1 \times B_1 + W_2 \times B_2 - W_3 \times C_1 \qquad (3.14)$$

Where :

W_1, W_2, W_3 = the weight assigned by the stakeholders for delay reduction, crash reduction and construction cost, respectively in Stage 1,

B_1 = the annual delay reduction benefit in million dollars,

B_2 = the annual crash reduction benefit in million dollars, and

C_1 = the annual construction cost in million dollars.

The higher the PI, the more attractive is the candidate UAID. Unlike the cost-benefit analysis, the performance index sufficiently considers the stakeholders' concerns and can help in better selecting the best UAID.

CHAPTER 4

BREAKDOWN TEST AND RESULT DISCUSSION

The purpose of this breakdown test was to identify when and where UAIDs will break down. To find the failure point, each UAID was tested with different combinations of vehicle volume and turn-movement percentage. Since existing UAIDs operate under limited volume ranges or have not been built, it was difficult to obtain enough field data. Therefore, simulation data was used in this dissertation to collect data on break down conditions. The simulation tool VISSIM (version 10) and signal optimization tool Synchro Studio 10 was used in this analysis. Tables 3.4 and 3.5 show vehicle volume and turn-movement percentage combinations for DLT, MUT, and RCUT used in performing the simulation. Each volume and turn-movement percentage simulation were run for 5 times with a one-hour simulation time and different random seeds. The left-turn percentage of minor street was set as 10%. The VISSIM settings used in the simulation included the following:

- Lane width: 12 ft.
- Driver characteristics, average speed: default value.
- Truck percentage: 2%.
- Signal plans: a pre-timed signal controller with 4 s amber and 1 s all-red intervals.

All signal plans followed best signal timing practices as found in *Signalized Intersection – Information Guide* (Rodegerdts et al., 2004). DLTs followed the six-phasing signal-timing plan; MUTs adopted the simple two-phasing plan, and RCUT (also named as “superstreets” in the *Signalized Intersection – Information Guide*) had a two-phase plan with protected left-turn phase.

Signal optimization: Esawye (2007) demonstrated Synchro’s optimizing signal-timing plans perform better than simple progression. In this dissertation, the optimization strategies for Synchro and VISSIM’s were utilized. As VISSIM optimizes cycle length under a preset cycle time and offset, while Synchro optimizes cycle time, cycle split and offset, Synchro Studio 10 was the final optimization approach used in this dissertation. Delay measurement: the sensor module Node was used to collect delays at each intersection and crossovers.

To make the simulation results comparable between UAIDs, all the tested UAIDs possessed similar geometric design. Detailed Geometric design information are as follows:

- All intersections were four-legged intersections.
- Each approach to the intersection had the same number of lanes and lane types: one exclusive left-turn lane, two through-only lanes and one channelized right turn lane.
- DLTs exclusive left turn lanes were 70m (232 feet) in length and RCUTs were 100m (328 feet) in length. The design of MUTs did not have exclusive left turn lanes.
- DLTs channelized right turn lanes were 130m (427 feet) in length; MUTs were 200m (656 feet), and RCUTs were 100m (328 feet).
- Intersection spacing for DLTs is 90m. For MUTs and RCUTs the signal spacing used is 200m (El Esawey & Sayed, 2013).

4.1 DLT Failure Test Result Discussion

As previously stated in the literature review, the full DLT comprises of one primary intersection and four secondary intersections as indicated by Figure 4.1. To investigate the bottleneck of DLT, its primary intersection and the secondary intersections are evaluated independently. For the DLT design, both balanced and unbalanced vehicle volumes were tested. As the performance under these two conditions was similar, only detailed results for the balanced volume condition is presented and discussed in this research. The details of the results for the balanced volume is presented later in this section.

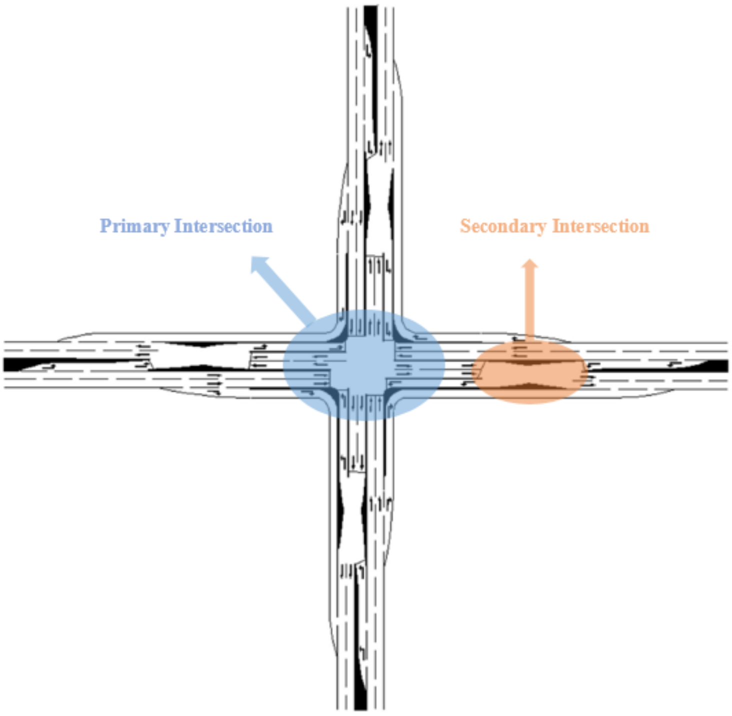


Figure 4.1 Geometry of simulated DLT.

For this simulated DLT intersection, two trends were found from the results: (1) the LOS for both the primary intersections and secondary intersections deteriorate as volume increases under the same left turn movement percentage; and

(2) the LOS of the primary intersection improves, and secondary intersection deteriorates as the left turn movement percentage increases while the volume remains the same. The first trend is not surprising as increased volumes leads to deteriorated LOS. The second trend occurs because as the left turn movement percentage increases, the secondary intersections (the crossovers at the median), which is designed to reroute left turn vehicles, becomes congested. For through movement vehicles entering the primary intersection, they must first pass through the secondary intersection. As the secondary intersection becomes congested, it becomes a bottleneck for these through vehicles which are queued at the secondary intersection. This queuing condition restricts the through vehicles entering the primary intersection, resulting in an improvement in the LOS at the primary intersection as the LOS at the secondary intersection deteriorates.

The failure of DLT movements (those movements operating under LOS E) occurs with the major street volume of 2000 vph and under all left turn movement percentages. Failure also occurs when there are 30% left turns for all volume levels. The through movements at the primary intersection may also fail under 2000 vph with 10% left turn movements. Under all volume and left-turn percentage combinations, the left-turn movement at the crossover is one of the worse operating movements of the DLT. This movement becomes critical as the left turn movement percentages increases. Table 4.1 shows the LOS by movement under three volume conditions and three left-turn percentages.

Table 4.1 LOS of All Movements in the Simulated DLT

Left Turn %	*10%*			*20%*			*30%*		
Volume:	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*
MOVEMENT	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*
Primary intersection: NB Left	A	A	A	A	A	A	A	A	A
Primary intersection: NB Through	B	C	E	B	B	D	C	C	C
Primary intersection: NB Right	A	A	B	A	A	A	A	A	A
Primary intersection: SB Left	A	A	A	A	A	A	A	A	A
Primary intersection: SB Through	B	C	E	B	B	D	C	C	C
Primary intersection: SB Right	A	A	A	A	A	A	A	A	A
Primary intersection: EB Left	B	B	B	B	B	B	B	A	B
Primary intersection: EB Through	C	C	D	C	C	C	B	B	B
Primary intersection: EB Right	A	A	A	A	A	A	A	A	A
Primary intersection: WB Left	B	B	B	B	B	A	B	A	B
Primary intersection: WB Through	C	C	D	C	C	C	C	B	C
Primary intersection: WB Right	A	A	A	A	A	A	A	A	A
Primary intersection	B	C	D	B	B	C	B	B	B
NB Crossover Through	A	A	A	A	A	A	A	A	A
NB Crossover Left	C	C	D	C	C	D	E	F	D
SB Crossover Through	A	A	A	A	A	A	A	A	A
SB Crossover Left	C	C	D	C	C	D	E	F	E
EB Crossover Through	A	A	A	A	A	A	A	A	A
EB Crossover Left	C	C	D	C	C	D	E	E	E
WB Crossover Through	A	A	A	A	A	A	A	A	A
WB Crossover Left	C	C	D	D	D	F	F	E	F

To view the trends concluded from Table 4.1 more intuitively, Table 4.2 (a) and 4.2 (b) were extracted from the Table 4.1. Table 4.2 (a) shows the LOS for the primary intersection under various vehicle volume and left turn percentages. Table 4.2 (b) shows the LOS for the critical movement in the DLT – left-turn movement at eastbound crossover (as indicated by Figure 4.2). The left-turn movement at the east bound crossover become critical because during the simulation, the left turn bay at the east bound crossover is always the first place to become congested. Improving the LOS at the left turn bay will not only improve the mobility of left turn vehicles but also benefit the overall performance of DLT. The left-turn movements at the other three crossovers showed similar patterns to this critical movement. The difference between the LOSs among the four crossovers may be caused by the different intersection spacing between the Primary intersection and crossovers.

Table 4.2 (a) LOS for the Primary Intersection of the DLT – Balanced Volume

Left Turn%	*Volume:*	*1000*	*1500*	*2000*
	INTERSECTION	*LOS*	*LOS*	*LOS*
10%	Primary intersection	B	C	D
20%	Primary intersection	B	B	C
30%	Primary intersection	B	C	B

Table 4.2 (b) LOS for the eastbound critical movement of the DLT – Balanced Volume

Left Turn%	*Volume:*	*1000*	*1500*	*2000*
	CRITICAL MOVEMENT	*LOS*	*LOS*	*LOS*
10%	EB Crossover Left movement	C	C	D
20%	EB Crossover Left movement	C	C	D
30%	EB Crossover Left movement	E	E	E

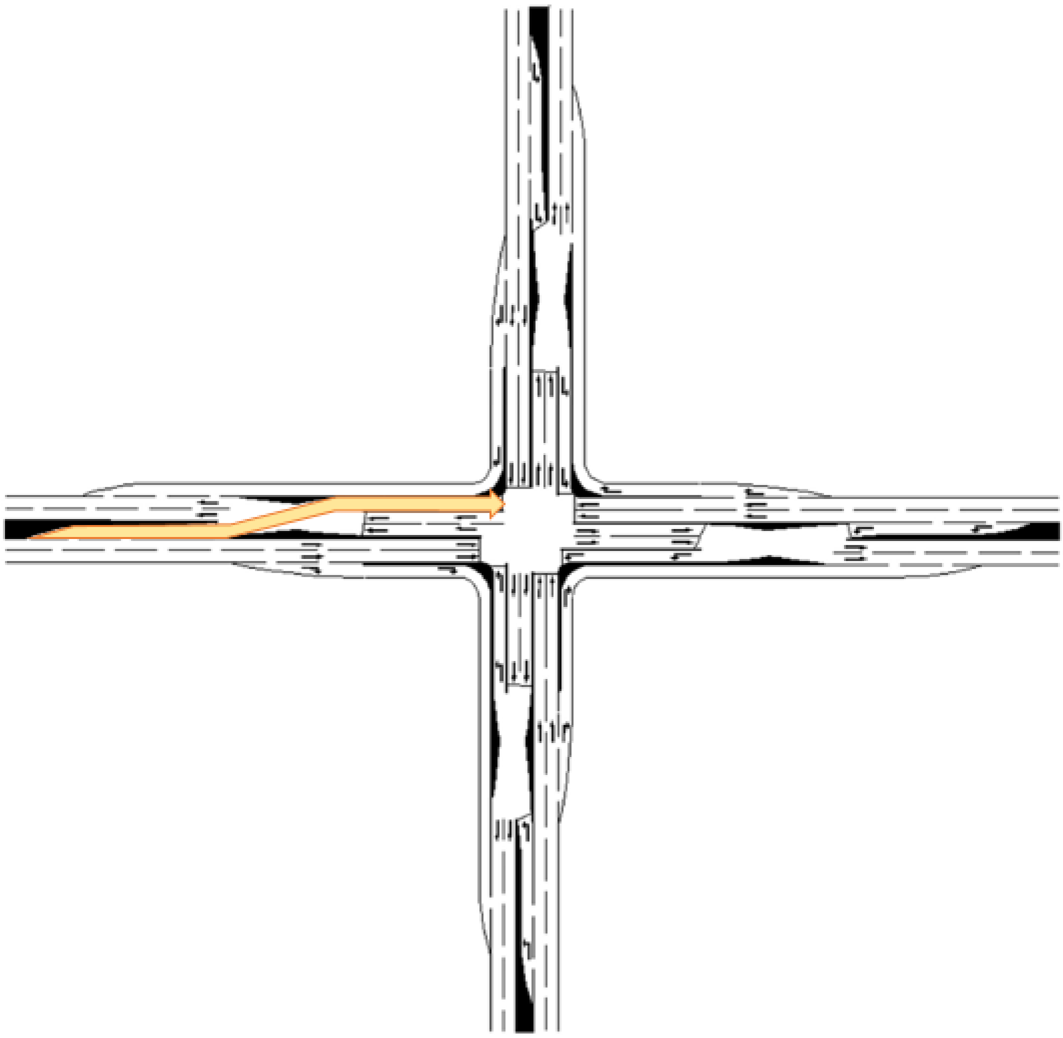


Figure 4.2 Critical movement of eastbound DLT.

The simulation under unbalanced volume leads to the same conclusion. Tables 4.3 (a) and 4.3 (b) show the results. For the following UAIDs, if no different conclusions were drawn under unbalanced volume situation, no special discussion will be provided in this chapter.

Table 4.3 (a) LOS of DLT for Primary Intersection – Unbalanced Volume

Left turn%	*Volume:*	*1000-500*	*1500-1000*	*2000-1500*
	INTERSECTION	*LOS*	*LOS*	*LOS*
10%	Primary Intersection	B	B	C
20%	Primary Intersection	B	B	C
30%	Primary Intersection	B	B	B

Table 4.3 (b) LOS for the Eastbound Critical Movement of DLT – Unbalanced Volume

Left turn%	*Volume:*	*1000-500*	*1500-1000*	*2000-1500*
	CRITICAL MOVEMENT	*LOS*	*LOS*	*LOS*
10%	EB Crossover Left movement	B	B	C
20%	EB Crossover Left movement	B	C	C
30%	EB Crossover Left movement	C	D	E

From Tables 4.3 (a) and 4.3 (b), it can be concluded that the LOS of both the primary intersection and the crossovers degrades as the volume increases. As the left-turn percentage increases, the traffic congestion at the crossover deteriorates while the primary intersection stays the same or improves slightly. This occurs because many left turn vehicles are blocked at the crossover and cannot pass through to entering the primary intersection. It is also very noteworthy that the LOS of left turns at the crossover are always higher than the LOS of primary intersection.

4.2 MUT Failure Test Result Discussion

Equivalent to the DLT, the simulated MUT is comprised of one primary intersection and four secondary intersections as indicated by Figure 4.3. The following discussion refers to the secondary intersection as “median openings”. The primary intersection and the secondary intersections are evaluated independently to investigate the bottleneck of MUT. Both balanced volume and unbalanced volume were tested. Since there were no differences in the conclusions drawn under unbalanced volume conditions, only the result of balanced volumes is discussed below. The details on the results for the entire simulation will be presented later in this subsection.

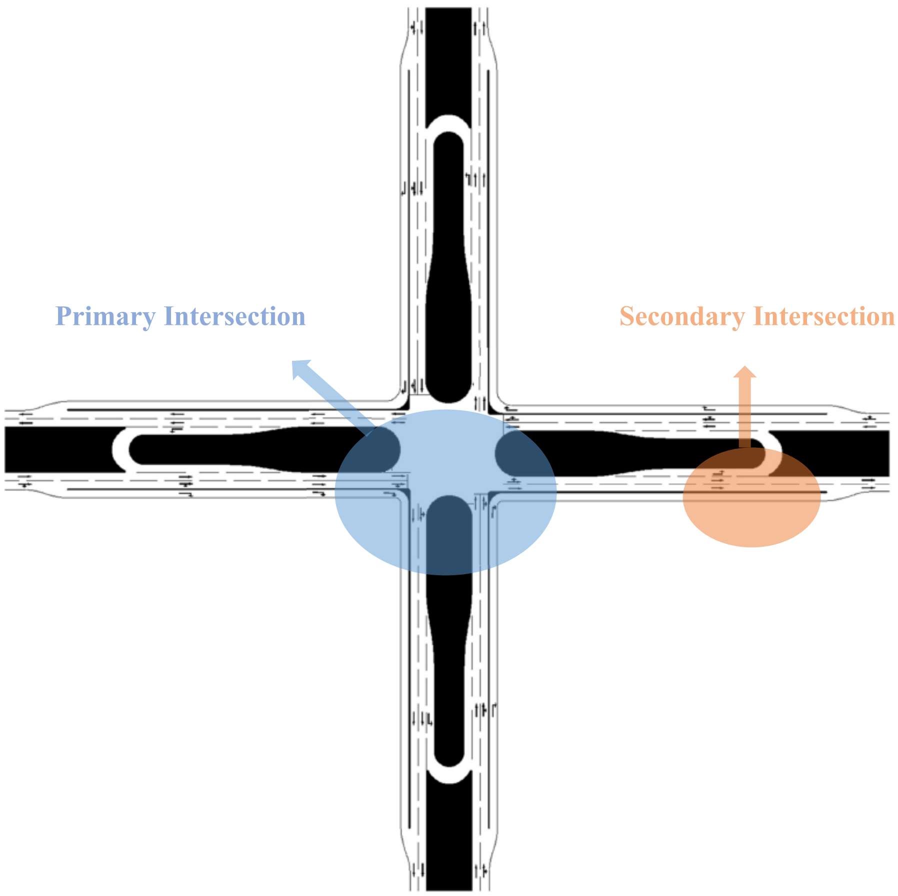


Figure 4.3 Geometry of simulated MUT.

The test results showed that (1) the simulated MUT performs well under 1000 vph and 1500 vph, but the LOS deteriorates drastically under 2000 vph for all left turn percentages. (2) For the 1000 vph and 1500 vph condition, the LOS at the secondary intersections outperforms the LOS at the primary intersection. And for the 2000 vph condition, the through movement at the secondary intersections become the worst movement. The bottleneck of MUT moves from primary intersection to the secondary intersection as the volume increases. (3) The U-turns movements at the secondary intersections perform at a LOS A under all test situations. Since a typical MUT typically has a wide median and an extra lane for merging and diverging from the U-turn movement, it is not surprising that the U-turn movement performs well.

In this simulation, no movement fails at 1000 vph and 1500 vph. Movements do fail under with the 2000 vph volume condition with all left turn movement percentages. The failed movements are the through movements at the secondary intersections. The channelized right turn for the west bound approach also fails under 2000 vph for 20% and 30% left turn movements. The slight difference among the LOS of channelized right turn movement is the consequence of different right-turn lane storage lengths.

Table 4.4 LOS of All Movements in the Simulated MUT-Balanced Volume

Left Turn%	*10%*			*20%*			*30%*		
Volume:	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*
Movement	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*
Primary intersection NB Through	B	B	C	B	B	D	B	C	D
Primary intersection NB Right	B	C	C	B	C	C	B	C	C
Primary int. NB channelized Right	A	A	B	A	A	C	A	A	C
Primary intersection SB Through	B	B	C	B	B	D	B	C	D
Primary intersection SB Right	B	C	C	B	C	C	B	C	C
Primary int. SB channelized Right	A	A	C	A	A	D	A	A	D
Primary intersection EB Through	B	C	C	B	C	D	B	C	D
Primary intersection EB Right	B	C	C	B	C	C	B	C	C
Primary int. EB channelized Right	A	A	C	A	A	D	A	A	D
Primary intersection WB Through	B	C	C	B	C	D	B	C	D
Primary intersection WB Right	B	C	C	B	C	C	C	C	C
Primary int. WB channelized Right	A	A	D	A	A	E	A	B	E
Primary intersection	B	B	C	B	C	C	B	C	D
NB Median Opening Through	A	A	C	A	A	E	A	A	E
NB Median Opening U-Turn	A	A	A	A	A	A	A	A	A
SB Median Opening Through	A	A	D	A	A	E	A	A	E
SB Median Opening U-turn	A	A	A	A	A	A	A	A	A
EB Median Opening Through	A	A	E	A	A	F	A	B	F
EB Median Opening U-turn	A	A	A	A	A	A	A	A	A
WB Median Opening Through	A	A	E	A	A	F	A	C	F
WB Median Opening Right	A	A	D	A	A	E	A	B	E

To investigate the characteristics of the bottleneck at the MUT, the primary intersection and secondary intersections were studied separately. Table 4.5 (a) shows the LOS of primary intersection. Tables 4.5(b) and 4.5 (c) present the LOS of westbound through movement which is the critical movement at MUT.

In Table 4.5 (a), the LOS of primary intersection deteriorates as the volume increases. This is also the same as the left-turn percentage increases.

Table 4.5 (a) LOS of MUT for Primary Intersection – Balanced Volume

Left turn%	*Volume:*	*1000*	*1500*	*2000*
	INTERSECTION	*LOS*	*LOS*	*LOS*
10%	Primary intersection	B	B	C
20%	Primary intersection	B	C	C
30%	Primary intersection	B	C	D

The bottleneck of the MUT varies as the volume increases. As an example, for the 1000 vph and 1500 vph volume conditions on the westbound approach, the critical movement is the through movement at the primary intersection. As the volume increases to 2000 vph, the bottleneck becomes the through movement at the westbound median opening. Figure 4.4 shows the exact location of the critical movement.

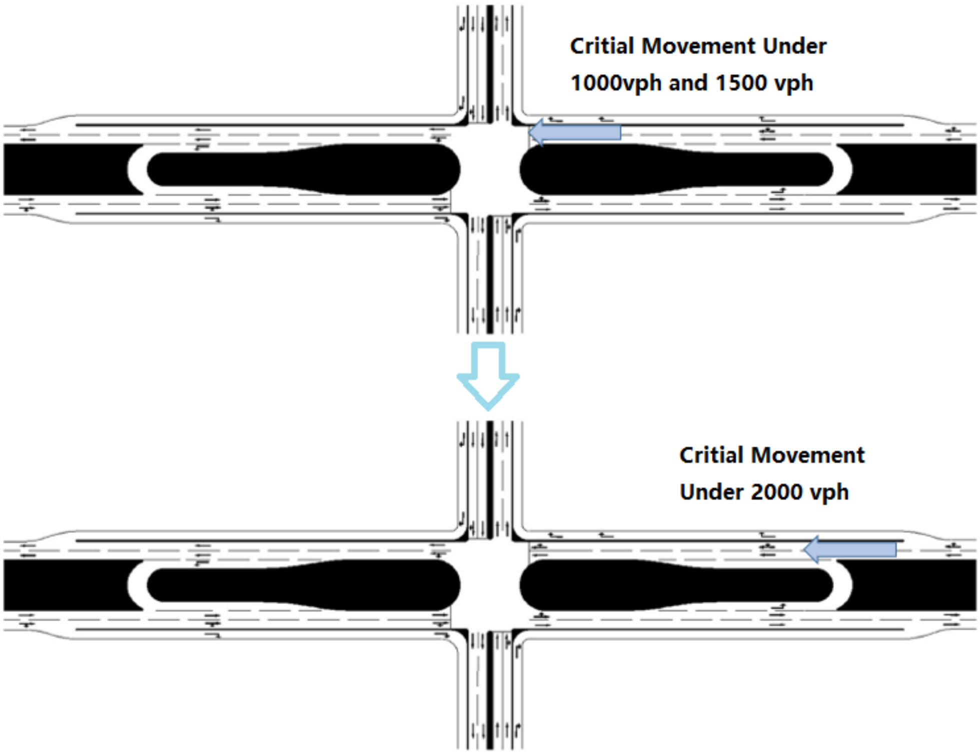


Figure 4.4 Critical movement of westbound MUT.

Table 4.5 (b) LOS of MUT for WB Critical Movement – Balanced Volume (1000vph & 1500 vph)

Left turn%	*Volume:*	*1000*	*1500*
	CRITICAL MOVEMENT	*LOS*	*LOS*
10%	Primary intersection WB Through	B	C
20%	Primary intersection WB Through	B	C
30%	Primary intersection WB Through	B	C

Table 4.5 (c) LOS of MUT for WB Critical Movement – Balanced Volume (2000vph)

Left turn%	*Volume:*	*2000*
	CRITICAL MOVEMENT	*LOS*
10%	WB Median Opening Through	E
20%	WB Median Opening Through	F
30%	WB Median Opening Through	F

Tables 4.5 (b) and 4.5 (c) indicate that the LOS of critical movement deteriorates as the volume and left-turn movement percentage increases. For low to medium volume conditions, vehicles at the median openings are cleared quickly. The LOS of the MUT highly depends on the capacity of primary intersection. With high volume conditions, even with extremely low left-turn percentages, the secondary intersection at the median opening fails and the LOS of the through movement reaches LOS E or higher. The primary intersection performs better than the median openings under high volumes since many vehicles are blocked at the median openings and cannot get through.

4.3 RCUT Failure Result Discussion

The simulated RCUT is comprised of one primary intersection and two secondary intersections as indicated by Figure 4.5. Since the RCUT is typically built to accommodate unbalanced volume, only unbalanced volumes were tested in the break down test.

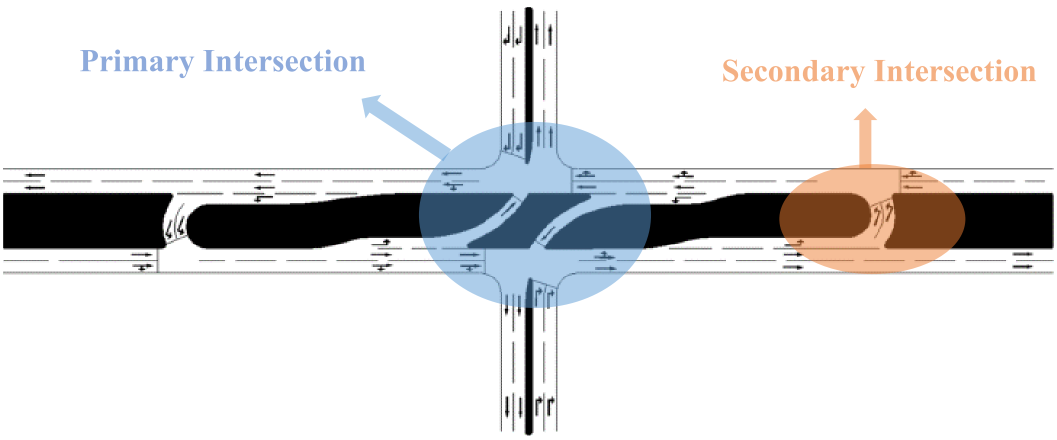


Figure 4.5 Geometry of simulated RCUT.

For the simulated RCUT, the volume of the cross street was set to 500 vph for all main street volume conditions. Main street volumes of 1000 vph, 1500 vph and 2000 vph were tested. As shown in Table 4.6, the LOS of this simulated RCUT is not significantly affected by the volume and left-turn movement percentages. Still some characteristics of the RCUT can be concluded. The overall LOS of the primary intersection outperforms the LOS of the secondary intersections. The LOS of the U-turn movement at the median openings are worse than the other movements. No break down occurs under the initial volumes evaluated. A higher cross street volume should be assessed to determine where the break point of the simulated RCUT occurs.

Table 1.6 LOS of All Movements in the Simulated RCUT-Minor Street Volume 500 vph

Left Turn%	*10%*			*20%*			*30%*		
Volume:	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*	*1000*	*1500*	*2000*
Movement	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*	*LOS*
Primary intersection NB Right	A	A	A	A	A	A	A	A	A
Primary intersection SB Right	B	B	C	B	B	C	B	B	B
Primary intersection EB Left	B	B	C	B	C	B	B	B	A
Primary intersection EB Through	A	A	A	A	A	A	A	A	A
Primary intersection EB Right	A	A	A	A	A	A	A	A	A
Primary intersection WB Left	A	A	A	A	A	A	A	A	A
Primary intersection WB Though	A	B	A	A	A	B	A	A	A
Primary intersection WB Right	B	A	C	C	B	B	C	B	A
Primary intersection	A	A	A	A	A	A	A	A	A
EB Median opening Through	B	A	A	B	A	A	B	A	A
EB Median opening U-Turn	B	B	C	B	C	C	B	B	C
EB Median opening	B	B	B	B	B	B	B	B	B
WB Median opening Through	B	B	A	B	B	A	B	A	A
WB Median opening U-Turn	B	C	C	B	B	C	B	B	C
WB Median opening	B	B	B	B	B	B	B	B	B

CHAPTER 5

SERVICE VOLUME TABLES FOR ALTERNATIVE INTERSECTIONS

The Highway Capacity Manual (HCM) provides guidelines on the development of service volume tables as a screening tool to assess the operational performance of transportation facilities. Generalized service volume tables estimate the maximum daily or hourly volume that a roadway can serve under an assumed set of conditions. These tables are useful tools for performing preliminary evaluations of facilities.

As part of this dissertation, service volume tables are proposed for use in performing a preliminary evaluation of alternative intersections. The tables will be used to identify feasible alternative intersection designs that may meet the desired LOS.

In a review of the literature on service volume tables, no service volume tables had been developed for MUTs and DLTs. North Carolina Department of Transportation developed a service volume table for the main street of a RCUT in 2010 (Joseph E. Hummer, 2010). The calculation of intersection delay, however, was not consistent with the prevailing methodology in the HCM 6th edition. In this research, a series of service volume tables were developed using methodologies consistent with the HCM methodology and easily adaptable to local conditions.

In this research, service volume tables for two-way stop controlled MUT intersections (TWSC-MUT) and partial DLT intersections were completed. An attempt to accomplish the service volume table of RCUT intersection was also conducted but, no final table was developed due to the limitation of available tools. The Highway Capacity Software version 7.0 (HCS 7.0) was used in developing service volume tables.

The service volume table development followed the procedure described in HCM 6th edition Chapter 6 Appendix B.

The following provides a brief introduction of the service volume tables' development procedure.

The first step involves identifying all non-volume default values (e.g., number of lanes, intersection spacing between primary intersection and crossovers, peak hour factor, percentage of heavy vehicles, area type, K- and D-factors) and the threshold intersection delay values associated with various level of services. Since alternative intersections are developed to improve the performance of signalized intersections, the Level of Service criteria for alternative intersections follows the criteria for signalized intersection proposed in HCM Chapters 19 & 20 and shown in Table 5.1.

Table 5.1 Level of Service Criteria for Signalized Intersections

Level of Service	*Average Control Delay (sec/veh)*	*General Description (Signalized Intersections)*
A	≤10	Free Flow
B	>10 – 20	Stable Flow (slight delays)
C	>20 – 35	Stable flow (acceptable delays)
D	>35 – 55	Approaching unstable flow (tolerable delay, occasionally wait through more than one signal cycle before proceeding)
E	>55 – 80	Unstable flow (intolerable delay)
F	>80	Forced flow (jammed)

The second step computes the delay for a throughput volume of 100 veh/hour for the daily service volume table as suggested by HCM. If the resulting delay is below the LOS A threshold delay, then step 3 is then followed. If the resulting delay exceeds the LOS A threshold delay, then the resulting delay is compared to the threshold delays for various Level of Services identified in step 1. If the resulting delay exceeds LOS A threshold delay, it will be compared with next LOS delay (e.g., LOS B) until a minimum achievable LOS is found. Then step 3 is the next step taken.

In the third step, the input volume is adjusted to find the maximum volume that achieves LOS A, or the minimum achievable LOS identified in step 2. Test volumes should be a multiple of 100 vehicles per hour for a daily volume table (HCM, 2016). A bisection search algorithm was adopted here to find the threshold volume. Details on the bisection search algorithm can be found in the HCM Chapter 6 Appendix B.

In the fourth step, the test volumes are increased to determine the threshold volume for the next LOS. steps 3 and 4 are repeated until threshold volumes have been found or it has been determined that service volumes cannot be achieved for each level of service. An un-achievable level of service indicates that for the given conditions, the intersection cannot achieve the level of service.

In step 5, the hourly threshold traffic volumes are divided by selected K- and D- factors to get the daily volumes. These volumes are rounded down to a multiple of 100.

The default intersection spacing selected in the first step is changed and step 2 to through step 5 are repeated to develop service volumes for a combination of various intersection spacings.

5.1 Two-way Stop-controlled Median U-turn Intersections

To determine service volumes for the alternatives where the break-down analysis was performed, the selected TWSC-MUTs have two through lanes and one exclusive right-turn lane for each approach at the main intersection and two through lanes and one U-turn lane at the upstream and downstream crossover. The intersection spacing between crossovers and main intersection is 660ft (200m).

When developing the service volume table for TWSC MUT, equal volumes from both the major street and the minor street was assumed. A default peak hour factor (PHF) of 0.95 was assumed, and a free flow speed on the major street of 50 mph was used. The signal timing plan was optimized for each volume combination using the built-in optimization tool in HCS 7.0. The signal optimization objective function was set as minimizing overall delay. Both the cycle length and signal split were optimized. The D-factor and K-factor followed the value recommended in HCM. Tables 5.2 and 5.3 show the recommended values used. Since the MUT was designed to mitigate traffic congestion on arterials that connect urban and rural areas, the closest approximate roadway type for MUT is rural-intercity type. Therefore, in this research, a D value of 0.59 is assumed. The K-factors used varied as the AADT changed. To be noted, the hourly volume is used in the HCS simulation. To locate the proper K-factor value to use in analysis, the AADT range should be converted to a directional design-hour volume range using Equation (5.1).

$$DDHV = AADT \times K \times D \tag{5.1}$$

Where:

DDHV = directional design-hour volume (veh/h),

AADT = annual average daily traffic (veh/day),

K = proportion of AADT occurring in the peak hour (decimal), and

D = proportion of peak-hour traffic in the peak direction (decimal).

Table 5.2 Various K-factor Values by AADT

AADT	*Average K-factor*	*Number of Sites Included in Average K-Factor*		
		Urban	*Recreational*	*Other Rural*
0-2500	0.151	0	6	12
2500-5000	0.136	1	6	8
5000-10000	0.118	2	2	14
10000-20000	0.116	1	2	15
20000-50000	0.107	11	5	10
50000-100000	0.091	14	0	4
100000-200000	0.082	11	0	0
>200000	0.067	2	0	0

Table 5.3 Various D-factor Values by AADT

Freeway Type	*D- Factor*
Rural-intercity	0.59
Rural-recreational and intercity	0.64
Suburban circumferential	0.52
Suburban radial	0.60
Urban radial	0.70
Intra-urban	0.51

Note: K factors are for the 30th highest traffic volume hour of the year.

Source: HCM 2016, Chapter 3.

Using the previously described procedure for developing service volume tables, the Generalized Directional Design-hour Volumes (DDHV) for each LOS for a TWSC MUT was obtained and these volumes are presented in Table 5.4. Using Equation (5.2), DDHVs in Table 5.4 were converted to AADT in Table 5.5. Notably, the DDHV in the following tables are for each approach on the main street, instead of all volumes entering the intersection.

$$AADT=\frac{DDHV}{K\times D} \tag{5.2}$$

As Table 5.4 shows, for 10% left-turns at each intersection approach and with an intersection spacing between the primary and secondary intersection of 660 feet, LOS A conditions is achieved for volumes below 750 vph, LOS B is achieved for volumes below 1290 vph, LOS C below 1510 vph, LOS D below 1630 vph and LOS E below 1740 vph. LOS E conditions describe capacity conditions for the approach. Service volumes for a left-turn percent of 20% and 30% are also provided.

As indicated by the DDHVs and AADTs in Tables 5.4 and 5.5, respectively, the performance of the MUT deteriorates as the left-turn percentage increases. This is expected and consistent with the conclusion from the break-down analysis previously discussed. The results also demonstrate that MUTs work better under low to medium left-turn percentages. The AADT in Table 5.5, which represents the daily volume under which a specific LOS occurs, decreases drastically as the left-turn percentage increases from 20% to 30%.

Table 5.4 Generalized Directional Design-hour Volumes (vph) for TWSC MUT

		Median U-turn Intersections				
Left turn%	*Intersection spacing (ft)*	*A*	*B*	*C*	*D*	*E*
10%	660	750	1290	1510	1630	1740
20%	660	570	1090	1220	1340	1460
30%	660	300	880	990	1090	1190

Table 5.5 Generalized Annual Average Daily Volumes for TWSC MUT

		Median U-turn Intersections				
Left turn%	*Intersection spacing (ft)*	*A*	*B*	*C*	*D*	*E*
10%	660	11000	18800	23900	25800	27600
20%	660	8200	15900	17800	19600	23100
30%	660	3700	12900	14500	15900	17400

5.2 Partial Displaced Left-turn Intersections

Service volume tables were also developed for partial displaced left-turn intersections. Same as the TWSC MUT, the service volume tables for partial DLTs were developed using a geometry of one exclusive left turn lane, two through lanes and one exclusive right turn lane. The partial DLT has two DLT approaches on the main street. Each DLT approach has one displaced left-turn lane. The predetermined intersection spacing between the primary intersection and the crossovers is 350 ft.

The service volume tables were developed assuming equal volumes for all approaches. A PHF of 0.92 was used. The free flow speed on the main street and the minor street were set to 35 mph. The signal timing plan was optimized using the built-in optimization tool in HCS 7.0. Both the cycle length and the signal split are optimized. The optimization objective function is the minimization of the overall delay. The selection of D factor and K factor value follows the procedure discussed in Section 5.1.

The Generalized Directional Design-hour Volumes or service volume for the Partial DLT was obtained following the procedures previously described and run using the HCS 7.0. The service volumes are shown in Table 5.6. Like the TWSC MUT, the hourly service volumes in Table 5.6 were converted to daily service volumes in Table 5.7 using Equation (5.1).

Table 5.6 Generalized Directional Design-hour Volumes for Partial DLT

		Partial Displaced Left-turn Intersections				
Left turn %	*Intersection spacing (ft)*	*A*	*B*	*C*	*D*	*E*
10%	350	220	1110	1520	1800	1890
20%	350	100	1180	1530	1710	1900
30%	350	--	1120	1500	1680	1830

Table 5.7 Generalized Annual Average Daily Volumes for Partial DLT

		Partial Displaced Left-turn Intersections				
Left turn %	*Intersection spacing (ft)*	*A*	*B*	*C*	*D*	*E*
10%	350	2500	16200	24100	28500	29900
20%	350	1100	16900	24200	27100	30100
30%	350	--	16400	23800	26600	29000

Table 5.6 demonstrates that the highest hourly volume for a Partial DLT to achieve LOS A is 220 vph for 10% left-turn percentage and 100 vph for 20% left-turn percentage. For 30% left-turn percentage, LOS A is unachievable, and the minimum achievable LOS is LOS B. It is worth noting that the service volumes are not necessarily increasing as left-turn percentage increase. For LOS B, LOS C, and LOS E, the service volumes increase as the left-turn percentage increases from 10% to 20% and decrease as left-turn percentage increase from 20% to 30%. It is possibly because the bottleneck of partial DLT is moving from primary through movement to the crossover left-turn movement as the left-turn percentage increases. During the moving of bottle neck, the trend of service volumes under various left-turn percentages for a specific LOS is

slightly impacted. However, the difference of service volumes between various left-turn percentages is small. For example, the service volume for LOS B increase 6% when left-turn percentage increases from 10% to 20% and decrease 5% as left-turn percentage increases from 20% to 30%. In general, the service volume of partial DLT increases as volume increases. The left-turn percentage affects the service volumes, but overall performance of partial DLT is not sensitive to the percent of left turns.

Table 5.7 converted the hourly service volume in Table 5.6 to the AADT, and followed the same trend as indicated by Table 5.6.

5.3 Restricted Crossing U-turn Intersections

Attempts were also made to develop service volumes for RCUTs. Unlike DLTs and MUTs' which seeks to reduce delays at the primary intersection, RCUTs are typically used to facilitate mobility on an arterial. Therefore, only unbalanced volumes were tested with the RCUTs having one exclusive left turn lane, two through lanes and one exclusive right turn lane on the major street approach and two right-turn lanes on the minor street. The predetermined intersection spacing between the main intersection and the crossover roadways is 800ft.

Since the RCUTs' primary objective is to relieve traffic congestion on the major street, the volume of the minor street was set as 500 vph while the volume of the major street varied from 500 vph to 2500 vph. Traffic conditions for volume under 500 vph is not considered. A PHF of 0.92 was used and the free flow speed on the main street and the minor street were set to 35 mph. The HCS does not optimize signal timing for

RCUTs, therefore the signal timing was optimized by Synchro 10. A default D factor value of 0.59 was used and the K factor value follows values in Table 5.2.

Figure 5.1 shows the delays of the RCUT under various left-turn percentage conditions. For traffic volume between 500vph and 1200vph, delay decreases as the volume increases for left-turn percentage of 10% and 20%. For 30% left-turn percentage conditions, the vehicle delay generally decreases as volume increases except a sharp increase around 1000vph. Vehicle delay does not necessarily increase as left-turn percentage increases for volume between 500 vph and 1200 vph. After a volume of 1400vph, the vehicle delay increases as volume increases for all three left-turn percentage conditions. Also, higher left-turn percentage results in higher delays under the same volume condition when the traffic volume exceeds 1400 vph.

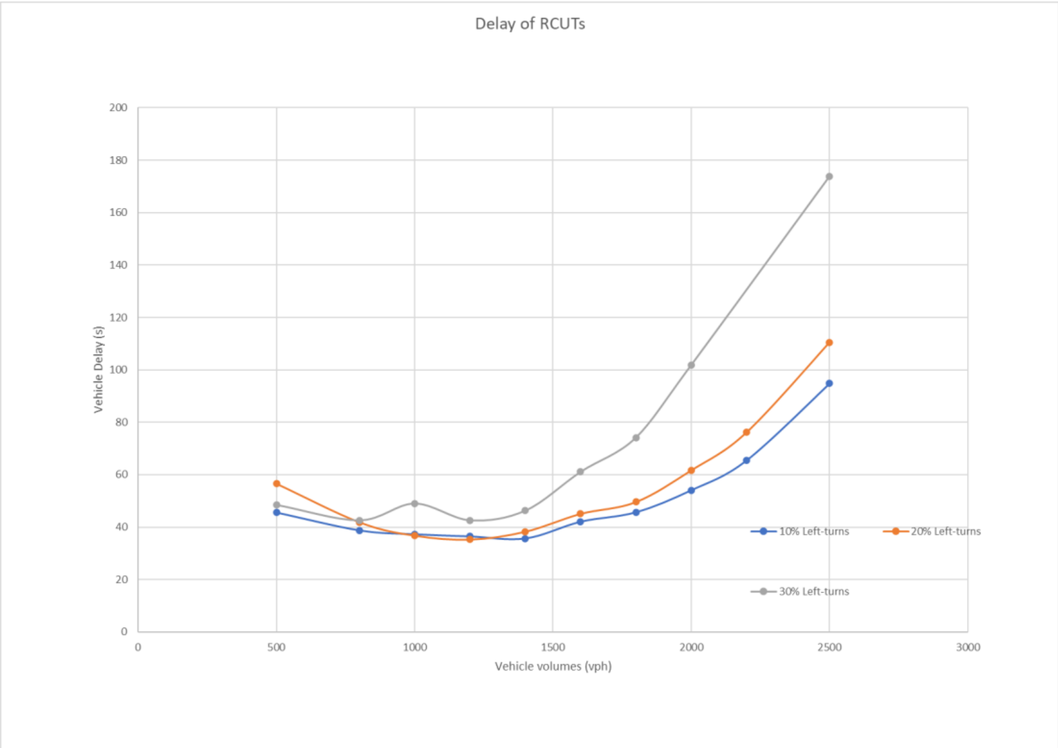


Figure 5.1 Delay of simulated Restricted Crossing U-turn Intersections.

In general, delays of RCUTs decrease as volume increase when volume is below 1200 vph and increase as the volume increase when volume is above 1400 vph. Also, the delays of RCUTs do not necessarily increase as left-turn percentage increases between volume of 500 vph and 1200 vph and increases as left-turn percentage increases when volume is above 1400 vph.

To understand the reasons behind the RCUT's delay trend, delay of each movement is calculated. The movement delay in this chapter includes control delay at each traffic signal and the extra distance travel time. Most of the movement delays increase as volume increases except through and right-turn movement from Main Street. Figures 5.2 and 5.3 present the delays of RCUT's through and right-turn movement under 10% left-turn percentage conditions. The delay trends of 20% left-turn and 30% left-turn percentages follow the same pattern of the delay trend under 10% left-turn percentage.

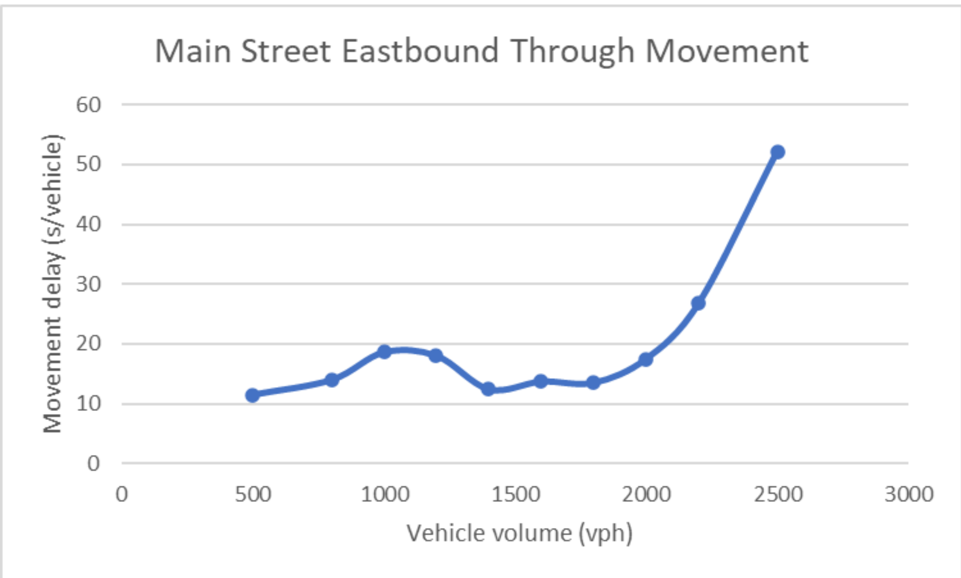


Figure 5.2 The main street eastbound through movement delay under 10% left-turn percentage.

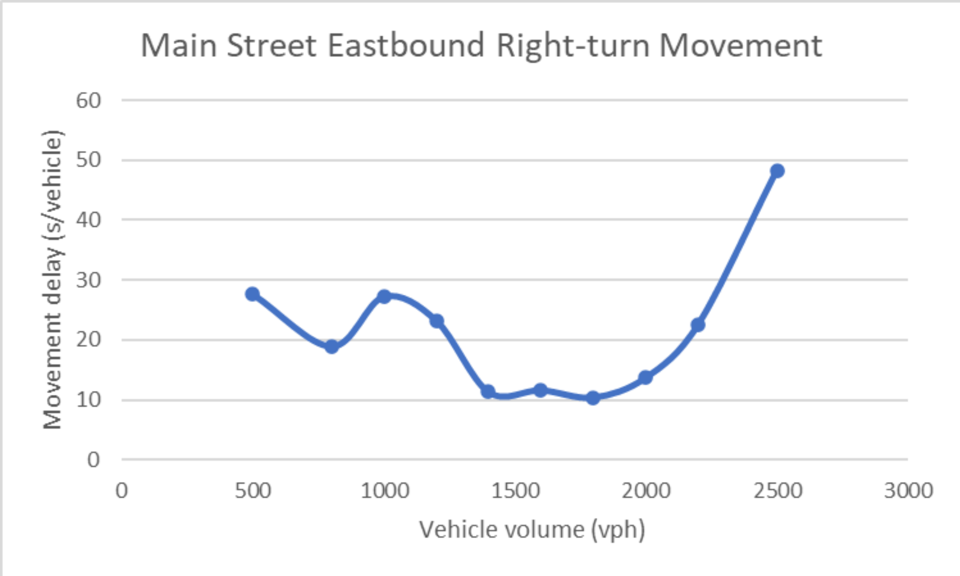


Figure 5.3 The main street eastbound right-turn movement delay under 10% left-turn percentage.

Apparently, the change of Main Street through and right-turn movements contribute to the delay reduction under low to medium traffic volume conditions. The reason behind the changing main street through and right-turn movements could occur due to Synchro 10's optimized signal plan for the entire intersection, the performance of Main Street through and right-turn movements are compromised.

CHAPTER 6

SAFETY ANALYSIS OF ALTERNATIVE INTERSECTIONS

As indicated in the literature review, one significant benefit of alternative intersections is they have demonstrated to reduce crashes at intersections. However, some recent research studies indicated that alternative intersections do not always improve traffic safety under all geometric and volume conditions. Therefore, this chapter investigates the safety performance of MUTs, RCUTs, and DLTs, and develops a methodology to identify UAID designs that provide under varying geometric and volume conditions with improved safety conditions. The safety parameter chosen in identifying appropriate alternative intersections is the Crash Modification Factor (CMF). To account for crash frequency and crash severity, the economic impact of the estimated crashes using the CMFs is determined and included in identifying the most appropriate UAID. This section, therefore, investigated the crash cost associated with various crash types. With CMFs and associated crash cost available, the selection of the most appropriate UAID can be used to understand the safety performance of UAIDs and their economic impact.

Section 6.1 in this chapter introduces detailed information about CMFs and the Section 6.2 summarizes the crash cost by crash types. Section 6.3 proposes a methodology for researchers and other transportation planners to select most appropriate UAID in terms of safety. The UAIDs with negative safety impacts will be excluded from future consideration.

6.1 Crash Modification Factors

According to the Highway Safety Manual, a crash modification factor is defined as a multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site. To better understand the concept of CMFs and to better understand CMFs developed for each type of UAID, it is necessary to review prevailing methodologies of intersection safety analysis and understand the advantages and disadvantages of each approach. The Subsection 6.1.1 summarizes CMFs development methodologies that have been adopted to date. The Subsection 6.1.2 identifies CMF values developed for alternative intersections under different conditions.

6.1.1 CMF Development Methodologies

Frank Gross introduced six methodologies in the *Guide to Developing Quality Crash Modification Factors* (Frank Gross, 2010). These methodologies include: (1) before and after crashes with a comparison group analysis; (2) empirical Bayes before-after analysis; (3) full Bayes analysis; (4) cross-sectional analysis; (5) case-control analysis; and (6) Cohort analysis. At present, three of them have been used to develop CMFs for alternative intersections: before and after crashes with a comparison group analysis; empirical Bayes before-after analysis; and cross-sectional analysis. Table 6.1 summarizes the methodologies used in the development of CMFs for alternative intersections.

Table 6.1 CMF Development Methodologies for UAIDs

<table>
<tr><th>Methodologies</th><th>Details</th><th>Data</th><th>Limitations</th></tr>
<tr><td>Before-after crashes with a comparison group analysis</td><td>Treatment group and comparison group are selected.
Sample odds ratio is calculated for each selected comparison group. The closer the ratio is to 1.0, the better comparable group is chosen.
CMF is calculated based on the value of the observed number of crashes in the after period for the treatment group and the expected number of crashes in the after period for the treatment group.</td><td>Traffic volume.
Observed number of crashes in the before period for the treatment group.
Observed number of crashes in the after period for the treatment group.
Observed number of crashes in the before period in the comparison group.
Observed number of crashes in the after period in the comparison group.</td><td rowspan="2">Traffic volume changes due to general trends or to the alternative intersection design itself.

Changes in reported crash experience due to changes in crash reporting practice, weather, driver behavior, effects of safety programs, etc.

Improper selected comparison group.</td></tr>
<tr><td>Empirical bayes before-after analysis</td><td>Regression-to-mean is considered.
A SPF is developed from the comparison group to predict the number of crashes in the treatment group.
Each SPF is assigned a weight. Together with the predicted number of crashes estimated by the SPF in the before period and the observed number of crashes in the before period for the treatment group for the treatment group, the number of crashes in the before period for the treatment group is estimated.
CMF is calculated based on the value of the observed number of crashes in the after period for the treatment group and the expected number of crashes in the after period for the treatment group.</td><td>Traffic volume.
Observed number of crashes in the before period for the treatment group.
Observed number of crashes in the after period for the treatment group.</td></tr>
</table>

Table 6.1 CMF Development Methodologies for UAIDs (Continued)

Methodologies	Details	Data	Limitations
Cross-sectional analysis	A SPF is developed to quantify the relationship between number of crashes and all variables that affect safety. Very useful when there is not enough data to perform a before-after analysis. A CMF can be inferred from the difference in mean predicted number of crashes from SPF when the value of a variable is increased by one unit.	Traffic volume. All the possible factors that impact intersection safety (i.e., intersection type, lane width, driving densities, intersection grade, etc.) Observed number of crashes in the before period for the target intersection. Observed number of crashes in the after period for the target intersection.	The crash rate change may be caused by other factors than those that have been identified in the SPF model. It is very difficult to properly identify and measure all the safety influential factors of alternative intersections. It is very likely that the derived CMFs are inaccurate if a function is improperly selected, some influential variables omitted, or the selected variables are correlated

Table 6.1 briefly introduces the information on the CMF development methodologies, required data and limitations. When crash data of the treatment intersection and comparison intersections are sufficient, the before-after analysis is conducted. When the available data is limited, a cross-sectional analysis is a better practice. The better quality of the data, the more reliable the developed CMFs.

6.1.2 CMF Developed for UAIDs

At present, four papers have studied CMFs for DLTs with one of them (Abdelrahman et al., 2020) included in the FHWA funded Crash Modification Factor Clearinghouse (CMF Clearinghouse) website (FHWA). Three papers calculated the CMFs for MUT and the research of Al-Omari (Al-Omari et al., 2020) was accepted by the CMF Clearinghouse. The CMF Clearinghouse website also summarized seven studies that developed CMFs for RCUTs. The selection of the most appropriate CMFs can be subjective and dependent on the experience of those selecting the CMF. Kentucky Transportation Center (KTC) created an Excel spreadsheet to assist in screening the most appropriate CMFs in the state of Kentucky (KTC, 2020). This Excel spreadsheet imported all the CMFs collected by the CMF Clearinghouse website and labeled each CMF with various filters (i.e., crash type, severity, roadway type, countermeasure group, etc.). By selecting the most appropriate filter, this Excel spreadsheet will present all the qualifying CMFs and rank them in terms of data quality. The CMF selection methodologies in other agencies like Washington DOT (WSDOT, 2015), Wisconsin DOT (WDOT, 2005), and Pennsylvania DOT (Smith, 2016) shares a similar concept for screening CMFs. Therefore, the CMF selection in this research will follow the same rule.

The available CMFs for alternative intersections are limited. The CMFs for DLTs and MUTs selected in this alternative intersection safety analysis will be the CMFs accepted by CMF Clearinghouse. Only one CMF is recognized by the CMF Clearinghouse for both DLTs and MUTs. Seven papers have been adopted by the CMF Clearinghouse for RCUTs and three of them calculated the CMF for converting a conventional intersection to RCUT. The CMFs from Sun's research (Q. Sun, 2019) have the highest quality score, therefore, will be accepted in this safety analysis. To be noted, all the CMFs presented below are the CMFs for converting a conventional intersection to alternative intersections. As the data and research are limited, the CMFs presented below should be used with caution and be replaced whenever better CMFs are available. Table 6.2, Table 6.3, and Table 6.4 show the selected CMFs for DLTs, MUTs, and RCUTs, respectively.

Table 6.2 Selected CMFs for Converting Intersection to DLT

Crash Type	*CMF*
Total Crashes	1.11
Fatal-and-Injury	1.22
PDO	1.07
Single vehicle	1.52
Non-motorized	0.612
Angle	1.244
Rear-end	0.946
Head on	0.713
Sideswipe same direction	1.11

Table 6.3 Selected CMFs for Convert Intersection to MUT

Crash Type	*MUT Types*			
	Type A	Type B	Type C	Non-specified
Total	0.6330***	0.7175***	1.132***	0.844
Fatal-and-injury	0.7732***	0.7029***		
Injury	0.7548***	0.6296***		0.702
PDO	0.5984***	1.4447**		0.912
Single vehicle	1.3800**	0.6108***		
Angle	0.6835***	0.3342***		
Head-on	0.2559***	0.1788***		
Head-on left-turn	0.1719***	0.5158***		
Rear-end	0.5258***	0.3940***		
Rear-end left-turn	0.3942***	1.2337		
Rear-end right-turn	0.9361	1.1316		
SD sideswipe	0.9155	0.1269***		
OD sideswipe	0.2167***	1.9576***		
Non-motorized	2.2432***	1.3877		

Noted: detailed information of type A MUT, type B MUT and type C MUT are included in Chapter 2 Literature Review.

Table 6.4 Selected CMFs for Convert Intersection to RCUT

Crash Type	*CMF*
Total	0.7632***
Fatal-and-injury	0.5669***
Injury	0.5726***
PDO	0.8414*
Single vehicle	1.3079
Angle	0.5854***
Head-on	0.0667***
Rear-end	0.7511**
SD sideswipe	0.9291
OD sideswipe	0.3299***

6.2 Crash Cost Analysis

To capture the impact of crash frequency changes caused by converting a conventional intersection to an alternative intersection, crashes were monetized and converted to crash costs. In most studies, crash costs utilize comprehensive crash costs which include comprises tangible crash costs and intangible crash costs. Tangible crash costs are the economic costs related to the crash, including goods and services cost related to the crash response, property damage cost, and medical costs. The intangible crash costs are meant to monetize pain and suffering caused by crash. A concept referred to as Quality-Adjusted Life Years (QALY) was used to quantify the lost quality of life due to death and injury related to a crash. In general, the QALYs cost are estimated by duration and severity of the health problems. Crash Cost for Highway Safety Analysis (Tim Harmon, 2018) introduced the detailed procedures to estimate the number of QALY costs. Combining tangible crash costs (i.e., economic costs) and intangible crash costs (i.e., QALY costs) , the comprehensive crash costs summarize all the impacts that related to crashes.

In most cases, crash costs are estimated by crash severity and in some rare cases also estimated by crash types. This is because there is a positive correlation between crash cost and crash severity and an even higher correlation between crash cost and crash type. Most crash summaries are reported with crash severity and few of them are reported with crash types. Two injury scales are used to identify the crash severity: KABCO scale and the Abbreviated Injury Scale (AIS). In the KABCO scale, K stands for fatal injury, A stands for suspected serious injury, B stands for suspected minor injury, C is possible injury, and O means no apparent injury and in some other circumstances are also mentioned as Property Damage Only injury (PDO). The KABCO scales are often used in many police crash reports. The Abbreviated Injury Scale are more commonly used by hospitals and motor vehicle crash investigators to identify the crash severities of single crashes. In the AIS scale, a number from 0 to 6 is assigned to a crash with 0 means no injury and 6 means maximum injury. In some cases, the number 9 is also used to classify crashes when the injury level is unknown or difficult to be classified. The death probability of no injury is 0% and the death probability of maximum injury is 100%. As mentioned before, the AIS scale is a tool to measure injury levels for single injuries. As many crashes involve more than one injured person, to describe the crash better, the concept of Maximum Abbreviated Injury Scale (MAIS) is many times used. MAIS is the score of the most severe injury suffered by an injured person in a crash. In the crash analysis, MAIS is more commonly used than AIS.

Crash Cost for Highway Safety Analysis (Tim Harmon, 2018) reviewed the previous studies that estimated the comprehensive crash unit cost by crash severities and proposed a methodology to estimate the crash unit costs that can be used as default value in FHWA's *Safety Benefit-Cost Analysis Tool*. Table 6.5 shows the estimated default comprehensive crash unit cost in KABCO scale.

Table 6.5 Comprehensive Crash Unit Cost in KABCO Scale (2016 dollars)

Severity	*Comprehensive Crash Unit Cost*
K	$11,295,400
A	$655,000
B	$198,500
C	$125,600
O	$11,900

Source: (Tim Harmon, 2018)

A few resources have been found to estimate crash costs by crash types (Blincoe et al., 2015; Council et al., 2005; Part, 2010). Possible modifications are required while applying these research findings. One major obstacle in estimating crash unit costs by crash types is the lack of data. In most police reports, crashes are categorized by crash severity and few of them are categorized by crash types. This limitation in the original data makes the calculation of crash unit costs by crash types much more difficult than estimating crash unit costs by crash severity. Another limitation in estimating crash unit cost by crash type is the correlation between crash unit cost and crash types are lower than the correlation between crash unit cost and crash severity. For crashes with the same crash type, the crash severity may be completely different due to various crash speed and different safety precautionary measures and the resulting crash unit costs could be widely divergent.

However difficult, the exploring of crash costs by crash types can be important. In the safety analysis of alternative intersections, the change of crash frequencies by crash severities due to alternative intersections is uncertain. The current available alternative intersection CMFs explained the difference of crash frequencies by crash types. To fully understand the safety impact of alternative intersections, exploring the crash unit costs by crash types can be equally important as identifying alternative intersection CMFs. Table 6.6 demonstrates the latest available comprehensive crash unit costs by crash costs. If updated or more recent costs are available, the numbers in Table 6.6 should be substituted by the updated or recent crash unit costs.

Table 6.6 Comprehensive Crash Unit Costs for Selected Crash Types

Crash Type	*Comprehensive Crash Unit Costs (2010 dollars)*
Roadway Departure Crashes	\$22000
Single-Vehicle Crashes	\$25400
PDO Vehicles	\$5300
Total	\$61600

Note: total number of crashes in Blincoe's research includes both reported crashes and unreported crashes.

Source: (Blincoe et al., 2015)

To avoid the possible inconsistency in identifying crash costs by crash types, the average comprehensive crash unit costs for total crashes of \$61600 will be used in this research and should be converted to the 2022 dollars \$81300 (rounded to the hundred).

6.3 Safety Impact of Alternative Intersections

With CMFs and comprehensive crash unit costs of alternative intersections available, it is feasible to monetize the safety impacts of alternative intersections. Equations (6.1) and (6.2) show the details of estimating the crash costs. Positive crash difference in Equation (6.1) describes the number of crashes reduced after conversion to the UAID and a negative crash difference indicates the number of crashes increased. Therefore, the positive annual safety cost difference describes the annual crash reduction, and a negative annual safety cost difference indicates the annual crash cost increase. The default value of f_{local} in Equation (6.1) is 1.0 if not otherwise specified.

$$Crash\ difference = Avg.Crashes\ before\ change\ \times (1 - CMF \times f_{local}) \tag{6.1}$$

$$Annual\ safety\ cost\ difference = crash\ difference \times comprehensive\ crash\ unit\ costs \tag{6.2}$$

Where f_{local} = localization factor for CMFs.

CHAPTER 7

CASE STUDY

7.1 Background Introduction

The methodology for selecting an alternative intersection was applied in a case study. The case study was performed based on an existing intersection. The criteria used in the selection of locations to apply the case study included the following:

- High percentage of left-turn movements on both major street and minor street.
- Right-of-way availability.
- Crash data and volume data availability.

To identify intersections that met the criteria, a review was made of upcoming state intersection improvement projects and through reviews of traffic impact assessment reports within the State. An intersection located at County Road 537 and Pine Drive in Jackson Township NJ met all the requirements. County Road 537 is the major street and Pine Drive is the minor street. However, the low volume and inadequate number of lanes on the minor street limited the selection of a broad range of potential UAIDs that could be utilized. None of the known UAIDs can be implemented into a single-lane roadway and result in significant improvement in the intersection performance when compared to a conventional intersection. To demonstrate the ability of the selection model to choose between a broad number of potential UAIDs, some elements of the selected intersection were changed for analysis purposes in the case study. The current minor street was expanded from a two-way single lane roadway to a two-way two-lane roadway. The volume from the minor street was also doubled of the original traffic volume. All the other aspects of this intersection remained the same as field conditions to insure the feasibility of applying the selection methodology to real world conditions. The following section introduces the characteristics of the selected intersection.

The selected case study intersection is located at County Road 537 and Pine Drive in Jackson Township, NJ. East of this intersection is Six Flags Great Adventure and Hurricane Harbor theme parks. Interstate 195 is on the west side providing access to CR 537 by a full interchange. Some fast-food restaurants such as McDonald’s and convenience stores like Wawa, are located to the north of the intersection. A large youth sport and entertainment center, Adventure Crossing, is under construction on the south of this intersection. Also, as planned by the Adventure Crossing developer, warehousing and an indoor recreational area will share space on the property.

Currently, this intersection is under reconfiguration. Once the construction is completed, the current three-leg signalized intersection will become a four-leg signalized intersection with high demand of turning movements from and to County Road 537. Figure 7.1 shows the pre-development intersection geometry and Figure 7.2 shows the post-development intersection geometry. To conduct the case study, the researchers slightly modified the intersection geometry, and the details of this modification are shown in Figure 7.3. The following analysis is based on the intersection geometry shown in Figure 7.3.

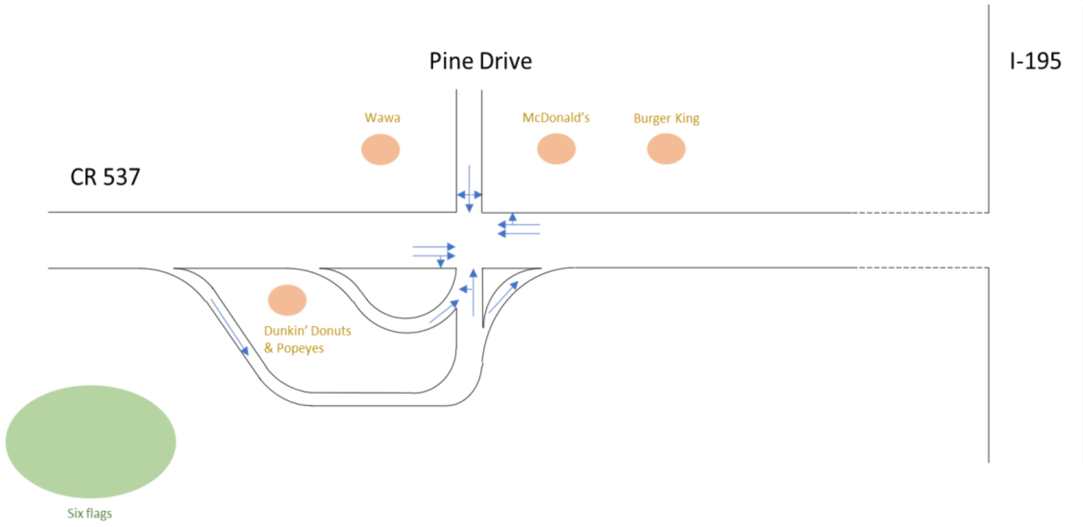


Figure 7.1 The pre-development intersection geometry.

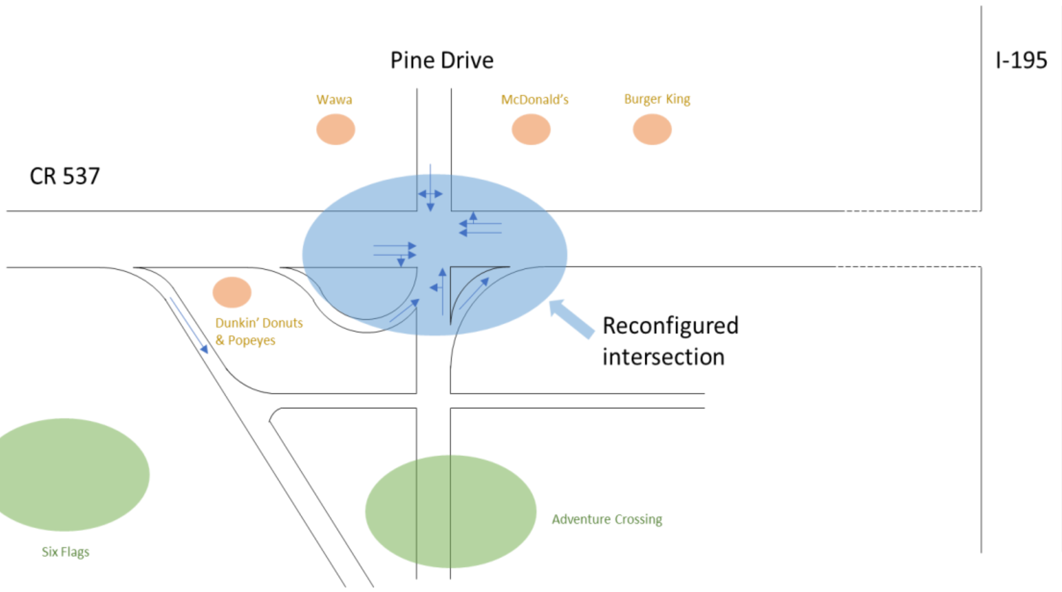


Figure 7.2 The post-development intersection geometry.

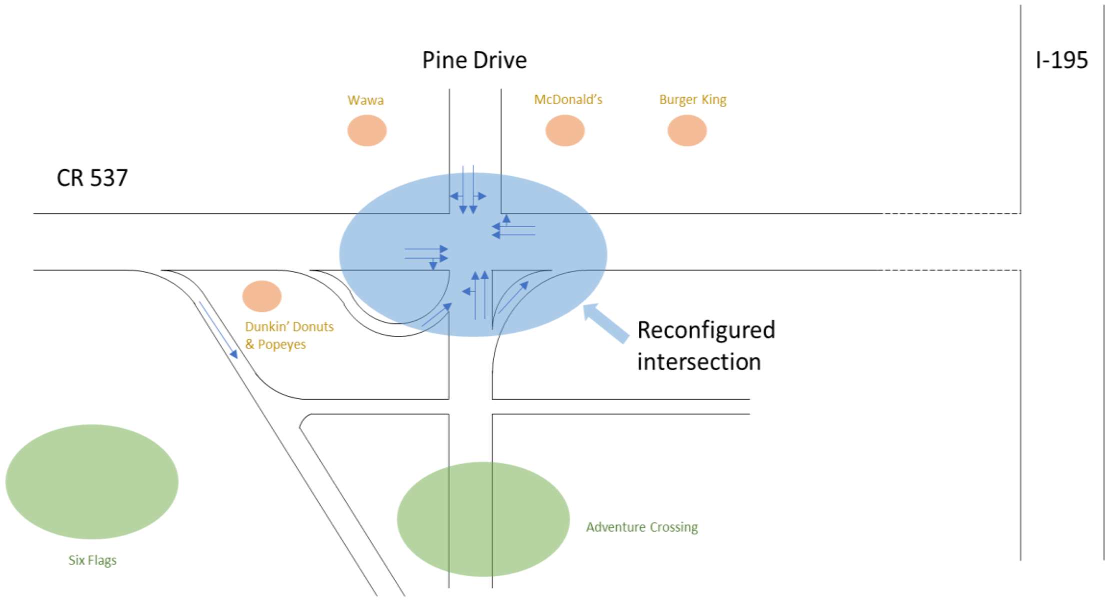


Figure 7.3 The post-development intersection geometry after modification.

McDonough & Rea Associates estimated the traffic volume after development in their Traffic Impact Analysis Report (McDonough & Rea Associates, 2020), and Figure 7.4 shows the detailed information. The latest available crash data for Monmouth County is for the 2019 year. For the cross of CR 537 and Pine Drive, a total of 6 crashes happened in 2019 and five of the crashes are property damage only crashes and 1 of the crashes involved one person injury. All the crashes are same-direction crashes. The designed service life of proposed alternative intersection is 30 years. The average occupation rate is 1.07 persons per vehicle.

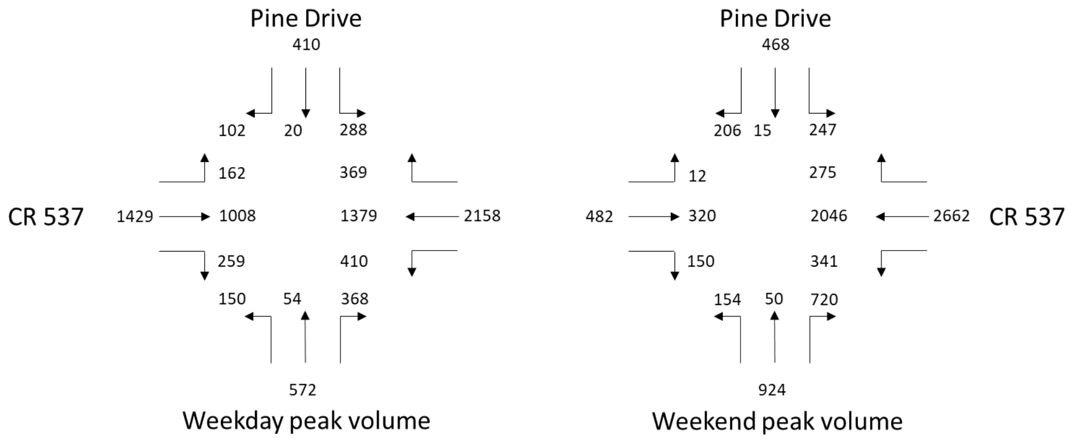


Figure 7.4 Intersection's weekday peak volume and weekend peak volume.

The reconfigured intersection is designed as a conventional signalized intersection. According to the given information, the LOS of this conventional intersection is LOS F with an intersection delay of 98.9 s/veh. As there is ample space for road-widening and a high demand of left-turn movements on both CR 537 and Pine Drive, this intersection is a perfect candidate for a UAID. Therefore, this intersection is chosen to demonstrate the alternative intersection selection methodology described in Chapter 3. The methodology identifies the most appropriate UAID design that can out-perform the proposed conventional design in terms of both mobility and safety.

Due to the limitations of available data, the potential candidate UAIDs to be considered for this case study is limited to DLT, MUT RUCT and their variants: full DLT, partial DLT on CR 537, DLT approach on EB CR 537, DLT approach on WB 537, partial DLT on Pine Drive, DLT approach on NB Pine Drive, DLT approach on SB Pine Drive, full MUT, partial MUT on CR 537, partial MUT on Pine Drive, RCUT on CR 537, and RCUT on Pine Drive.

7.2 Target and Constraints Collection

As introduced in the Chapter 3 methodology, the selection procedure includes three stages:

Stage 1: the initial stage – define project objectives and constraints.

Stage 2: the filter stage – screen out inapposite UAIDs candidates.

Stage 3: the analysis stage – detailed mobility and safety analysis.

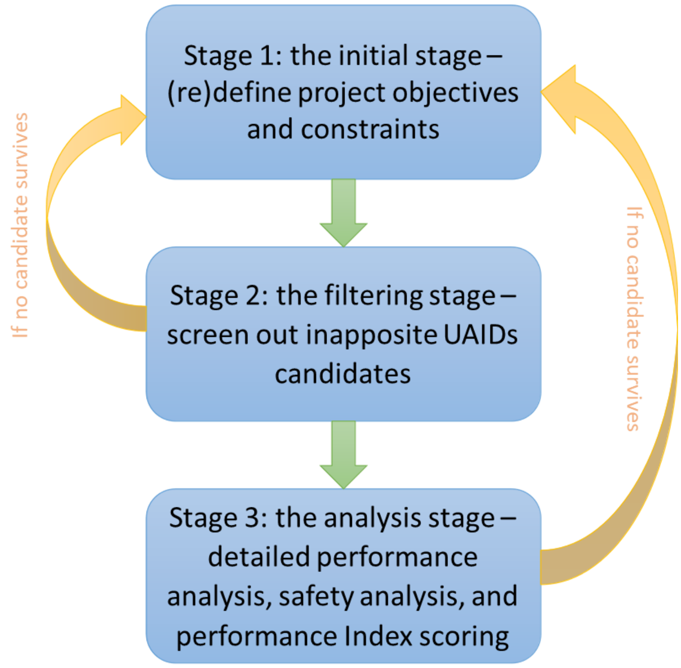


Figure 7.5 The structure of UAIDs selection model.

To start the selection process, the objective(s) of intersection improvement should be determined. Due to the impact of COVID-19 and other physical difficulties to obtain data from local transportation agency officers, the project improvement objectives, and objective weights adopted in the case study were hypothetical. The hypothetical project objectives and objective weights do not affect the procedure of this selection methodology. The assumptions about project objectives and objective weights were made based on the Traffic Impact Analysis and Adventure Crossing (McDonough & Rea Associates, 2020)

Table 7.1 Project Objective Summary of Case Study

Project Objectives	*Weight*
Improve intersection safety	0.4
Reduce delays	0.5
Financially feasible (cost/benefit <1)	0.1

Table 7.1 describes the determined intersection improvement objectives and weight assigned to each objective. In this case, the intersection improvement objectives are improving intersection safety, reducing intersection delays, and maintain project financial feasibility. The weight assigned to the objective of improving intersection safety is 0.4, the weight assigned to reducing intersection delays is 0.5 and the weight assigned to maintain project financial feasibility is 0.1.

Table 7.2 Summary of Case Study Constraints

Project constraints	Details
• *ROW constraints*	50ft beyond the boundary of CR 537 and Pine Drive
• *Available budget*	$10M
• *Intersection accessibility*	Medium
• *Friendly to pedestrians and bicycles*	N/A
• *Friendly to public transportations*	N/A
• *Current vehicle volumes, especially for the left-turn movement (Peak and Non-peak)*	See Figure 7.4
• *Current intersection geometries*	See Figure 7.3
• *Desired intersection LOS*	C and above
• *All the other concerns (please specify)*	

Table 7.2 describes the constraints of case study. In this table, the information of ROW constraint, budget constraint, intersection accessibility constraint, volume conditions, intersection geometries and desired intersection LOS are collected. It is assumed that the 50ft ROW beyond the boundary of CR537 and Pine Drive is available to be used for the intersection, which allows possible road-widening on both roadways. Detailed intersection volume is presented in the Figures 7.4 and Figure 7.3 introduces the intersection geometry for the case study.

7.3 Preliminary Screening

After identifying the project improvement objectives and project constraints, all the candidates will be preliminarily checked to rule out candidate UAIDs that do not meet the criteria and therefore are inappropriate choices. The preliminary check for selection includes: a ROW check, mobility check, and a budget check. The ROW check will exclude candidates that cannot fit into the available ROW. The mobility check will remove candidate UAIDs that have lower operational performance than the existing intersection. However, although the candidate design may perform well holistically, a design that constantly fails for one or more movements will not be an attractive choice. Therefore, if an intersection design experiences LOS E or higher for one or more movements, the UAID will be removed from the candidate list. The failed movement is identified as the bottleneck of the intersection. A design breakdown test will be applied to assist planners to perform a bottleneck check before performing a detailed mobility analysis. In addition, all the feasible UAID candidates must be determined to be within budget constraints.

7.3.1 ROW check

In general, the footprint of MUTs and RCUTs are similar and DLTs have a smaller footprint compared to the previous two designs. As shown in Figure 7.3, CR 537 is a two-lane highway with a 4-foot shoulder on each side of the roadway. To accommodate a MUT or RCUT on CR 537 requires additional right-of-way on both sides of the highway. Table 7.2 stated that the CR537 owns ROW 50 ft. beyond its current boundary. The additional ROW is enough to accommodate both MUTs and RCUTs on CR537. A DLT design does not require as much right-of-way as a MUT or a RCUT design. Therefore, the additional ROW is enough for a DLT on CR537.

Currently Pine Drive is a two-way two-lane roadway with a center line. There is ample space between Pine Drive and the nearby commercial land to accommodate a MUT, RCUT, and DLT.

Due to the sufficient ROW of the selected intersection, candidate UAIDs were not removed because of a ROW constraint.

7.3.2 Mobility Check

As estimated by the Traffic Impact Analysis Report (McDonough & Rea Associates, 2020), peak hours volumes of this intersection are different during weekdays and weekends. The estimated traffic volume is based on the post-development intersection design. The objective of this case study is to redesign the intersection using the most suitable UAID. Figure 7.6 shows the estimated peak volume during weekdays and weekends with turning movements' percentages.

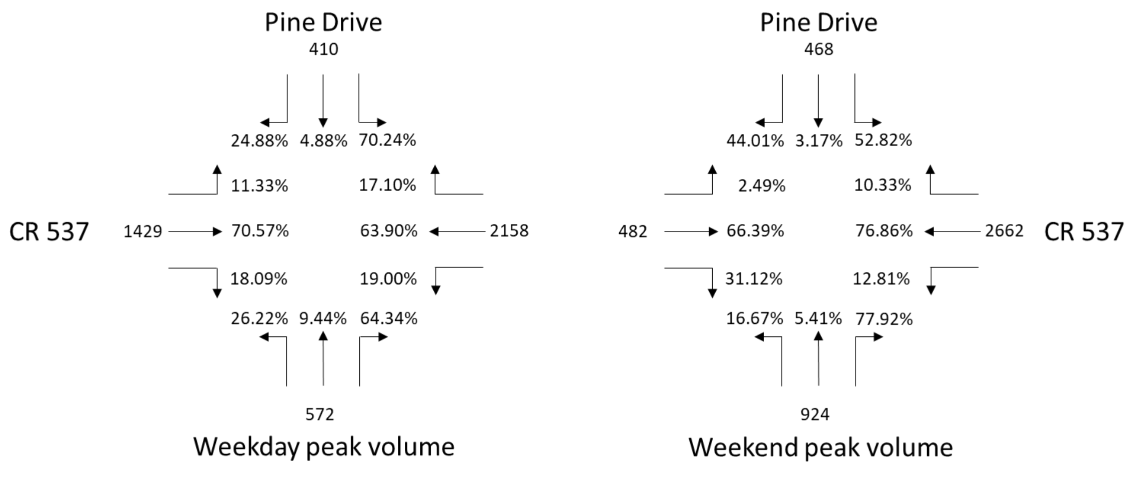


Figure 7.6 Intersection's weekday peak volume and weekend peak volume with turning movements percentages.

As previously mentioned, the traffic pattern differed between the weekdays and weekends. For the weekday traffic, there is an equal distribution of vehicles between the east and westbound movements on CR 537, with a slightly higher volume in the west bound direction. The traffic volume on Pine Drive is much lower than the volume on CR 537. During weekends, vehicles travelling east bound were reduced to 482 vph and those on the west bound increased to 2662 vph as many people are travelling to the Six Flags Theme Park in the west of intersection. The north bound Pine Drive traffic volume increases to 924 vph and the south bound Pine Drive volume remains almost the same. In this case study, the weekday peak volume will be adopted in alternative intersection designs selection. (I am not sure if the previous volume information is helpful. Perhaps restating why there was a need to make small adjustments to the volume data obtained from the traffic analysis report. The details are a bit confusing to me.

To perform a preliminary screening of candidate UAIDs, a set of break-down test tables are adopted. Break-down test tables are the tools designed for this methodology to identify whether a UAID candidate will fail under given volume conditions. The failure of an intersection is defined as experiencing LOS E or worse at one or more movements. The movement which reaches LOS E first as traffic volume increases is referred to as the intersection bottleneck. Using the breakdown tables developed in Chapter 4, for DLT, MUT and RCUT, UAIDs with movements that fail, or having bottlenecks, will be removed.

In the development of the breakdown tables, a total of three traffic volume conditions and three left-turn volume percentages were tested. For DLT and MUT, the lowest traffic volume condition was 1000 vph, the middle traffic volume condition was 1500vph and the highest traffic volume condition was 200vph. For RCUT, the traffic volume for minor streets is fixed to 500 vph; the lowest traffic volume for the major street is 1000 vph, the middle traffic volume is 1500 vph and the highest traffic volume is 2000 vph. Also, the three left-turn percentages tested included 10%, 20% and 30%. Therefore, the breakdown-test included nine conditions for each design. The real-world intersection volume conditions are unlikely to be the same as those used in the development of the breakdown-test tables. To make the best estimate as possible for DLT and MUT designs, any volume below 1000vph is viewed as 1000vph when using the breakdown-test tables. Volumes between 1000 vph and 1500 vph are treated as 1500 vph, and volumes between 1500 vph and 2000 vph are treated as 2000 vph. Any volume above 2000 vph is treated as 2000 vph. Follow the same approach, any left-turn percentage below 10% is treated as 10%, left-turn percentages between 10% and 20% are treated as 20%, left-turn percentages between 20% and 30% are treated as 30%, and those above 30% are treated as 30%. A similar approach is used for a RCUT design, with the exception that the traffic volume below 500 vph is treated as 500 vph for the minor street of RCUT. The through movements and right movements are not the critical movements for UAIDs and will not dominate the control delays of an UAID, therefore the traffic volume percentages of through and right movements are not considered when performing a break-down test. With this consideration, traffic conditions in Figure 7.6 are simplified to those in Figure 7.7.

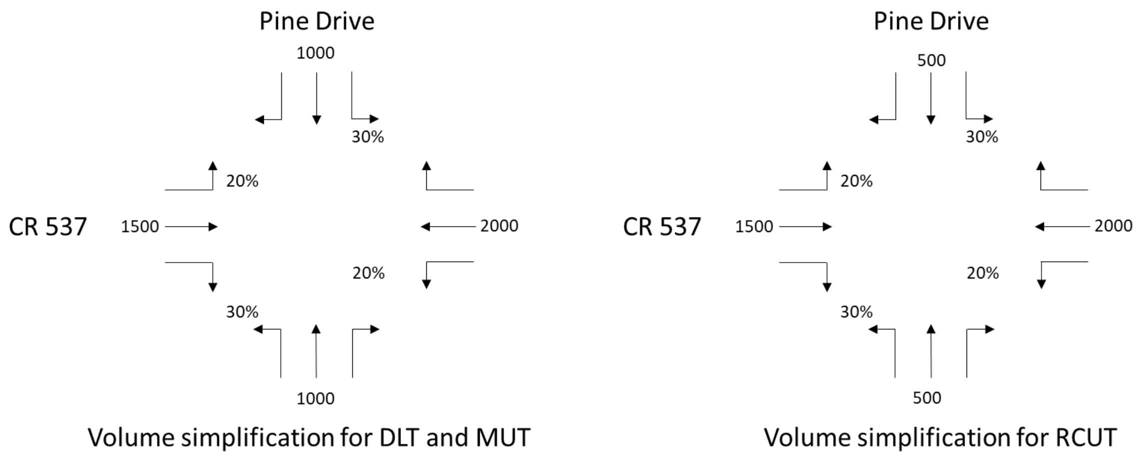


Figure 7.7 Intersection's weekday peak volume simplification for preliminary screening.

After simplifying the volume conditions, transportation planners can directly use the break-down test table using the volume and left-turn percentage for the intersection being evaluated. By referring to the breakdown table, any designs exceeding LOS E for at least one movement will be excluded from consideration. For the case study intersection under the DLT design, the east bound approach volume and left-turn percentage falls under the 1500 vph and 20% left-turn vehicle column and the highest LOS for the DLT design is LOS D. For the west bound approach, the volume and left-turn percentage falls under the 2000 vph and 20% left-turn vehicle column and the highest LOS is LOS F. Both the north and south bound approaches of this intersection fall under the column for 1000 vph and 30% left-turn vehicles, and the highest LOS is LOS F. Therefore, the only possible option for DLT is to construct a DLT approach on the east bound intersection approach. The same procedure is followed to perform the break down test for the MUT and RCUT UAID alternatives. The possible approach for the intersection is MUT on Pine Drive and RCUT on CR 537.

The break-down analysis demonstrated the possible UAID candidates for this intersection is a DLT on the east bound approach and a MUT on Pine Drive and RCUT on the CR 537.

7.3.3 Budget Check

The estimated investment into the construction of Adventure Crossing and its nearby infrastructures is a total of $800M. Although the actual budget assigned to the intersection reconfiguration is not available, it is reasonable to assume that a construction cost within $10M for the intersection reconfiguration is allowed.

Abdelrahman (Abdelrahman et al., 2020) investigated the construction cost of 3 DLT intersections. The lowest bid price was $4.4 million dollars in the year of 2006 and the highest construction cost was $7.5 million dollars in the year of 2007. The cost to retrofit a conventional intersection to a MUT for the Indiana Department of Transportation, ranged from a few hundred thousand dollars to more than $1 million (INDOT). Hummer (Hummer & Rao, 2017) estimated the construction cost for RCUTs in Alabama, Ohio, and Texas, and the average construction cost was $3.75 million in 2014 dollars.

Therefore, a reconfiguration of the case study intersection from a conventional intersection to a DLT, MUT or and RCUT is within budget and no UAID candidates is excluded due to budget constraints. The possible UAID candidates for this intersection after budget check are a DLT approach on the east bound intersection approach and MUT on Pine Drive and RCUT on the CR 537.

7.4 Detailed Mobility Analysis and Safety Analysis

In this stage, a detailed mobility analysis and safety analysis will be performed to investigate the remaining candidates.

7.4.1 Mobility Analysis

As demonstrated in Chapter 5, service volume tables were developed to perform a more detailed mobility analysis for MUT and RCUT UAIDs. An intersection delay figure was developed for RCUT UAID. The service volume tables provide the maximum volume for which a LOS can be reached under three left-turn percentages. The RCUT intersection delay figure provided the estimated intersection delay under three left-turn percentage conditions. For DLTs and MUTs, the three levels of left-turn percentages used were 10%, 20%, and 30%. In the RCUT intersection delay figure, the traffic volume ranges for the main street are 500 vph to 2500 vph, and 500 vph for minor streets; three levels of left-turn percentages are 10%, 20%, and 30% for main street, 10% left-turn for minor streets.

Real-world conditions are much more complicated than those from which the service volume tables, and intersection delay figure are based. Therefore, a simplification is necessary before implementing the mobility analysis. Left-turn percentages less than 10% will be treated as 10%. Left-turn percentages between 10% and 30% will be rounded to the nearest boundary value (e.g., 10% or 30%). For left-turn percentages higher than 30%, a left-turn percentage of 30% will be used. The traffic volumes remain the same as original data but were simplified as previously described in Subsection 7.3.2. By checking the volumes and corresponding left-turn percentages, transportation planners are easy to identify the Level of Service for this UAID candidate. Figure 7.8 shows the simplified traffic volume conditions for mobility analysis.

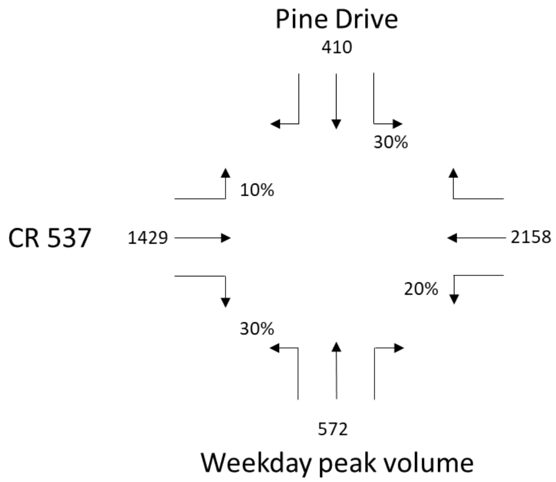


Figure 7.8 Intersection's weekday peak volume simplification for mobility analysis.

For this intersection improvement project, the target LOS is LOS C. From the previous analysis, the remaining UAID designs are DLT approach on the east bound intersection approach and MUT on Pine Drive and RCUT on the CR 537. According to the DLT service volume table, a DLT with 10% left-turn traffic volume has a service volume of 1520 vph for LOS C. The peak volume of east bound CR 537 is 1429 vph and is below the 1520 vph threshold. Therefore, a DLT approach on the east bound intersection approach passes the mobility analysis. The service volume of MUT with 30% left-turn percentage for LOS C is 990 vph. The peak volume of Pine Drive is 572 vph for the northbound and 410 vph southbound, which is below the 990 vph service volume. Therefore, MUT on Pine Drive passes the mobility analysis. Referring to the RCUT delay chart, the delay of a RCUT with 10% left-turn and 1429 vph volume will be approximately 38 second/vehicle, and the delay of a RCUT with 20% left-turn and 2158 vph volume will be approximately 75 second/sec, which is above the 35 seconds for LOS C threshold. Therefore, a RCUT on CR537 will be removed from future consideration.

The remaining UAID designs are DLT approaches on the eastbound intersection approach and MUT on Pine Drive.

7.4.2 Safety analysis

According to the latest available crash report, a total of 6 crashes happened at the crossing of CR 537 and Pine Drive in 2019. As indicated in Chapter 6, the current best available CMF for a partial DLT of total crashes is 1.11, and the best available CMF for type A MUT of total crashes is 0.6330. Blincoe (Blincoe et al., 2015) estimated the national comprehensive crash unit cost for total crashes as \$ 61600 in 2009 dollars. Converting to 2022 dollars, the national comprehensive crash unit cost for total crashes is \$81300. Combing the available CMFs and comprehensive crash unit costs, the monetized safety impact of partial DLT and type A MUT can be calculated by the Equations (7.1) and (7.2). The default value of f_{local} 1.0 is used in Equation (7.1).

$$Crash\ difference = Avg.Crashes\ before\ change\ \times (1 - CMF \times f_{local}) \tag{7.1}$$

$$Annual\ safety\ cost\ difference = crash\ difference \times comprehensive\ crash\ unit\ costs \tag{7.2}$$

Where f_{local} = localization factor for CMFs.

Therefore, the annual safety impact benefit of partial DLT is 6 × (1-1.11×1.0)× \$81300, which is negative \$53658; and the annual safety impact benefit of type A MUT is 6 × (1-0.6330×1.0)× \$81300, which is \$179023 in 2022 dollars.

The alternative intersection designs that have negative safety impacts will be removed from future consideration, therefore, the remaining UAID candidates after safety analysis is a type A median U-turn on the Pine Drive.

7.4.3 Construction Costs

A detailed construction cost estimation at the planning stage is unfeasible. To get the approximate construction cost at the planning stage, it is reasonable to estimate construction costs from a similar construction project that have been accomplished before. The construction cost of a type A MUT including road-widening resembles the construction costs of a DLT. Abdelraham (Abdelrahman et al., 2020) investigated the construction cost of three existing DLT construction projects, and the construction cost were \$4.4 million in year 2006, \$4.5 million in year 2007 and \$7.5 million in year 2007 respectively. To be noted, the \$4.4 million cost and \$4.5 million cost are the bid price of the project, and the \$7.5 million cost comprises all cost related to the project including planning/environmental, engineering, and right-of-way. Since in this case, no additional right-of-way is needed, the actual construction cost of case study intersection is closer to the \$4.4 million and \$4.5 million cost. Considering the currency inflation and neglecting the price variation in labor cost and material cost, the construction cost of a DLT is close to \$6.26 million. Abdelraham also investigated the maintenance cost of DLT and proposed an estimate cost of \$8000 annually. The average inflation rate for the past 30 years is 2.43%. Assume the average inflation rate for the next 30 years follows the pattern of the past 30 years, it is reasonable to assume that the average inflation rate for the next 30 years is 2.43%. Using Equation (7.3), the annual construction cost is obtained.

$$
\begin{aligned}
&\mathit{Annual\ construction\ cost} \\
&\qquad = \text{construction cost } \times \left[\frac{i}{1-(1+i)^{-n}}\right] \\
&\qquad + \mathit{maintenance\ cost}
\end{aligned}
\tag{7.3}
$$

Where:

Construction cost = the present value of the cost for construct UAID,

i = interest rate, and

n = designed service life of UAID

Substituting Equation (7.3) with numbers, the annual construction cost equals to $\$6.26 \times 10^{6} \times \left[\frac{2.43\%}{1-(1+2.43\%)^{-30}}\right] + \$8000$, which is \$304300 in 2022 dollars (rounded to the hundred).

7.4.4 Cost-Benefit Analysis

The cost-benefit ratio is chosen as one indicator to rank candidate alternative intersections. Equation (7.4) explains the calculation process.

$$
\begin{aligned}
&\text{Cost} - \text{benefit ratio} \\
&= \frac{\mathit{annual\ construction\ cost}\ (\$)}{\mathit{annual\ delay\ reduction\ benefit}(\$) + \mathit{annual\ crash\ reduction\ benefit}(\$)}
\end{aligned}
\tag{7.4}
$$

According to Subsection 7.4.2 and 7.4.3, the annual crash reduction benefit is \$179,032 in 2022 dollars and the annual construction cost is \$304,300 in 2022 dollars. To calculate annual delay reduction benefit, it is necessary to identify delay reduction of candidate alternative intersections and the Value of Travel Time (VTT). Equations (7.5) and (7.6) present the details.

Annual delay reduction benefit (7.5)

$$= \frac{Delay\ reduction}{C} \times VTT \times Persons\ Volume\ \times 8760$$

$$\text{Persons Volume} = \text{Vehicle volumes} \times \text{occupation rate} \quad (7.6)$$

Where:

Delay reduction = delay reduced by UAID in a circle length (s),

C = circle time (s),

Persons volume= total number of drivers and passengers entering the intersection per hour,

Vehicles volume = total number of vehicles entering the intersection per hour,

Occupation rate = average number of persons in a vehicle, and

8760 = number of hours in a year.

The USDOT (USDOT, 2016) recommended \$14.1 per person-hour in 2015 dollars for all local travels, and \$20.4 per person-hour in 2015 dollars for all intercity travels. Since this is an intersection close to the Interstate Highway I-195 and recreational facilities like Six Flags Great Adventure, Hurricane Harbor theme park and Adventure Crossing sports and recreational center, most of the traffic entering this intersection are intercity travels. Therefore, it is reasonable to assume the VTT of this intersection as \$20.4 per person-hour in 2015 dollars, which is \$24.84 per person-hour in 2022 dollars.

Subsection 7.4.1 estimated the lowest achievable LOS of type A MUT on Pine Drive with 30% left-turn percentage is LOS B and the LOS of corresponding conventional intersection is LOS F with an intersection delay of 98.9 s/veh. Therefore, the delay reduction of type A MUT on Pine Drive is 78.9 s/veh. The cycle time for type A MUT is 50 seconds.

According to the Equation (7.5), the annual delay reduction benefit is \$1678 million in 2022 dollars.

The cost-benefit ratio is 0.0002, and smaller than 1. Therefore, the benefits of type A MUT on Pine Drive are much higher its costs and should be preserved for the future consideration.

Since the type A MUT on Pine Drive is the only candidate, it is ranked in the first place in cost-benefit analysis, and therefore, the best option in cost-benefit analysis.

7.4.5 Performance Index Scoring System

The PI score of alternative intersection candidate is given by Equation (7.7).

$$PI = W_1 \times B_1 + W_2 \times B_2 - W_3 \times C_1 \tag{7.7}$$

Where :

W_1, W_2, W_3 = the weight assigned by the stakeholders for delay reduction, crash reduction and construction cost, respectively in Stage 1,

B_1 = the annual delay reduction benefit in million dollars,

B_2 = the annual crash reduction benefit in million dollars, and

C_1 = the annual construction cost in million dollars.

As stated in Section 7.2 target and constraints collection stage, the value of W_1,

W_2, W_3 is 0.5, 0.4, and 0.1 respectively. The value of B_1, B_2, and B_3 were calculated in the cost-benefit analysis. According to Equation (7.7), the PI score of type A MUT on Pine Drive is 839.042.

Since the type A MUT on Pine Drive is the only candidate, it is ranked in the first place in the PI scoring system, and therefore the best option in PI scoring system.

CHAPTER 8

CONCLUSION

In this research, a three-stage planning methodology is proposed to assist alternative intersection design selections. Stage 1 is the initial stage, focusing on collecting project objectives and constraints. Stage 2 is the preliminary screening stage, aiming at ruling out inappropriate designs before the detailed analysis. Stage 3 is the detailed analysis, including detailed mobility analysis, safety analysis, construction cost analysis, cost-benefit analysis, and performance index scoring. If no candidate survives the three-stage screening, then the practitioners should go back to the stage one and loose some project targets and constraints, and then repeat the procedures. Figure 8.1 illustrates the methodology details.

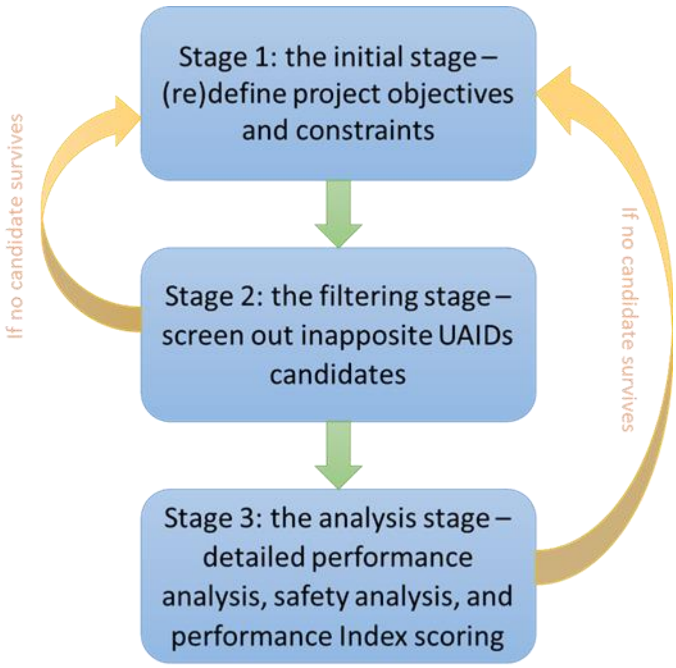


Figure 8.1 The structure of UAIDs selection model.

In Stage 1, detailed project improvement objective list and constraints list should be provided. In the improvement objective list, improving intersection safety, reducing intersection delay and project's financial feasibility are the most common project improvement objectives. While identify the project improvement objectives, an objective weight should also be assigned to each project improvement objective. This objective weight is one of the key components in stage three's performance index scoring system. Apart from the project improvement objectives, project constraints and general information like ROW constraints, budget constraints, intersection geometries, vehicle volumes, and desired intersection Level of Service should also be collected.

To screen out the inappropriate alternative intersection designs, a dedicatedly designed breakdown test is proposed in Stage 2. The breakdown test is able to identify the most congested location of an alternative intersection design and the most congested location is referred as intersection bottleneck in the breakdown test. A set of breakdown test tables is developed for each alternative intersection. In the breakdown test tables, traffic conditions are simplified into nine conditions for each alternative intersection design: (1) 1000vph with 10% left-turn percentage, (2) 1000vph with 20% left-turn percentage, (3) 1000vph with 30% left-turn percentage, (4) 2000vph with 10% left-turn percentage, (5) 2000vph with 20% left-turn percentage, (6) 2000vph with 30% left-turn percentage, (7) 3000vph with 10% left-turn percentage, (8) 3000vph with 20% left-turn percentage, and (9) 3000vph with 30% left-turn percentage. Simulation software VISSIM (version 10) is used to evaluate the mobility performance of the nine traffic conditions for the three alternative intersection types: DLT, MUT, and RCUT. Providing the corresponding intersection type, vehicle volume, and left-turn movement percentages, and approximate the traffic conditions to the nearest threshold, the practitioner is able to find out the approximate LOS of each movement. Any intersection designs with movements experiencing LOS E or higher will be removed from future consideration.

In Stage 3, detailed mobility analysis and safety analysis are performed. Service volume tables for Partial DLT and TWSC MUT are developed to assist practitioners identify alternative intersection candidates. Delay chart is provided for RCUT since the performance of RCUT does not follow the same pattern as partial DLT and TWSC MUT. With intersection type available and approximate the actual left-turn percentage to the nearest threshold, the users of the service volume tables are able to locate the service volume for the desired LOS. If the traffic volume does not meet the service volume requirements, the candidate design will be removed from future consideration. The users of RCUT delay chart are also able to locate the intersection delay with given traffic volume and left-turn percentage. If the intersection delay exceeds the desired LOS requirement, the candidate design will be removed from future consideration.

Table 8.1 Generalized Annual Average Daily Volumes for Partial DLT

Partial Displaced Left-turn Intersections						
Left turn	Intersection spacing (ft)	A	B	C	D	E
10%	350	2500	16200	24100	28500	29900
20%	350	1100	16900	24200	27100	30100
30%	350	--	16400	23800	26600	29000

Table 8.2 Generalized Annual Average Daily Volumes for TWSC MUT

Median U-turn Intersections						
Left turn	Intersection spacing (ft)	A	B	C	D	E
10%	660	11000	18800	23900	25800	27600
20%	660	8200	15900	17800	19600	23100
30%	660	3700	12900	14500	15900	17400

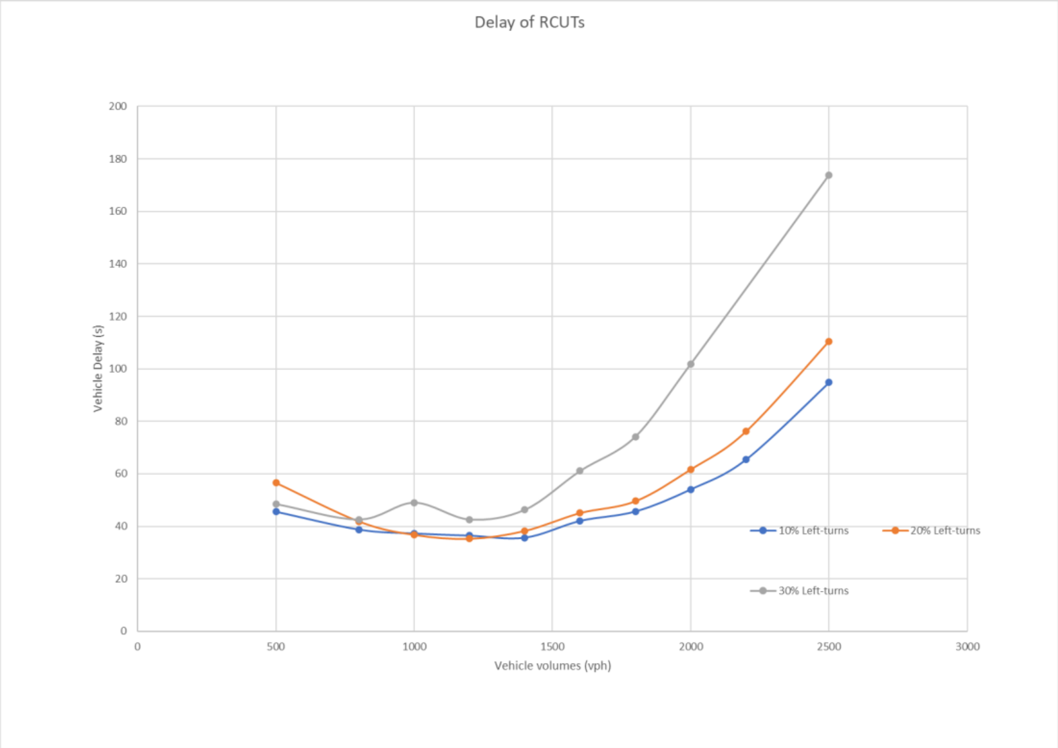


Figure 8.2 Delay of simulated Restricted Crossing U-turn Intersections.

In the detailed alternative intersection safety analysis, the best available CMF for DLT, MUT, and RCUTs are identified. The CMF for total crashes will be used in the safety analysis. According to Chapter 6, he current best available CMF for a partial DLT of total crashes is 1.11, the best available CMF for type A MUT of total crashes is 0.6330, and the best available CMF for RCUT of total crashes is 0.7632. To be noted, due to the limitation of available data, the developed current best available CMF should be used with caution. Whenever a better-quality CMF is available, the value of CMFs in this research should be replaced with the updated CMF. Equations (8.1) and (8.2) show the detailed calculation for the safety impact cost difference. The candidate with negative safety impact will be removed from future consideration.

$$Crash\ difference = Avg.Crashes\ before\ change\ \times (1 - CMF \times f_{local}) \tag{8.1}$$

$$Annual\ safety\ cost\ difference = crash\ difference \times comprehensive\ crash\ unit\ costs \tag{8.2}$$

Where f_{local} = localization factor for CMFs.

If one or more candidates survive the detailed mobility analysis and safety analysis, two measurements are chosen as the ranking indicator for the survivals: cost-benefit ratio and Performance Index (PI) score. Equations (8.3) and (8.4) present the calculation process for cost-benefit ratio and the PI score. The annual construction cost and annual delay reduction benefit in Equations (8.3) and (8.4) can be calculated by Equations (8.5) and (8.6). The candidate with lowest cost-benefit ratio and highest PI score is the best option.

$$\text{Cost} - \text{benefit ratio} = \frac{annual\ construction\ cost\ (\$)}{annual\ delay\ reduction\ benefit(\$) + annual\ crash\ reduction\ benefit(\$)} \tag{8.3}$$

$$PI = \frac{W_1\ \times B_1 + W_2\ \times B_2 - W_3\ \times C_1}{B1 + B2} \tag{8.4}$$

$$Annual\ construction\ cost = \text{construction cost}\ \times \left[\frac{i}{1 - (1 + i)^{-n}}\right] + maintenance\ cost \tag{8.5}$$

Where:

Construction cost = the present value of the cost for construct UAID,

i = interest rate, and

n = designed service life of UAID

$$\text{Annual delay reduction benefit} \tag{8.6}$$

$$= \frac{Delay\ reduction}{C} \times VTT \times Persons\ Volume\ \times 8760$$

$$\text{Persons Volume} = \text{Vehicle volumes} \times \text{occupation rate} \tag{8.7}$$

Where:

Delay reduction = delay reduced by UAID in a circle length (s),

C = circle time (s),

Persons volume= total number of drivers and passengers entering the intersection per hour,

Vehicles volume = total number of vehicles entering the intersection per hour,

Occupation rate = average number of persons in a vehicle, and

8760 = number of hours in a year.

Advantages and Contributions

The advantage of this planning methodology is that it is the first alternative intersection planning methodology that incorporate HCM's methodology in estimating alternative intersection LOS. This provides a nationally recognized criteria surpasses the previously developed planning methodologies that only applied to a specific state. The breakdown test tables, service volume tables and delay charts are developed for alternative intersections planning. This methodology saves the time and energy that is usually necessary for practitioners performs complicated simulation analysis and provides an equally competitive result. Simultaneously it eliminates the possible mistakes in the practitioners' simulation process and insures a consistent result regardless of the practitioner's skills and knowledge.

Limitations and Future Research

Due to the limitation of available data and related research, only three alternative intersection designs are covered in this research. When more related data and research available, more alternative intersection designs should be included in this planning methodology. Also, the CMF values adopted in this research should be updated whenever a better-quality CMF is available. The service volume tables and delay charts can also be expanded to cover a wider range of traffic conditions, for example, adding a series of service volume tables for alternative intersections with different intersection spacing.

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