

Operational Analysis of Alternative Intersections

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Abstract

Alternative intersections and interchanges, such as the diverging diamond interchange (DDI), the restricted crossing u-turn (RCUT), and the displaced left-turn intersection (DLT), have the potential to both improve safety and reduce delay. However, partially due to lingering questions about analysis methods and service measures for these designs, their rate of implementation remains low. This research attempts to answer three key questions. Can alternative intersections and interchanges be incorporated into the existing level of service and service measure schema, or is a new service measure with an updated level of service model required? Is the behavior of drivers at alternative intersections fundamentally similar to those at conventional intersections, such that traffic microsimulation applications can accurately model the behaviors observed in the field? Finally, is the planning level tool made available through FHWA an accurate predictor of the relative performance of various alternatives, or is an updated tool necessary?

Discussion and case study analysis are used to explore the existing level of service and service measure schema. The existing control delay measure is recommended to be replaced with a proposed junction delay measure that incorporates geometric delay, with the existing level of service schema based on control type recommended to be replaced by a proposed schema using demand volume. A case study validation of micro- and macroscopic analysis methods is conducted, finding the two microscopic methods investigated to match field observed vehicle delays within 3 to 7 seconds for all designs tested, and macroscopic HCM method matching within 3 seconds for the DDI, 35 seconds for the RCUT, and 130 seconds for the DLT design. Taking the critical lane analysis method to be a valid measure of operations, the demand-volume limitations of each alternative design is explored using eighteen geometric configurations and approximately three thousand volume scenarios, with the DLT design predicted to accommodate the highest demand volumes before failure is reached. Finally, six geometries are examined using both the planning-level tool and the validated microsimulation tool, finding that the curve of the capacity-to-delay relationship varies for each alternative design, invalidating the use of critical lane analysis as a comparative tool.

Dedication

This work is dedicated to my wife Kerry, and my children, Jonas, Emily, and Isaac. It took a lot of teamwork to succeed during graduate school, and they earned this every bit as much as I did.

Table of Contents

Abstract	ii
Dedication	iii
List of Figures	v
List of Tables	viii
Chapter 1. Introduction	1
Chapter 2. Literature Review	9
Chapter 3. Operational Measures of Effectiveness for Alternative Intersections	24
Chapter 4. Validation of Simulation for Alternative Intersections	38
Chapter 5. Sample Comparative Analysis of Alternative Geometries	59
Chapter 6. Conclusions from Planning-Level Analysis	75
Chapter 7. Identifying the disconnect between planning and simulation analysis	95
Chapter 8. Capacity-based Predictions and Delay-based Results	112
Chapter 9. Conclusions and Recommendations for Future Research	128
References	134

List of Figures

FIGURE 1-1	Research flow chart	6
FIGURE 1-2	Chapter flow chart	8
FIGURE 2-1	Sample jughandle geometry	13
FIGURE 2-2	Sample median u-turn geometry	14
FIGURE 2-3	Sample restricted crossing u-turn geometry	15
FIGURE 2-4	Sample displaced left-turn geometry	16
FIGURE 2-5	Sample quadrant roadway geometry	16
FIGURE 2-6	Sample diverging diamond interchange geometry	18
FIGURE 3-1	Aerial location maps for data collection sites	28
FIGURE 3-2	Flow chart to calculate delay using field data and proposed method	31
FIGURE 4-1	Aerial location maps for data collection sites	44
FIGURE 4-2	Observed peak period signal operations for DDI, DLT, and RCUT facilities	48
FIGURE 4-3	Displaced left-turn green band platoon progression	49
FIGURE 4-4	Restricted crossing u-turn green band platoon progression	50
FIGURE 4-5	Delay observations for diverging diamond interchange	52
FIGURE 4-6	Delay observations for displaced left-turn intersection	53
FIGURE 4-7	Delay observations for the restricted crossing u-turn intersection	54
FIGURE 5-1	Example through-about geometry	61
FIGURE 5-2	Existing domestic implementations of the through-about	62
FIGURE 5-3	Geometry of (a) conventional, (b) roundabout, and (c) through-about intersections	63
FIGURE 5-4	Average delay as a function of total volume on all approaches	68

FIGURE 5-5	Average delay as a function of percent left-turning vehicles on major approach	68
FIGURE 5-6	Average delay as a function of percent of total volume on major approach	69
FIGURE 5-7	Through-movement delay on major arterial	70
FIGURE 5-8	Conflicts created by (a) conventional signalized, (b) roundabout, and (c) through-about intersection designs	71
FIGURE 5-9	Fuel consumption measures from volume magnitude sensitivity analysis	72
FIGURE 6-1	Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major road and one lane approaches on the minor roads	80
FIGURE 6-2	Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major road and one lane approaches on the minor roads	81
FIGURE 6-3	Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major and minor roads	82
FIGURE 6-4	Correlation of critical sum values against the average intersection delay expected for a conventional intersection, (a) without and (b) with saturation flow rate adjustments	88
FIGURE 6-5	Sample applications of critical sum to the signal warrant framework, including a (a) conventional signalized intersection (2x1), and a (b) restricted crossing u-turn intersection (2x1)	89
FIGURE 6-6	Maximum operational thresholds based on the Critical Sum method for (a) conventional signalized intersection, (b) roundabout, (c) Jughandle, (d) median u-turn, (e) restricted crossing u-turn, and (f) displaced left-turn designs	91
FIGURE 6-7	Capacity failure thresholds for typical intersection designs with (a) one approach lane from each direction, (b) two approach lanes on the major roadway and one approach lane on the minor roadway, and (c) two approach lanes from each direction	92
FIGURE 7-1	Geometric layout and signal phases of quadrant roadway intersection	99
FIGURE 7-2	Pattern of left turns for quadrant roadway intersection	100

FIGURE 7-3	Screen captures of VISSIM roadway network simulation	102
FIGURE 7-4	Change in critical sum due to the quadrant roadway design	103
FIGURE 7-5	Change in intersection delay (HCM) due to quadrant roadway design	105
FIGURE 7-6	Change in turn-movement delay (HCM) due to quadrant roadway design	105
FIGURE 7-7	Change in intersection delay (simulated) due to quadrant roadway design	105
FIGURE 7-8	Change in average travel time due to quadrant roadway design	105
FIGURE 7-9	Use of critical sum as indicator variable	109
FIGURE 7-10	Change in critical sum as an indicator of change in delay or travel time	110
FIGURE 8-1	Geometries used for comparative analysis	115
FIGURE 8-2	Sample signal phase diagrams for the simulation of alternatives	117
FIGURE 8-3	Anticipated failure thresholds for intersections investigated	120
FIGURE 8-4	Control delay and total delay for a sample volume scenario, with (a) total average delay, (b) major approach left-turn, (c) major approach through, and (d) minor approach	121
FIGURE 8-5	Conventional signalized intersection: relationship of average delay to (a) total volume and (b) the critical sum value	122
FIGURE 8-6	Average total delay relating to total volume and critical sum measures	123
FIGURE 8-7	Critical sum versus average delay for selected lane groups	124

List of Tables

TABLE 3-1	Level of service thresholds for HCM 2010, in terms of control delay (seconds/vehicle)	26
TABLE 3-2	Origin-destination traffic demands by location	27
TABLE 3-3	Measures of travel time and delay by location and turn movement	32
TABLE 3-4	Proposed LOS threshold relationships with peak hour demand volumes	34
TABLE 3-5	Origin-destination traffic demands and resulting LOS threshold metric	35
TABLE 4-1	Pretimed-equivalent signal timings	47
TABLE 4-2	Origin-destination traffic demands by location	50
TABLE 4-3	Average delay per vehicle	51
TABLE 5-1	Volume scenarios used in sensitivity analysis of (a) volume magnitude, (b) left-turn percentage, and (c) major/minor split	67
TABLE 5-2	Cost indicators for alternatives analysis	72
TABLE 6-1	Generation of volume combinations for analysis	79
TABLE 6-2	Equation parameters to generate failure thresholds, with validity measures	90
TABLE 7-1	Generation of volume combinations	100
TABLE 7-2	Sample Highway Capacity Manual method delay calculations	104
TABLE 8-1	Capacity-based predictions and delay-based results	125

Chapter 1. Introduction

1.1 Background and Motivation

The principal arterial system amounts to less than 10 percent of US street system mileage, but accounts for nearly one-half of all vehicle-miles travelled [1]. As nationwide congestion continues to increase, the arterials bear much of the brunt, with ever increasing delays and limited opportunities for increasing capacity. These roadways serve an important purpose in our transportation network by balancing access with mobility. Though the need exists to increase their capacity, their purpose remains balanced and converting them to freeways is not a valid solution.

With arterial corridors in failure, it is often the case that the single greatest limiting factor for capacity are signalized intersections with multiple left turn phases, which fail to accommodate high through-put volumes while also accommodating turning movement and side-street flows. The addition of lanes to increase capacity is insufficient to solve the problem of limited green-time percentage for through traffic. Additional lanes can often exacerbate existing problems, as intersection size expands both lost time and minimum pedestrian phase lengths.

Alternative intersections seek to address these capacity issues by segregating or diverting turning movements, which concurrently reduces and separates the number of conflict points. Some professionals object to alternative solutions due to a perceived increase in cost, but when a high cost intersection improvement can postpone the need to add additional lanes along a corridor or prevent the construction of an interchange, the cost savings of alternative intersections become apparent. Due to the potential for increased safety, congestion reduction, and cost savings, the Federal Highway Administration (FHWA) has recently been an advocate for what they refer to as “novel” intersection designs [2].

Even with the backing of the Federal Government, these designs remain fringe solutions, as problems with driver expectancy continue to be a legitimate concern. While public information campaigns in advance of implementation can assist local drivers, a certain percentage of all traffic will be new to an area. Convincing an agency responsible for any potential alternative intersection that the anticipated benefits outweigh the unknown liabilities often involves providing multiple existing examples with observed data. The logical fallacy of circular cause and consequence has prevented these unique designs from being put into practice, with designs failing to be implemented on the basis of them having not already been built elsewhere.

1.1.1 Overview of Alternative Intersections

The category of intersection design known as alternative intersections was known as unconventional intersections only five years ago, and five years from now may be so commonplace as to no longer hold the title alternative. These designs have come a long way since the first proposal of what is now called the restricted crossing u-turn design in the late 80's

[3]. The near-universal adoption of the modern roundabout as a successful alternative intersection treatment is creating a more progressive outlook within transportation engineering practice, and other alternative designs may be ready to follow the modern roundabout's example. Sustained support by the Federal Highway Association (FHWA) for these designs has helped in normalizing them, such as with the production of the Alternative Intersections and Interchanges Informational Report [4].

These designs work by rerouting some of the turning movements made at a junction, often reducing and separating conflict points, with theoretical benefits to both safety and mobility. Conflict points represent each position where the paths of two vehicles cross, with potential collision locations identified as crossing, merging, or diverging conflicts. Any reduction in the number of potential conflicts, especially of the most dangerous crossing type, provides proportional benefit to intersection safety. This reduction in conflict points at these designs often comes with a reduction in the number of signal phases at the junction, providing a reduction in delay to through movements on the major arterial, while often increasing travel time and travel distance for secondary movements. Depending on the degree of benefit to the through movement on the major arterial and the degree of degradation to the secondary movements, the overall average delay may be more or less than that experienced by an equivalent conventional signalized intersection.

1.2 Research Questions and Contribution

This research seeks to answer questions about the operational analysis of alternative intersection and interchange designs. While many questions are explored by this work, three main focus topics can be identified. The first question explored is how the existing level of service and service measure schemas for conventional intersections can be modified for application to alternative intersections and interchanges. Next, field data is used for comparison to micro- and macroscopic modeling techniques, to determine the ability of these methods to accurately model the operational performance of these designs. Finally, the state-of-the-practice planning level tool is explored, both in terms of what it can tell us about the variety of designs, as well as how well it serves as a prediction tool.

The existing service measure for operational performance of intersections and interchanges is based on control delay, and translated into level of service grades using a system that differentiates between various types of intersection control, be it roundabout, signalized intersection, signalized grade-separated interchange, etc. As the control delay measure explicitly does not include travel time impacts caused by roadway geometry, a new measure of delay which includes this geometric component is developed, called junction delay by the author. Examining the schema that translates control delay into level of service, the alternative designs are seen to lack a place, as their behavior is somewhere between that of a conventional signalized intersection, and a grade-separated interchange. A new schema is proposed based not on control type at the junction, but based on total demand volume at the location, split into three categories

of small, medium, and heavy traffic, roughly corresponding to unsignalized, signalized, and grade-separated conditions.

Many of the alternative intersection designs have been implemented sparingly, some not at all, such that there is a general lack of field data available to examine how they impact driver behavior. Microsimulation of these designs is relied upon as a robust analysis technique, and is essential in modeling comparative analysis when considering these designs for a new location. However, many practitioners are known to use macroscopic analysis tools to analyze performance of alternatives because they lack access to the more costly and time-intensive microscopic tools. Using field data from a diverging diamond interchange, a restricted crossing u-turn intersection, and a displaced left-turn intersection design, peak hour traffic was observed, with individual travel times recorded for vehicles traversing the facility. Using the field geometry combined with the observed signal timing and demand volumes, these conditions are then modeled both microscopically and macroscopically to examine the degree to which each of these methods is able to accurately portray the behavior as observed in the field.

The cost of microscopic simulation can be prohibitive for consulting engineers working on projects with tight budget constraints, so it is necessary to model only the most likely candidate geometries when modeling for comparative analysis of an intersection improvement project. The selection of likely candidate geometries is an essential step in the design process for intersection improvements, and is completed for alternative geometries using a tool recently developed and provided by FHWA. This tool provides a simple method for comparative analysis between many alternatives, allowing for general observations to be made about the nature of demand volume that can be served by each. However, this research seeks to examine if the recommended subset of designs proposed are indeed the best performing alternatives for a given situation, and if not, how are the assumptions of the model incorrect.

1.3 Methodology

The primary analysis tools used to conduct the research include video recordings of in-situ traffic conditions, and operational analysis using various tools at the preliminary-engineering (planning level), design (macroscopic) and microsimulation levels. The development of new level of service measures relied primarily on field-observed traffic conditions. The validation of macro- and microsimulation tools used the tools themselves, in combination with field-observed traffic conditions. Comparative analysis of alternatives using the developed service measures relied on microscopic analysis tools. The exploration of conclusions regarding the various alternative geometries relied solely on the planning-level analysis tool. The case study testing the results of the planning-level tool used a single geometry to compare the tool against microsimulation results for a variety of volume scenarios, while the more extensive testing of the relationship between capacity and delay used the same two tools but with a wider variety of geometric configurations and a smaller variety of volume scenarios.

The investigation of service measures at alternatives utilized field data collected for three alternative designs, including the diverging diamond interchange (DDI), the displaced left-turn intersection (DLT), and the restricted crossing u-turn intersection (RCUT) designs. Video data

using multiple GoPro cameras was collected, covering both AM and PM peak periods with simultaneous feeds covering each direction of traffic. Using a combination of the video input feeds and the roadway geometry, a location along each approach and exit from the junction was chosen as the limits of the network for the sake of travel time calculations. Time was logged for each vehicle as it crossed the threshold entering the network, and subsequently as it crossed the corresponding threshold exiting the network, with the individual travel times then aggregated by turn movement to determine the mean travel times for each origin-destination pairing. Two theoretical base travel times are denoted, which are the vehicle-path travel time from the radius and the proposed node-to-centroid base travel time. The difference between these two values for each given turn movement provides the geometric component of the delay. The combination of conventional control delay with the proposed geometric delay component yields the new measure of junction delay. The junction delay for the observed geometric configurations is then used to illustrate the proposed level of service methodology utilizing a demand volume schema.

The video data used for the investigation into service measures is also used for the validation of micro- and macroscopic analysis methods. In addition to the observed vehicle travel times, the video data additionally provides insight into the signal phasing as applied in the field, with analysis conducted using the in situ signal phasing and timings, rather than optimized for the specific volume demands observed. By coordinating the video times from the various feeds, signal offsets are calculated for those designs with multiple signalized intersections operated from a central master controller. Model creation is conducting taking any readily observable information, such as the roadway geometry, the speed limits, the signal timings, and the demand volumes, as opposed to the calibration of driver behavior which was not done for the purpose of examining the base results one would find from comparative analysis of proposed solutions. The Highway Capacity Manual method is used for macroscopic analysis, with INTEGRATION and VISSIM software used as a validation of microsimulation analysis. One note on the difference between the two is that microscopic analysis has built in variance in each simulation run. To collect a sufficient amount of data, both simulation software application were run for ten seeds of twenty-five minutes each, with fifteen minutes of data used from each run, with recorded vehicles scheduled to enter the network between five and twenty minutes during the simulation. Having validated the microsimulation approach for use in comparative analysis of alternative geometries, INTEGRATION is then used in this fashion to examine the relative performance of the through-about, roundabout, and conventional signalized intersection designs.

The planning-level analysis tool utilizes the critical sum method which calculates capacity at an intersection location by examining the combination of critical movements at the facility. The idea behind the calculation is that while a traffic signal services every turning movement demand at an intersection, only some of those demands are governing the signal timing, while the others just happen to occur simultaneously. The critical sum method determines which movement is governing at any given time during a signal cycle, and sums the demands for each sequential critical movement, determining a total value for the demand at the intersection, in units of vehicles per hour per lane (veh/ln-hr). The specific intersection designs analyzed include the

conventional signalized intersection, the roundabout, the jughandle, the median u-turn, the restricted crossing u-turn, and the displaced left-turn. Three different sizes of intersection were considered for each of the six designs, with an effort to make each design an “equivalent” option within a given size regime. Documenting the specific equations utilized for each of the eighteen geometries analyzed, a total of approximately 335,000 volume combinations are applied to each configuration to determine the failure thresholds for each design, assuming the method to produce valid results.

The validated microsimulation methodology is compared against the planning-level method using the case study of the quadrant roadway design. A total of 36 volume scenarios are tested using the two methods, examining the relative results from both the quadrant roadway and an equivalent conventional intersection. The quadrant roadway investigation primarily focuses on the question of whether, for any given volume scenario, an advantage seen for a design based on the critical sum value translates to a similar advantage for that design in terms of delay. The final investigation moves beyond the individual case study comparison, and compares six intersection geometries, including the conventional signalized intersection, the modern roundabout, the median u-turn, the restricted crossing u-turn, the displaced left-turn, and the jughandle designs. In this case sensitivity analysis is conducted on each design using both microsimulation and the planning-level tool, seeking to determine if the relationship between capacity and delay is consistent between designs.

A summary of the research progression and how each investigation is related to the other is provided in **FIGURE 1-1**.

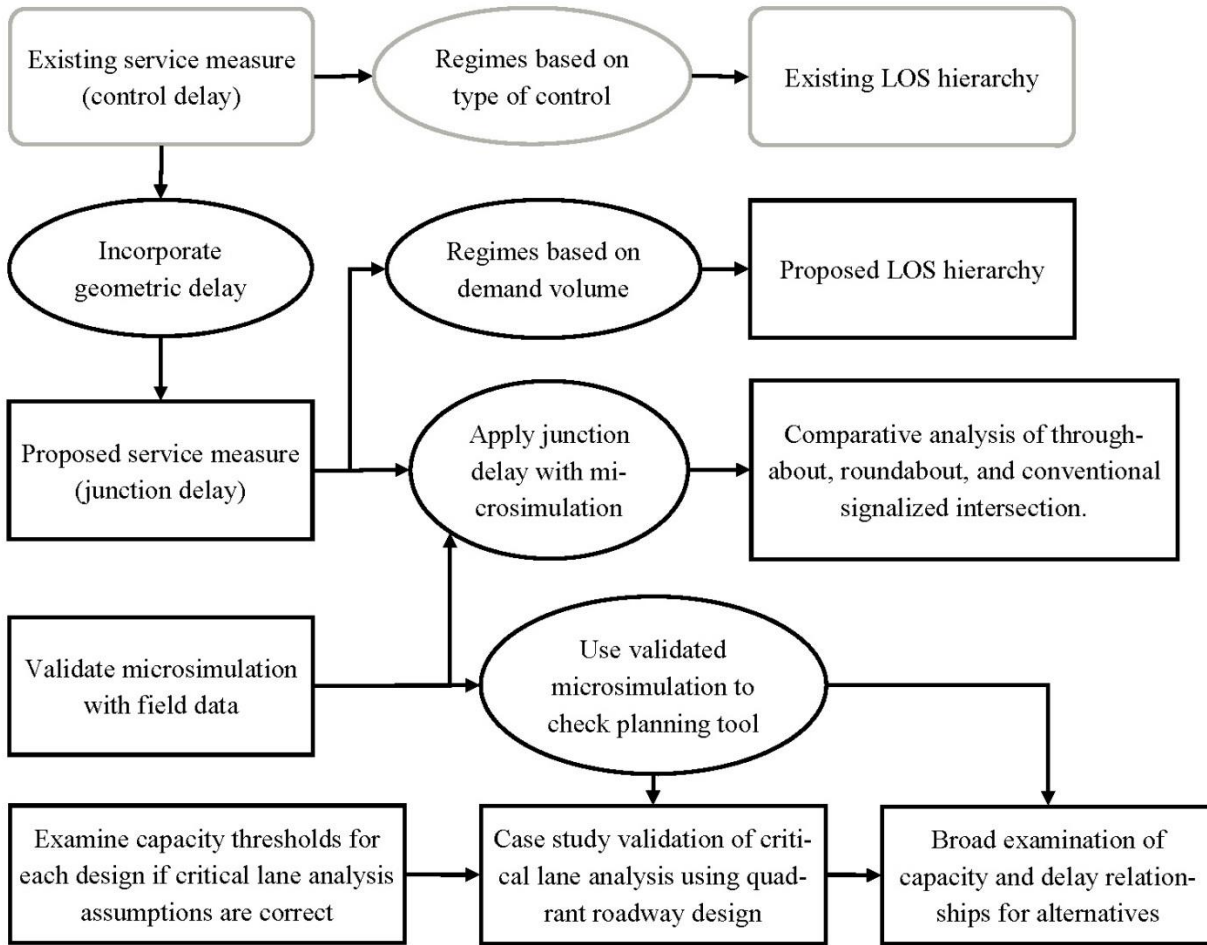


FIGURE 1-1 Research flow chart

1.4 Document Layout and Attribution

This research is organized into nine chapters, beginning with the introduction as the first chapter. The second chapter is intended to provide a review of the relevant literature on alternative intersections including the motivation for using them, intersection geometries, interchange geometries, methods for comparative analysis, and the adoption and implementation of these designs. Each additional chapter includes its own literature review section, highlighting the purpose of each research endeavor in the context of previous work. The third through the eighth chapters are the body of this work, each consisting of an individual paper published by or under review for a conference or journal. The ninth chapter provides summary conclusions for the dissertation.

The third chapter is under review for the 2016 Transportation Research Board conference and the Transportation Research Record journal. This paper, authored by John Sangster and advised by Hesham Rakha, is titled *New Perspectives on Delay and Level of Service*. Its contribution to the narrative of the dissertation document is to provide an updated methodology

for the delay service measure, incorporating the necessary adjustments to allow for a unified approach to analyzing intersections and interchanges, both conventional and alternative.

The fourth chapter is under review for the 2016 Transportation Research Board conference and the Transportation Research Record journal. This paper, authored by John Sangster and advised by Hesham Rakha, is titled *Validating Analysis Methods for Alternative Intersections using Field Data*. Its contribution to the narrative of the dissertation document is to provide a validation of the state of the art practice of conducting operational analysis of alternative intersections using traffic microsimulation software.

The fifth chapter was originally presented at the 94th annual meeting of the Transportation Research Board, in January of 2015. This paper, authored by John Sangster and advised by Hesham Rakha and Ahmed Al-Kaisy, is titled *Comparative Analysis of the Through-about, Roundabout, and Conventional Signalized Intersection Designs*. Its contribution to the narrative of the dissertation is to provide an example of how comparative analysis can be conducted for intersection designs which are rare in implementation, and rarer in the literature.

The sixth chapter was originally presented at the 93rd annual meeting of the Transportation Research Board, in January of 2014. This paper, authored by John Sangster and advised by Hesham Rakha, is titled *Implications of CAP-X: Operational Limitations of Alternative Intersections*. Its contribution to the narrative of the dissertation document is to demonstrate the capacity analysis methods used in programs such as CAP-X, and provide some of the conclusions that are reached using such methods.

The seventh chapter was originally presented at the 91st annual meeting of the Transportation Research Board, in January of 2012. This paper, authored by John Sangster and advised by Hesham Rakha, is titled *Critique of the Critical Sum Method: A Case Study on the Quadrant Roadway Design*. Its contribution to the narrative of the dissertation is to raise questions about the validity of the capacity-based analysis methods used for preliminary engineering, in this case using the case study of a single alternative intersection design.

The eighth chapter was originally presented at the 94th annual meeting of the Transportation Research Board, in January of 2015. This paper, authored by John Sangster and advised by Hesham Rakha, is titled *Capacity-based Predictions and Delay-based Results: CAP-X Limitations and Suggestions for Improvement*. The analysis in this chapter examines six intersection geometries using the total delay measure discussed in chapter 7, looking for relationships between capacity and delay.

The ninth chapter provides summary conclusions for the dissertation document, summarizing the findings of the individual chapters, and drawing overall conclusions from the body of work.

Presenting the flow of the research in visual format, the **FIGURE 1-2** expresses a simplified view of how the various chapters are related.

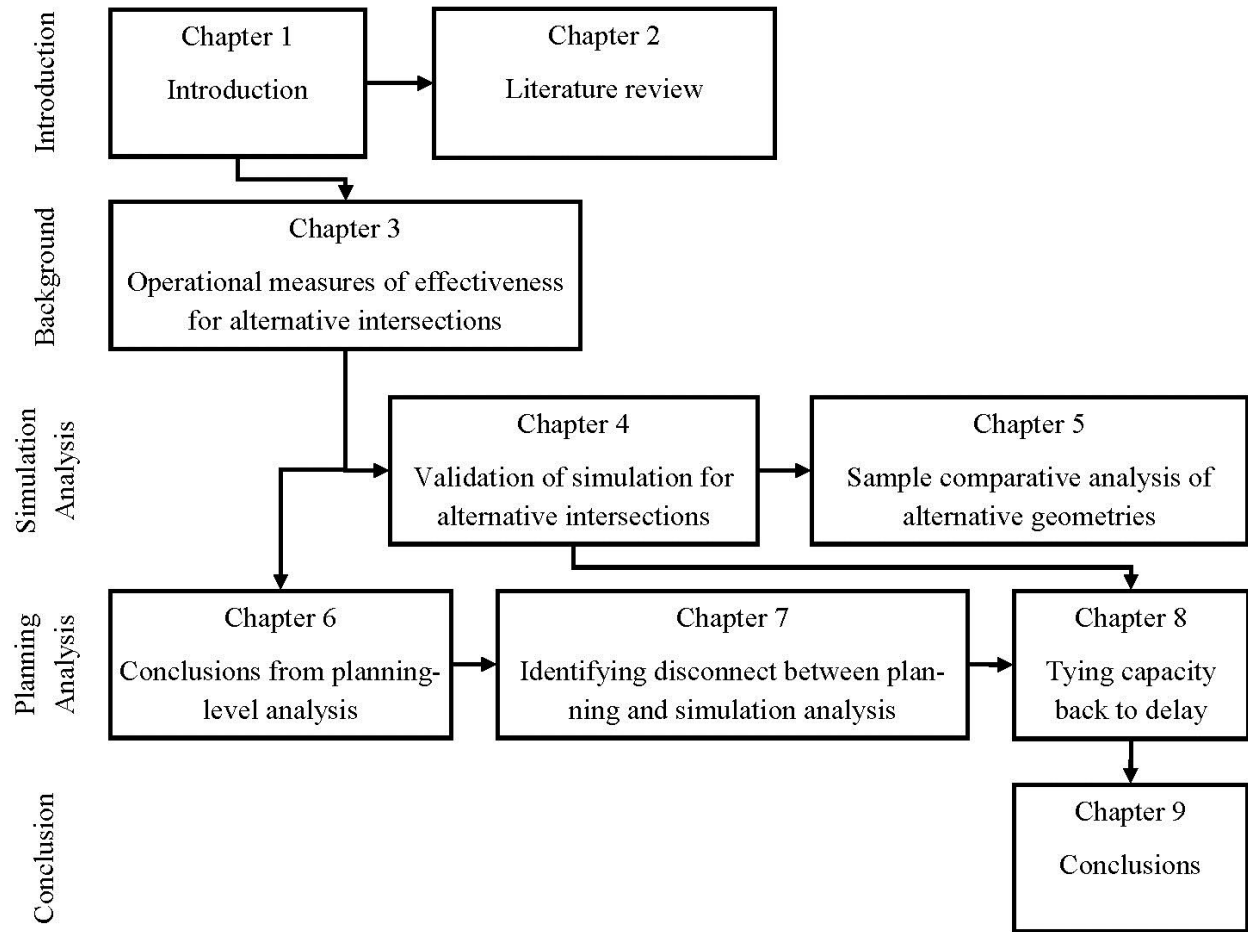


FIGURE 1-2 Chapter flow chart

Chapter 2. Literature Review

Though a relatively new topic, there is a growing body of work published examining alternative intersections. This chapter provides a review of the current literature on the subject. A short discussion is provided on the most common general references used in intersection design and analysis. A number of summative documents have been produced on alternative intersections, which are subsequently called upon throughout the chapter to fill in any gaps in specific sub-topic areas. A discussion of the motivation for seeking intersection alternatives is followed by a section detailing the background of each intersection and interchange geometric design. Having established the key features of each design, the various methods in the literature for conducting comparative analysis of them is then discussed. Finally, a review of the adoption and implementation history for these designs is conducted.

2.1 Intersection Design and Analysis

The standard reference on the topic of geometric design of roadways is A Policy on Geometric Design of Highways and Streets, commonly referred to as the “Green Book,” published by the American Association of State Highway and Transportation Professionals (AASHTO), with the sixth edition published in 2011 being the most current [6]. The most pertinent sections of the Green Book applicable to this research is Chapter 9 – Intersections, with Chapter 10 – Grade Separations and Interchanges related to a lesser degree. This manual provides design guidelines necessary both to provide safe and consistent facilities throughout the United States. Specific information on intersections includes sight distance calculations, channelization of roadways, minimum turning radii, design of auxiliary lanes, design of median openings, and provisions for indirect left-turn movements. Basic functionality is described within the Green Book for a number of alternative designs, including the jughandle, displaced left-turn, median u-turn, and roundabout. Interchange information includes conventional diamond, cloverleaf, and directional overpass designs, as well as the less frequently used single point urban interchange (SPUI) and double roundabout designs.

Whereas the Green Book provides the guidelines for geometric layout of roadways, the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) provides regulations for pavement markings and signage for these facilities [7]. This document serves as a legal national standard, and is administered by the Federal Highway Administration (FHWA) of the United States Department of Transportation (US DOT). The latest version of the MUTCD is the 2009 edition. While the signage section of this reference was not extensively utilized, the pavement marking guidelines were used when conducting the geometric design for the intersections analyzed. An additional use for this manual is the signal warrants it provides, providing guidance on the magnitude of traffic that should be serviced with signalized control.

A secondary reference, FHWA’s Signalized Intersections: Informational Guide, published in 2004, provides a thorough document bringing together the geometric design of intersections with the signage and control, as well as safety and operational analysis methodologies, and a wide

array of intersection treatments [8]. Of the standard reference materials, this document provides both the most information about intersection alternatives, as well as the most thorough and compelling argument for their use. Unfortunately, the Signalized Intersections guide does not provide a sufficient amount of information on the design and operation of signals, which is covered in another FHWA publication, the Traffic Signal Timing Manual, from 2008 [9].

One critical component thus far missing from the generalized reference material is the ability to analyze the operational performance of intersections, which is addressed in the Highway Capacity Manual, a publication of Transportation Research Board, with the most recent copy in circulation being the 2010 version [5]. The current iteration of the document is separated into four volumes. The first three volumes are hard-copy print volumes, with the fourth volume being an online document allowing for updated content between releases. The first volume addresses concepts including modal analysis, traffic flow and capacity, quality and level of service, HCM and alternative analysis tools, as well as the interpretation of results from HCM and alternative analysis tools. The second volume covers uninterrupted flow facilities, including freeway facilities, basic freeway segments, freeway weaving segments, freeway merge and diverge segments, two-lane highways, and multilane highways. The third print volume covers interrupted flow facilities, including urban street facilities, urban street segments, signalized intersections, two-way and all-way stop controlled intersections, roundabouts, interchange ramp terminals, and pedestrian and bicycle facilities. At this time, there is not accommodation made within the HCM for alternative intersection designs.

2.2 Alternative Intersections and Interchanges

The first summative report on alternative geometries came from consulting, produced by Jonathan Reid while working for Parsons Brinckerhoff. [1] His background in alternative designs came from research conducted obtaining a master's degree under the direction of Joseph Hummer, then at North Carolina State University. The report, titled *Unconventional Arterial Intersection Design, Management and Operations Strategies*, covers eight at-grade geometries and five grade-separated solutions, as well as focused discussion on the theoretical benefits to conflict points and signal phasing, and the hurdles needed to be overcome to implement more of them in the field.

Following the publication of Reid's report, FHWA funded a larger effort to put together the first federal guide on these designs, the Alternative Intersections/Interchanges: Informational Report. [4] This document, released in 2010, remains the most comprehensive document on the topic to date. Going into far more detail, the AIIR document focuses on the most promising of the current designs being considered, and includes the displaced left-turn, median u-turn, restricted crossing u-turn, and quadrant roadway intersection designs, as well as the diverging diamond and displaced left-turn interchange designs. The document also provides a detailed multi-objective assessment methodology, which relies on operational analysis from a planning level tool, the analysis of which takes up much of the following document. Additionally, this document marks the transition in terminology between "unconventional" and "alternative" for

these designs, as the FHWA works to revise the perceptions of practitioners and make these designs more palatable both in terms of design and psychology.

Though the AIIR and Reid documents represent the first large-format reports on these designs, the first publication in the broad literature can be traced to 1998, with an ITE Journal publication by Joseph Hummer. [10] The *Unconventional Left-turn Alternatives for Urban and Suburban Arterials* article provides traffic-flow line diagrams, descriptions, and general advantages and disadvantages for the median u-turn, restricted crossing u-turn, and bowtie intersection alternatives, without providing sample operational analysis results.

2.3 Motivation for Seeking Intersection Alternatives

The groundwork that led to the advent of alternative intersections came well in advance of their adoption within the research community. Working through the ITE Journal and related conferences, a practitioner working on the problem of maintaining arterial flow published two papers on the underlying arguments for the adoption of alternative designs in the late 1980s. With the '86 publication of *The A to Z Techniques to Reduce Congestion and Increase Capacity of Streets and Intersections* provides the arguments for maintaining arterial flow, and the subsequent detailed geometric design of the restricted crossing u-turn being presented in '87, Mr. Kramer is sometimes cited as the progenitor of alternative intersections. [3], [11]

A second, though less influential background source for the arguments behind alternative intersection functionality comes from a mid-90s publication in the journal *Places*, on the operational benefits of boulevard street configurations. [12] Simply titled *Another Look at Boulevards*, the document provides additional literature support to those designs requiring significant median spaces, such as the median u-turn and restricted crossing u-turn designs.

2.4 Intersection Geometrics

The at-grade designs considered in subsequent chapters include the conventional signalized intersection, roundabout, median u-turn, restricted crossing u-turn, displaced left-turn, jughandle, and quadrant roadway.

2.4.1 Standard Intersections

Intersection geometries which may be considered standard or typical include unsignalized and signalized conventional intersections, as well as roundabouts. The inclusion of the roundabout geometry with this group is a function of its broad adoption domestically by both researchers and practitioners over the course of the previous twenty years.

2.4.1.1 Conventional Intersections

Unsignalized intersections include the junction of three or more approaches at which there may be stop-control, yield-control, or no control whatsoever. The signalization of a junction is carefully controlled, often justified by the signal warrants from the MUTCD, and must the recommendation must be accompanied by an engineering report that includes a safety assessment of the location. [7] The design of the junction in practice comes from a combination

of guidance provided by the Green Book, MUTCD, and the required geometry of the turning radius of the design vehicle chosen by the engineer. There is quite a bit of guidance in the literature on the design of conventional intersections which is outside the purview of this document. Two studies potentially of note, given the requirements of alternative geometric designs, involve guidance on the design of auxiliary lanes at intersections. [13], [14]

2.4.1.2 Modern Roundabout

The modern roundabout design may be the most often asked about topic when lay people are introduced to traffic engineers. Though the design has been relatively slow to catch on domestically, it has long been a popular design choice in Europe. The origin of the modern roundabout design and its transition from the older traffic circle format is well documented. [15], [16] The late 1990s provided the surge in research publications for the modern roundabout design domestically, which subsequently led to the boom in its construction by practitioners. [17], [18]

With geometric design and signage standards becoming more uniform across the country, operational analysis of these designs has been shifting, as the driving public becomes more used to them and adjusts its own behavior. [19], [20] Many communities have experienced first-hand how initial public rejection of the design option has become widespread embrace of its benefits. [21] As with other conventional designs, many researchers have moved past understanding its operational parameters, and are now exploring the design with a multi-modal perspective, including pedestrian functionality and environmental impacts. [22], [23]

2.4.2 Alternative Intersections

Signalized intersections have been on our surface transportation network since the 1800s, but alternatives to the conventional signalized intersection have been slow to develop, and even slower to be embraced. The recent widespread adoption of roundabouts as a viable intersection alternative within the United States suggests that there may be support within the transportation engineering community to embrace a wider selection of options in intersection design.

Alternative intersections are a class of intersections which is generally designed to increase capacity and safety by reducing and separating the number of theoretical vehicle-path conflict points. By rerouting vehicles through alternate paths, these designs are able to reduce the number of signal phases at an intersection to increase arterial throughput. A corresponding negative impact is experienced as additional travel time for the rerouted vehicles. The most comprehensive report to date on the topic of alternative intersections is Federal Highway's Alternative Intersections/Interchanges Informational Report (AIIR) [4].

2.4.2.1 Jughandle

The jughandle intersection design has been popular as an arterial intersection solution within the state of New Jersey since the 1960s. It features the right-hand exit of left-turning traffic, with facilities provided for this traffic movement to subsequently join the traffic flow on the minor approaches. The FHWA Techbrief on the design provides an effective summary of its operation,

including diagrams of three types of typical installations as specified by the New Jersey Department of Transportation (NJDOT). [24] Beyond its inclusion in comparative analysis papers, there is not a great deal of additional literature on the design. However, because of the length of time these designs have been implemented, safety data is more accessible for them than for other alternative designs. [25] A sample jughandle implementation produced by the author is provided in **FIGURE 2-1**.

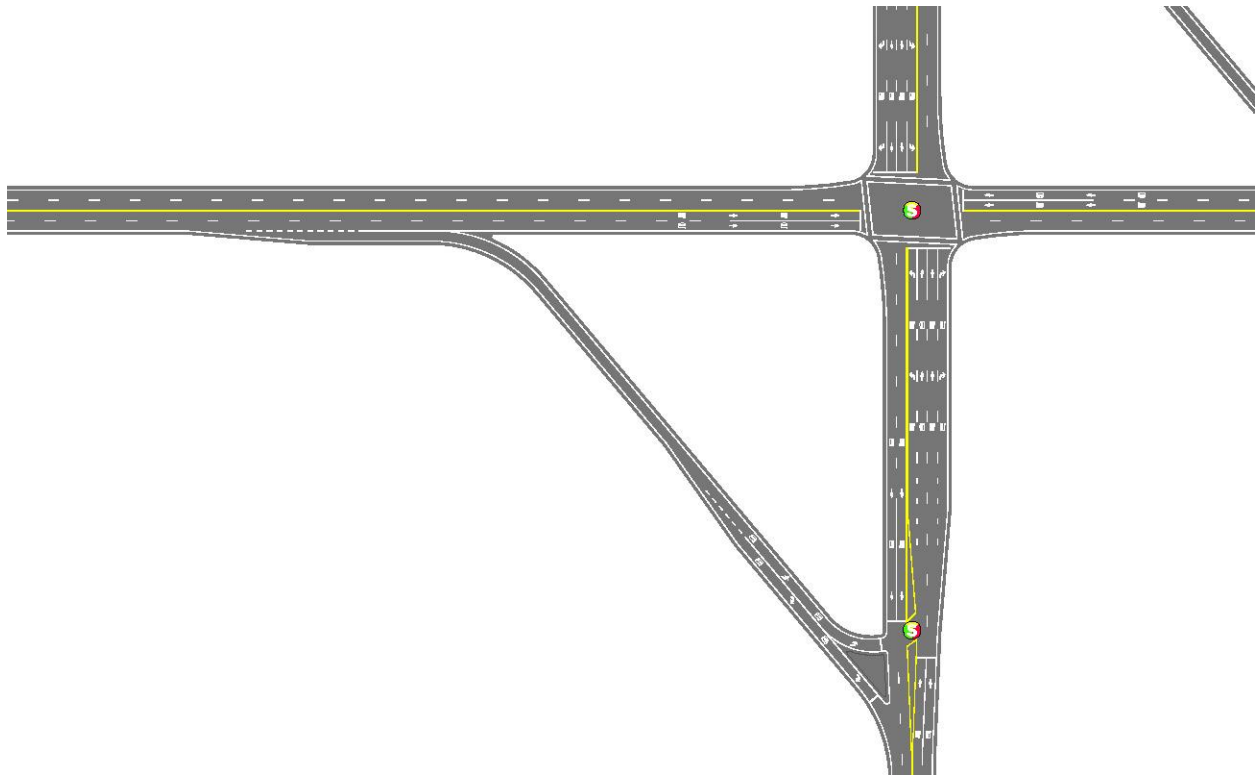


FIGURE 2-1 Sample jughandle geometry

2.4.2.2 Median U-turn

The median u-turn, similar to the jughandle design, was a popular local solution for arterial traffic flow in the 1960s, in this case in the suburbs of Detroit, Michigan. Compared to the jughandle design, the median u-turn has been written about far more extensively in the literature, including two FHWA Techbrief documents. [26], [27] This design functions along boulevard streets with wide medians, by prohibiting left-turn movements at the main junction, and forcing left-turning vehicles to proceed through the intersection to a downstream u-turn location. A sample median u-turn geometry produced by the author is provided in **FIGURE 2-2**.

The operational effects of the median u-turn, in comparison to its equivalent conventional alternative, have been repeatedly reported upon, at least in terms of sensitivity analysis. [28]–[33] From a design perspective, a number of different aspects have been examined including: the overall design [34]; the application of loons at directional crossovers [35]; the optimal

location of the of median u-turn opening [36]; and guidelines on the design of raised medians [37].

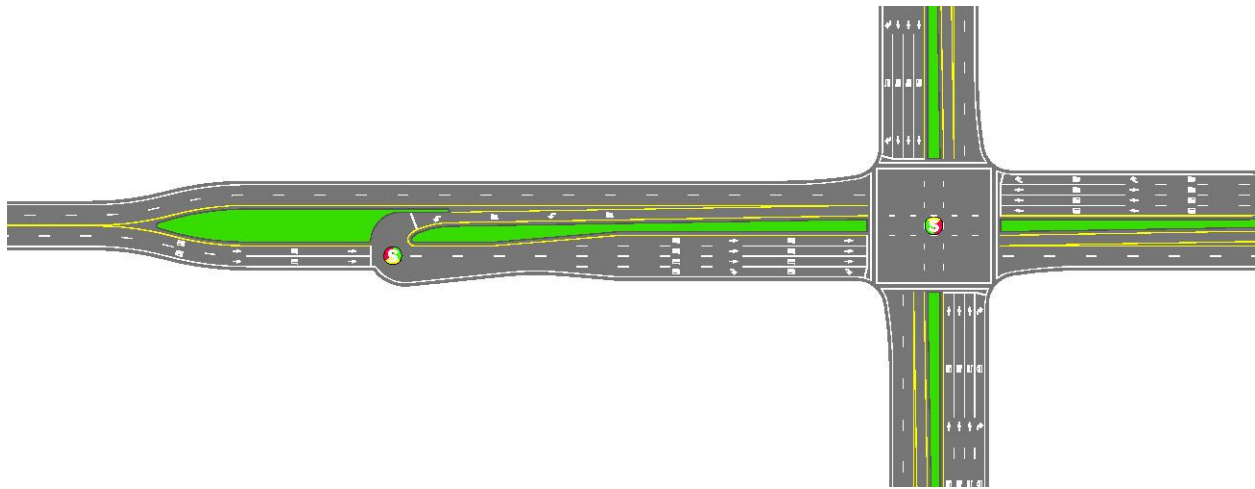


FIGURE 2-2 Sample median u-turn geometry

2.4.2.3 Restricted Crossing U-turn

The restricted crossing u-turn (sometimes called the j-turn design, or the superstreet) is a more-recent addition to the alternative intersection family, first proposed by Kramer in the late-80s. [3] As with the jughandle and median u-turn designs, the FHWA Techbrief on the restricted crossing u-turn serves as an effective and concise summary of its functionality. [38] Similar in configuration to the median u-turn, this design permits the main approach left-turning movements, but instead prohibits all left-turn and through movements from the minor approaches, forcing all vehicles from the minor approach to make a right-turn onto the major roadway and make use of the downstream u-turn bay, as needed. A sample restricted crossing u-turn geometry produced by the author is provided in **FIGURE 2-3**.

With the North Carolina Department of Transportation (NCDOT) serving as a primary advocate for the implementation of this design, a number of arterials have been constructed using this control method in recent years. Along with the implementation of the design has been a number of studies regarding both their operation, as well as lessons learned in their implementation. [39]–[44] The research on this design has been more practice-oriented, and perhaps more practical, than predecessor, the median u-turn intersection.

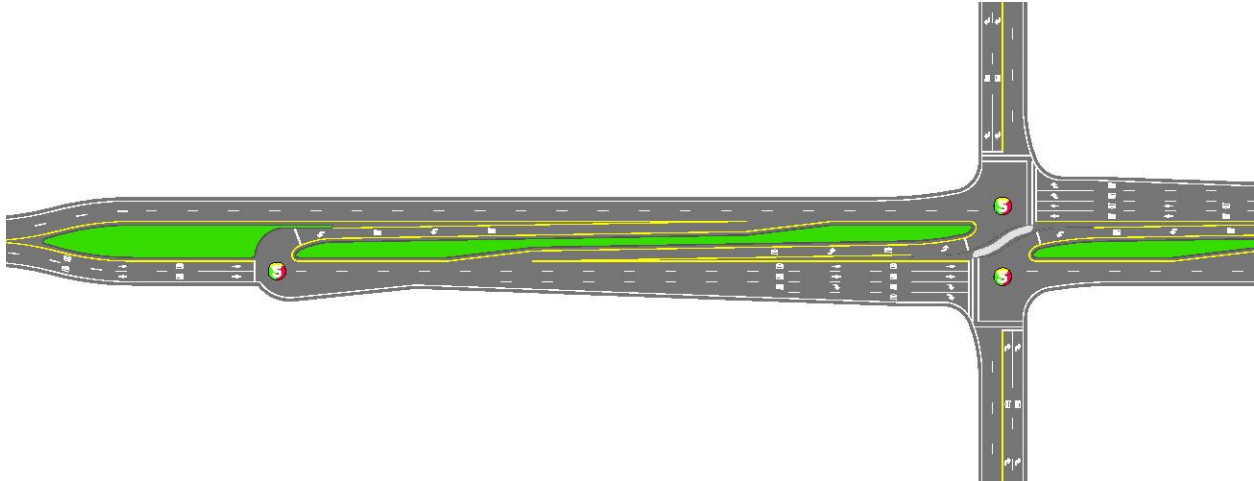


FIGURE 2-3 Sample restricted crossing u-turn geometry

2.4.2.4 Displaced Left-turn

Another family of intersection designs are related to the displaced left-turn (DLT), with variations referred to as the continuous flow intersection (CFI) the parallel flow intersection, and the crossover displaced left-turn intersection. This family of designs has been attributed to Francisco Mier, who patented the design in 1991. [45] A succinct and effective summary is provided again by an FHWA Techbrief. [46] Effectively, this design crosses the left-turning traffic to the other side of the opposing through traffic at some location upstream to the main intersection location; allowing both the through movement and the left-turning movement to proceed concurrently from both sides of an arterial. This design is particularly effective when a high demand volume of left-turning vehicles is present, such that the demand for through and left-turning vehicles is approximately the same in terms of vehicles per lane per hour. The literature is fairly extensive in analyzing the DLT/CFI family of designs, including a few overview articles [47]–[50], a number of operational analysis articles [51]–[56], and even a few articles exploring the traffic control design including signals, signage, and pavement markings at these locations [57]–[59]. Additional measures of effectiveness have been investigated for this family of designs [60], with particular attention paid to pedestrian accommodations [61], [62], as well as safety and environmental perspectives [63]. A sample displaced left-turn geometry produced by the author is provided in **FIGURE 2-4**.

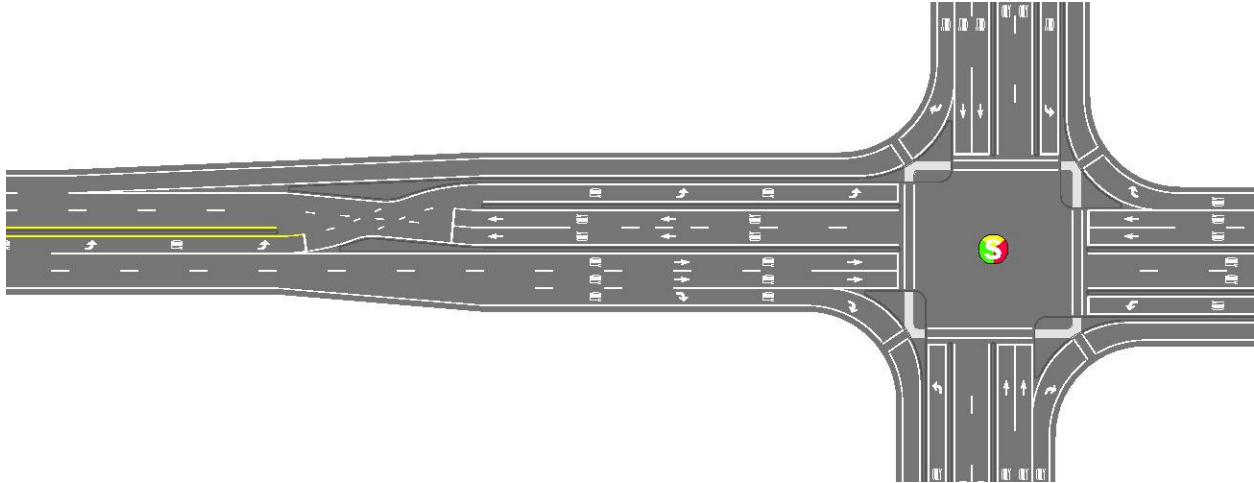


FIGURE 2-4 Sample displaced left-turn geometry

2.4.2.5 Quadrant Roadway

First published in 2000 [64], the Quadrant Roadway intersection design developed by Jonathan Reid achieves the goals of an alternative intersection: increasing safety and arterial throughput by separating conflict points and reducing the number of signal phases. A unique aspect of the quadrant roadway design is that the three signalized intersections operate with a single controller, with each of the three signals sharing a three-phase split phasing plan. A sample quadrant roadway geometry produced by the author is provided in **FIGURE 2-5**.

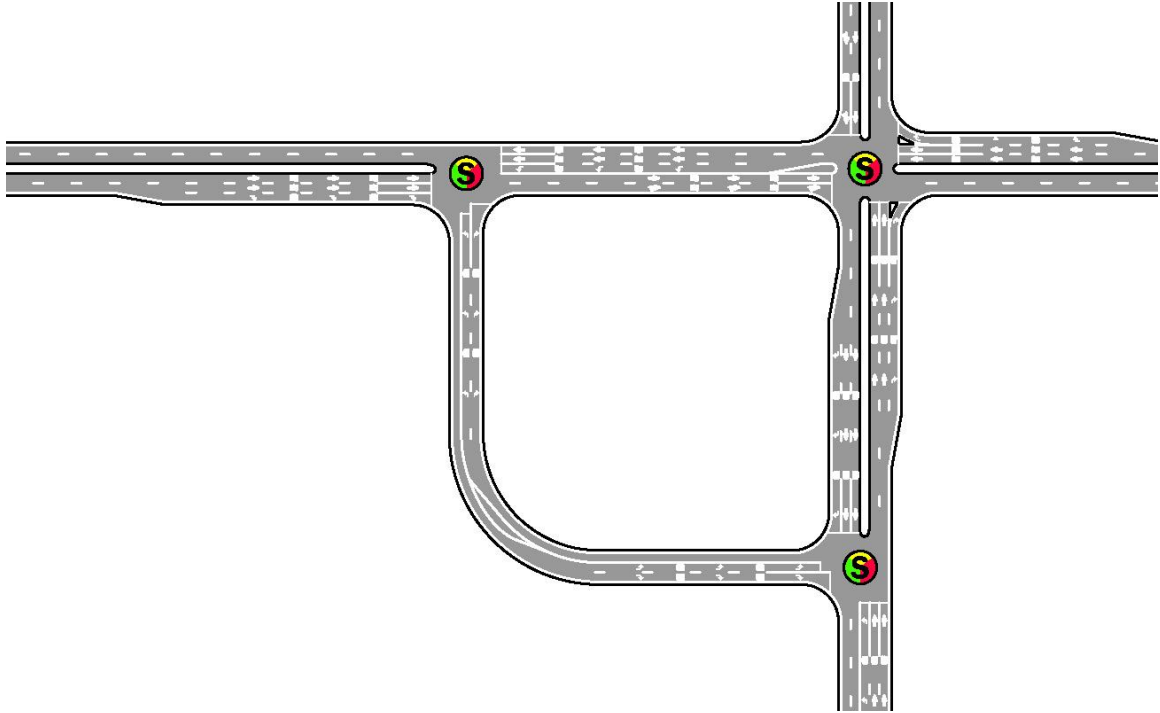


FIGURE 2-5 Sample quadrant roadway geometry

2.4.2.6 Other Roadway Designs

A few other designs in the alternative intersection category have garnered some attention. The split intersection serves as a prelude to a grade separated junction, and has been used with limited success to gain more throughput in the short-term [65]–[67]. On a smaller scale, the double-wide design provides additional throughput for an arterial facing a flow constraint at a signalized intersection [68]. For junctions with a t-intersection, the continuous green-t intersection has shown promise, and been implemented in a number of locations, including the greater Miami area in Florida [69]–[71].

2.5 Interchange Geometries

Subsequent chapters are focused primarily on at-grade intersection alternatives, although the diverging diamond intersection plays a role in multiple chapters.

2.5.1 Conventional Interchanges

There are a number of standard interchange designs used for grade-separated conditions, depending on the scale of the traffic being processed. The most common grade-separated design is the conventional diamond interchange, which allows for uninterrupted flow on the limited-access roadway, with off-ramps upstream of the crossover and on-ramps downstream of the crossover, creating the diamond shape for which it is named. In cases where large volumes of left-turns moving from the highway onto the side streets are present, a single point urban interchange may be selected, which allows the junction to operate as one large three-phase signalized intersection, avoiding the presence of queued vehicles on/under the bridge. Larger volumes may dictate the use of a partial- or full-cloverleaf design, allowing uninterrupted movements on some or all of the turning movements at the junction. For the highest flowrate conditions when two or more highways meet, a directional interchange is used that provides high-speed and high-volume transitions from one roadway to another.

2.5.2 Alternative Interchanges

When successful, new interchange geometries have come from a desire to overcome a set of specific constraints or high levels of conflict between demand volumes. To date the only interchange geometry that has been embraced is the diverging diamond interchange, sometimes called the double crossover diamond, though a few other designs have been proposed.

2.5.2.1 Double Crossover Diamond Interchange

Originally proposed in 2003, the diverging diamond interchange represents the newest and most successful alternative geometry. [67], [72] An overview is provided yet again through an FHWA Techbrief. [2] Essentially, this design crosses traffic over on either side of an interchange overpass, with traffic in both directions moving along the left-hand side of the road over/under the bridge, to be brought back to the right side of the road at the downstream signal. The benefit of this configuration is that left-turning vehicles are then allowed to move without interruption, removing the need for auxiliary turn lanes across the over/underpass. A sample diverging diamond roadway geometry produced by the author is provided in **FIGURE 2-6**.

With the first of these designs being installed in 2009 in Springfield, Missouri, many subsequent locations have been installed in a short time. A great deal of interest has been placed on these designs both within the research and practitioner communities. However, due to the lack of time that has passed since this design was embraced, the majority of the literature is on the operational analysis of the geometry, with little yet in the way of multi-objective analysis. [73]–[78]

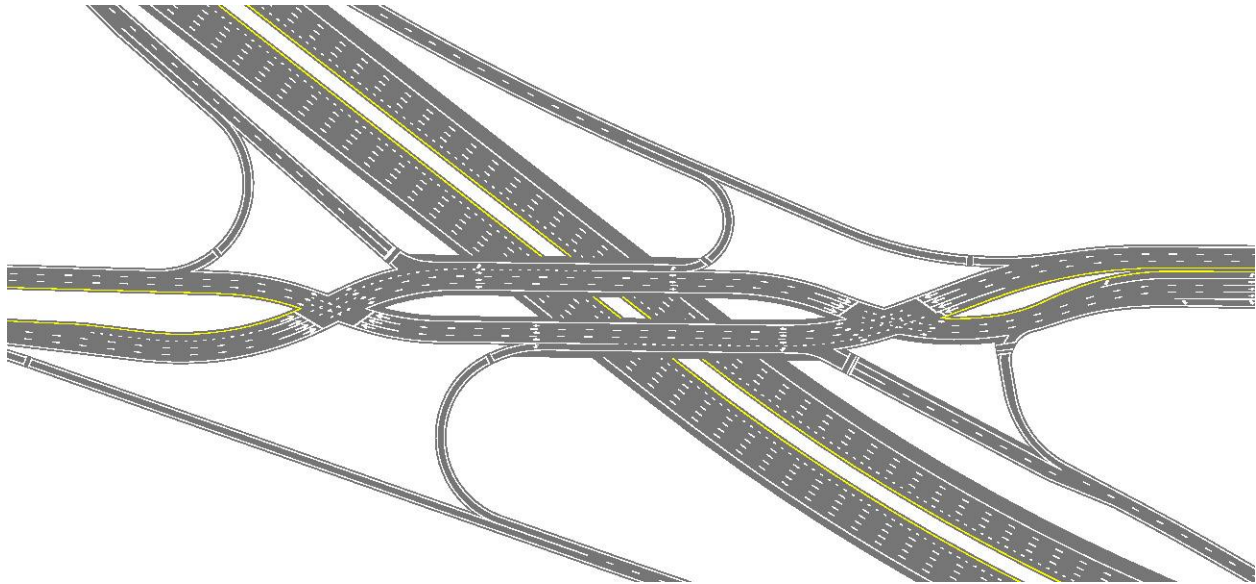


FIGURE 2-6 Sample diverging diamond interchange geometry

2.5.2.2 Other Interchange Designs

A number of other grade-separated designs have been proposed that fall into the alternative geometry category. The displaced left-turn design has been proposed for use in a grade-separated condition, but to the author’s knowledge this design has not been implemented. The required span on the bridge to accommodate all of the auxiliary lanes along with the required median space between opposing directions of traffic would become cost prohibitive. A double-roundabout design, sometimes in a tear-drop shape with the u-turn capability removed, has been proposed and implemented in a small number of cases [76]. An iteration of this design called the W-interchange includes right-only movements onto and off of ramps, providing u-turn bays downstream similar to a median u-turn design [79]. Addressing the issue of access for managed lanes (HOV/HOT) on the interior of a freeway, the dual-system urban interchange design introduces a network of ramps, bridges, and signals that allow access to the freeway simultaneously to either the managed lanes in the middle or the regular lanes on the right side [80]. Lastly, the folded interchange design was proposed as a potential solution for reconstruction of cloverleaf interchanges in failure, as a cost-effective alternative to a directional interchange [81].

2.6 Comparative Analysis

At the heart of any decision to select a particular geometry for a proposed facility is the understanding that the chosen design will work better than the other options. To recommend a new design, with the potential liability that comes along with that decision, engineers must have very high confidence in the operational performance of a given design. To this end, much of the research conducted on new or proposed intersection and interchange designs is of the comparative analysis nature.

The mindset of comparative analysis in research is significantly different from the process conducted in practice for design. The practitioner is given a specific site condition, and a specific set of demand volumes which they are trying to accommodate. The engineer then can apply any design type desired to the given problem, growing each design with additional through and auxiliary lanes until the desired level of service is met for the location. Given a set of equally performing design alternatives, the practitioner may then choose which design to move forward with based on a separate set of parameters, such as cost, site constraints, safety considerations, etc. In contrast, the researcher is working with a featureless non-site, and starts by defining a set of “equivalent” geometries, such that each design has the same number of approach lanes, costs approximately the same, etc. The researcher then selects a variety of volume combinations which may identify patterns in the performance of a given design, and applies each volume scenario to each equivalent geometry, identifying patterns of failure and success to determine the relative performance of each design. The researcher may identify the “best” design as the one which exhibited the lowest average delay for the most volume scenarios, while the practitioner works with a set of designs that are all successful for the given problem at hand, and identifies a “winner” based on a number of varied criteria, often strongly influenced by the wishes of the client.

There are some national standards published on guidelines for conducting comparative analysis of intersection geometries, and these guidelines appear to have evolved from the early comparative analysis research on unconventional intersections conducted by Joseph Hummer and his research group [82], [83].

2.6.1 Operational Analysis

When analyzing the operations of traffic on a road network, two common methods of understanding the functionality of a facility are to examine the delay experienced by drivers at the location, or the capacity of the roadway in comparison to the corresponding demand experienced.

Delay is the standard by which specific intersections are typically deemed to be adequate or in failure. Usually provided in units of seconds per vehicle, it measures the amount of additional time that a given vehicle will take to travel from point A to point B, in comparison to if there had been no traffic control at a location, and no other vehicles on the road. Calculating the average delay manually is a fairly rigorous process, and it is standard practice to rely on software

packages to generate these values, based on equations generated from research and incorporated into the HCM.

As an alternative measure of operations, capacity is related to delay but instead refers to the theoretical volume of traffic that can pass along a roadway, or through an intersection, during a given period of time. Capacity is usually presented in units of vehicles per hour, or vehicles per lane per hour. In contrast to calculating average delay, calculations of capacity at an intersection can be completed quickly without the aid of software, providing a planning-level analysis of design sufficiency.

2.6.1.1 Capacity-based Analysis

In the early stages of engineering design, preliminary engineering must generate a rough assessment of which geometries will solve a given set of site and demand volume constraints. The culture of practitioners is saturated with budgets and deadlines, and there is neither the time nor the desire to fully assess the potential of every design to solve an engineering problem. The standard method for conducting preliminary engineering for intersections and interchanges is to examine measures of the volume-to-capacity ratio for a given solution.

The development of capacity-based analysis came from a wish to explain ideas of signal control and intersection design to lay people [84],[85]. Work has been done to enhance the critical lane volume analysis [86], but the heart of it remains as the underpinning theory behind the Highway Capacity Manual method, and also serves as the current state-of-the-practice basis for analyzing alternative intersections and interchanges as well. With the release of the AIIR, FHWA also developed an Alternative Intersection Selection Tool [87]. This spreadsheet-based tool applies an automated critical lane analysis procedure to numerous conventional and alternative intersection designs, and served as the precursor to the subsequent updated tool produced by FHWA, Capacity Analysis for the Planning of Junctions (CAP-X.)

CAP-X utilizes the critical sum method [84] which calculates capacity at an intersection location by examining the combination of critical movements at the facility. The idea behind the calculation is that while a traffic signal services every turning movement demand at an intersection, only some of those demands are governing the signal timing, while the others just happen to occur simultaneously [84]. For example, on a commuter route with a high volume of traffic during the morning peak in the eastbound direction and a low volume in the westbound, the light remains green in both directions while the eastbound direction exhibits constant flow and the westbound direction only has a trickle of vehicles after the initial queue dispersal. It would not be possible to increase the capacity at the intersection with additional lanes on the approach with a low volume, because the high-volume approach is governing at the time. The critical sum method determines which movement is governing at any given time during a signal cycle, and sums the demands for each sequential critical movement, determining a total value for the demand at the intersection, in units of vehicles per hour per lane (veh/ln-hr).

The CAP-X software assesses six different intersection designs and five different interchange designs. The intersection designs included are the conventional signalized intersection, the

quadrant roadway design, the displaced left-turn design, the restricted-crossing u-turn design, the median u-turn design, and the roundabout. The interchange designs include the traditional diamond interchange, the partial cloverleaf interchange (type A4,) the displaced left-turn interchange, the double-crossover diamond interchange, and the single point urban interchange. Because the software allows for differentiation between the primary alignments of the various designs (east-west or north-south), in all there are twenty different intersection options and ten different interchange options.

The first step in conducting analysis with the CAP-X tool is to enter in the origin-destination demand volumes into the “Input Worksheet.” These take the form of vehicle/hour turning movement counts, with additional information provided including the percentage of trucks for each of the four directional approaches, the adjustment factors for saturated flow of u-turns, left-turns, and right-turns. The recommended default values for these parameters are 2% trucks, with adjustment factors of 0.80 for u-turns, 0.95 for left-turns, and 0.85 for right turns. The user may also choose to enter a failure-level value for the Critical Sum, with a default value of 1,600 veh/ln-hr. Comparative analysis within CAP-X relies on an assumption that all intersections reach failure at the same critical sum value, a simplification which may not hold true, given that a main feature of alternative intersections is the reduction of phases and associated loss-time.

With the turning movement (O-D) values entered, along with the truck percentages and saturation flow rate modifications, the next step is to visit each of the alternative designs in turn, which is computed on its own tab within the spreadsheet. The authors have observed that this software works well from two different approaches: either a practitioner may choose to enter an equivalent number of approach/auxiliary lanes for each design and perform comparative analysis across “equal” options, or they may instead choose to increase the size of each design until a desired level of service is reached, comparing the relative costs of “unequal” designs that perform equally well.

With the number of lanes for each turning movement group identified, the input portion of the CAP-X software application is complete, and the next step is to go to the “Results Worksheet,” where measures of relative demand/capacity are provided in a color-coded summary fashion. These results are based on capacity, and are intended to be predictive only of which intersections will or will not be in failure, not of which intersection will out-perform the others in terms of average intersection delay. Additionally, these values are representative only of the most critical location at an intersection or interchange, and are not indicating an average condition across the facility. Recognizing which intersection design options provide promising results, additional analysis can be performed on a subset of the total designs, resulting in cost-effective alternatives analysis.

2.6.1.2 Delay-based Analysis

While capacity is used for preliminary engineering, the average delay per vehicle traversing a facility is the gold standard for operational analysis, primarily because it serves as the basis for the level of service determinations made by the Highway Capacity Manual (HCM). The HCM defines level of service (LOS) as “a quantitative stratification of a performance measure or

measures that represent quality of service.” [5] There are six grades defined for LOS, ranging from A to F, which are stratified based on a variety of service measures. The grading scheme is intended to represent travelers’ perceptions, and simplify decision making regarding potential future changes to a roadway facility. While the LOS measure has had many critics over the years, it continues to see near-universal adoption among practicing engineers, largely for its ability to describe facility performance to nontechnical decision makers. LOS is reported separately for each mode of travel using a given facility. The analysis is concerned primarily with automobile LOS at an intersection or interchange facility, which are based on the average delay experienced per vehicle as they traverse the location.

Control delay replaced stop delay as the primary service measure for automobile analysis of intersections in the 1994 update to the 1985 HCM, and is anticipated to remain the primary service measure for all junction types beyond the current 2010 edition. Control delay is defined as “the delay brought about by the presence of a traffic control device,” and includes the slowing of vehicles in advance of an intersection, the stopped time at the intersection approach, the time required to progress in the queue, and the time required to accelerate back to the desired speed [5]. Notably, the control delay excludes delay caused by geometric features, whether from vehicles slowing to navigate a sharp turn or an indirect route that vehicles must take through the junction.

The use of control delay without the inclusion of geometric components poses several problems for the analysis of alternative intersections and interchanges, many of which feature the rerouting of vehicles that causes additional travel time. There are a number of publications recommending the appropriate procedures to conduct comparative analysis of alternative designs, but universally these procedures rely on travel-time measures [88], [89]. The author works to address this issue in a subsequent chapter of the dissertation document. There are many comparative analysis publications on the topic of alternative intersections using travel time measures from simulation analysis [90]–[99]. While the earliest of these was innovative in its approach, the majority of these studies offer little in the way of additional insight into either the analysis of these designs, or the situations in which one would choose one over another.

2.6.2 Multi-Objective Analysis

While the research community has been primarily focused on developing a fully operational understanding of these designs, a number of studies in the literature have endeavored to move beyond the primary service measures of the design and consider the secondary impacts. In a few cases, researchers have developed and proposed a “program” for evaluating these designs, though in the opinion of the author these generally boil down to checking for operational performance and then looking at site constraints and cost to make a decision [100],[101]. The list of secondary considerations is generally short, and includes site constraints, safety considerations, preferential treatment for some turn-movements over others, and cost.

Safety analysis of alternative designs, and subsequent improvements to the geometric, signage, and pavement markings for these designs, is perhaps the area in need of the greatest

growth once operational analysis of them is fully understood. A number of studies have previously worked to understand safety at intersections [102]–[104], and along arterials [105], [106], which may provide the background necessary for tackling the specific issues encountered at alternative designs. Perhaps the greatest concern with implementing an alternative intersection design is that of “driver expectancy,” that a driver unused to the traffic flow pattern of the newly encountered design may behave in an unsafe manner. A great deal of work was done with the diverging diamond design to address concerns with driver expectancy prior to its installation in 2009, and this work sets a path for future studies on this topic [2]. Some overall safety concern studies have been done looking at alternative intersections as a whole and at safely implementing these designs [107], [108]. Safety studies on specific geometries remain rare, but have been attempted on median crossovers, as well as jughandle designs [25], [109].

Due to the large foot-prints needed by some of these designs, such as the medians required for the restricted crossing and median u-turn designs, site constraints are a primary cause of disqualifying a given design in an engineering study, however, perhaps because of the limited need to explain this in more detail, there’s also a limited amount of research specifically targeted at understanding the site constraints of these geometries [110].

The analysis of multi-modal service is trending among both researchers and practitioners, and a fair amount of interest has been raised in the accommodation for bicycles and pedestrians on the roadway network. In addition to the accommodation of pedestrians at roundabouts, research in the alternative intersection realm has included the analysis of pedestrians at continuous flow intersections, which often lack sufficient clearance times for pedestrian crossings [22], [61], [62].

Returning briefly to the topic of operational analysis, one type of secondary consideration for these designs is the prioritization of one movement over another. The primary use of the restricted crossing u-turn design within North Carolina has been as an access measure along what the state has defined as “Strategic Transportation Corridors,” where throughput is prioritized over the service provided to adjacent developments, while still providing access to the roadway network to those developments. Other designs in the alternative intersection realm provide benefits to specific turn-movements over others, and may serve as effective solutions where prioritization can be identified.

Ultimately, the decision to pursue one of these designs may come down to cost. In many cases, these alternative intersection designs feature additional pavement, signalization, signage, and potentially liability, and are likely to be discounted relative to the conventional alternative due to cost constraints. However, in cases where the conventional solution is not able to address the projected traffic demands, many of the at-grade alternative intersection solutions will be able to address the problem at a fraction of the cost associated with the construction of a conventional grade-separated interchange. When faced with trying something new that costs \$3 million, or using the traditional solution that costs \$10 million, decisions makers may find themselves thinking outside of the box.

Chapter 3 Operational Measures of Effectiveness for Alternative Intersections

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Abstract

As the variety of geometric designs of junctions increases, we are presented with an opportunity to re-think both the way in which we understand the delay caused by these designs, and the way in which service provided by these designs is perceived. The pending inclusion of the diverging diamond interchange, the displaced left turn intersection, the median u-turn, and restricted crossing u-turn intersections into the canon of the Highway Capacity Manual creates analysis problems not previously encountered. Some of these designs reroute vehicles through multiple signalized intersections which are part of a single junction configuration, imparting not just multiple points of conflict and delay, but also additional travel time.

The purpose of this paper is to explore the concepts of delay and level of service using field data from alternative intersections and interchanges. The authors propose a new approach for junction delay, a service metric that can be universally applied to all intersection and interchange types. Subsequently, the authors revisit the relationship between level of service and delay, proposing a new approach that scales the grade-regimes by peak-hour demand volume uniformly for all control types.

3.1 Background

3.1.1 Alternative Intersection and Interchange Geometries

Designs that fall into the category of alternative intersections and interchanges work by rerouting some of the turning movements made at the junction, often reducing and separating conflict points, with theoretical benefits to both safety and mobility. Conflict points represent each position where the paths of two vehicles cross, with potential collision locations identified as crossing, merging, or diverging conflicts. Any reduction in the number of potential conflicts, especially of the most dangerous crossing type, provides proportional benefit to intersection safety. This reduction in conflict points at these designs often comes with a reduction in the number of signal phases at the junction, providing a reduction in delay to through movements on the major arterial, while often increasing travel time and travel distance for secondary movements. Depending on the degree of benefit to the through movement on the major arterial and the degree of degradation to the secondary movements, the overall average delay may be more or less than that experienced by an equivalent conventional signalized intersection.

The most comprehensive resource at the moment is the Alternative Intersections / Interchanges Informational Report (AIIR). [4] The primary focus of this document covers the displaced left-turn, median u-turn, restricted crossing u-turn, and quadrant roadway intersection designs, as well as the diverging diamond (also known as the double crossover diamond) interchange. This report provides detailed information on existing applications, typical geometric design and signal locations, as well as potential signal phasing for each design.

3.1.2 Delay and Level of Service

The Highway Capacity Manual defines level of service (LOS) as “a quantitative stratification of a performance measure or measures that represent quality of service.” [5] There are six grades defined for LOS, ranging from A to F, which are stratified based on a variety of service measures. The grading scheme is intended to represent travelers’ perceptions, and simplify decision making regarding potential future changes to a roadway facility. While the LOS measure has had many critics over the years, it continues to see near-universal adoption among practicing engineers, largely for its ability to describe facility performance to nontechnical decision makers. LOS is reported separately for each mode of travel using a given facility – the analysis is concerned primarily with automobile LOS at an intersection or interchange facility, which are based on the average delay experienced per vehicle as they traverse the location.

Control delay replaced stop delay as the primary service measure for automobile analysis of intersections in the 1994 update to the 1985 HCM, and is anticipated to remain the primary service measure for all junction types beyond the current 2010 edition. Control delay is defined as “the delay brought about by the presence of a traffic control device,” and includes the slowing of vehicles in advance of an intersection, the stopped time at the intersection approach, the time required to progress in the queue, and the time required to accelerate back to the desired speed. [5] Notably, the control delay excludes delay caused by geometric features, whether from

vehicles slowing to navigate a sharp turn or an indirect route that vehicles must take through the junction.

The Highway Capacity and Quality of Service (HCQS) committee believes that “travelers’ expectation of performance varies at different system elements (e.g., unsignalized intersections versus signalized intersections)” but they also acknowledge that “further research is needed to understand fully the variation in traveler perceptions of LOS across facility types.” The 2010 edition of the HCM is the first to incorporate LOS methodologies that are based directly on the results from traveler perceptions of LOS, with the HCQS committee selecting thresholds in cases where research is not available. The threshold between LOS E and F for interrupted-flow facilities is generally set at the change point between undersaturated and oversaturated flow conditions, when the demand-to-capacity ratio reaches one and perpetual queues begin to form. The interpretation within HCM 2010 of how delay translates to LOS for a variety of intersection and interchange facilities is shown in **TABLE 3-1**. Additional information on LOS interpretation for alternative intersections and interchanges will be forthcoming from NCHRP Project 03-115, but this material is not yet available outside of the HCQS committee. [115]

TABLE 3-1 Level of service thresholds for HCM 2010, in terms of control delay (seconds/vehicle)

Level of Service	Stop-Controlled Intersection	Roundabout	Signalized Intersection	Roundabout Interchange	Signalized Interchange
A	≤ 10	$x \leq 10$	$x \leq 10$	$x \leq 15$	$x \leq 15$
B	$10 < x \leq 15$	$10 < x \leq 15$	$10 < x \leq 20$	$15 < x \leq 25$	$15 < x \leq 30$
C	$15 < x \leq 25$	$15 < x \leq 25$	$20 < x \leq 35$	$25 < x \leq 35$	$30 < x \leq 55$
D	$25 < x \leq 35$	$25 < x \leq 35$	$35 < x \leq 55$	$35 < x \leq 50$	$55 < x \leq 85$
E	$35 < x \leq 50$	$35 < x \leq 50$	$55 < x \leq 80$	$50 < x \leq 75$	$85 < x \leq 120$
F	$50 < x$	$50 < x$	$80 < x$	$75 < x$	$120 < x$

3.2 Purpose

The purpose of this paper is to reexamine the current standards for calculating delay at intersections and interchanges, and subsequently to look at how this delay is translated into level of service (LOS). The authors feel that the current approach to delay and LOS have become increasingly convoluted, and confusion will only increase with the introduction of alternative intersection designs. Using field data collected at alternative intersection locations, the authors propose a new measure of junction delay that includes both control delay and geometric delay,

applied to all intersection and interchange geometries equally. Subsequently, the authors propose an LOS regime system that ties the junction delay service measure to LOS grades, differentiated by demand volumes and uniformly applied to every type of control condition.

3.3 Data Collection and Reduction

Video data collected at multiple intersection and interchange locations is used to provide observations of in-situ driver behavior. Travel times for individual vehicles traversing each facility are generated using observations from the video data.

3.3.1 Field Data Collection

Three alternative designs were chosen for conducting this study, including the diverging diamond interchange (DDI), the displaced left-turn intersection (DLT), and the restricted crossing u-turn intersection (RCUT) designs. The DDI selected is located at the junction of Interstate-15 and West Main Street in American Fork, Utah. The DLT selected is located at the junction of Eisenhower Boulevard and Madison Street in Loveland, Colorado. The RCUT selected is located at the intersection of Ocean Highway E. and Ploof Road in Leland, North Carolina. Field data collection was conducted in late May and early June of 2013. Video data using multiple GoPro cameras was collected, covering both AM and PM peak periods with simultaneous feeds covering each direction of traffic. In some cases, such as with the DDI installation, as many as 12 simultaneous video feeds were collected during two hours each peak period. On the following page, **FIGURE 3-1** provides the geometry for each site analyzed, as seen in the aerial images. The observed peak-hour flow rates for the turn movements (origin-destination pairings) at each respective location are provided in **TABLE 3-2**.

TABLE 3-2 Origin-destination traffic demands by location

Location (Design)	Northbound			Southbound			Eastbound			Westbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
I-15 and SR92 (DDI)	924	x	556	464	x	268	120	492	692	368	756	484
Eisenhower and Madison (DLT)	52	184	136	316	188	64	80	1524	40	132	780	340
Ocean Hwy E. and Ploof (RCUT)	52	44	72	192	32	240	228	1296	84	100	1500	228



(a) Junction of Interstate 15 and State Route 92, Lehi, Utah (DDI)



(b) Junction of Eisenhower Boulevard and Madison Avenue, Loveland, Colorado (DLT)



(c) Junction of Ocean Highway East and Ploof Road, Leland, North Carolina (RCUT)

FIGURE 3-1 Aerial location maps for data collection sites

3.3.2 Travel Time and Delay from Field Data

Using a combination of the video input feeds and the roadway geometry, a location along each approach and exit from the junction was chosen as the limits of the network for the sake of travel time calculations. Time was logged for each vehicle as it crossed the threshold entering the network, and subsequently as it crossed the corresponding threshold exiting the network, with the individual travel times (TT_O) then aggregated by turn movement to determine the mean ($TT_{O,M}$) and 5th percentile travel times ($TT_{O,5\%}$) for each origin-destination pairing.

The fifth percentile measure was obtained in the absence of a speed study, to be used in lieu of the 85th percentile speed of free-flowing traffic. However, some movements were observed to have no unimpeded vehicles, so a secondary measure of base travel time (BTT_O) was calculated, using the vehicle path through the network and the associated posted speed limits. The minimum of the two values is used for calculations involving base travel time (BTT_U), as shown in equation 3-1.

$$BTT_U = \min_{o-d}(TT_{O,5\%}, BTT_O) \quad (3-1)$$

Control delay (d_C) is then determined for each origin-destination pairing as the mean observed travel time minus the base travel time, as shown in equation 3-2.

$$d_C = TT_{O,M} - BTT_U \quad (3-2)$$

The overall intersection average vehicle delay is calculated by taking the weighted average of each origin-destination pair, as weighted according to the demand volumes for that movement. This approach for calculating vehicle delay is consistent both with the HCM methodology, and with the delay calculations performed by microsimulation applications. However, it fails to provide an adequate service measure for alternative intersection geometries, many of which impart significant additional travel time to vehicles through rerouted paths.

3.4 Proposed Analysis Methodology

Alternative intersection designs pose a problem for the standard measure of control delay, that is, the amount of time in seconds that a particular vehicle is delayed from completing its travel due to crossing through a junction. Microsimulation calculates the control delay for a vehicle as the cumulative difference between the expected velocity and the experienced velocity as it traverses a network. In the case of alternative intersections, the vehicle may encounter additional travel distance without incurring additional control delay, so long as the desired velocity is met along the stretch. The current recommended practice is to create equal-sized networks and compare the average total travel time per vehicle as the output. The authors propose tying the travel time metric back to the delay metric by defining a base condition independent of geometry, with each origin-destination point is equidistant from the centroid of the intersection. Placing a circle centered at the middle of the junction, the radius would be approximately equal to the minimum distance required to reach the limits of intersection influence on all approaches. In the absence of any intersection control, all twelve origin-destination pairs would then travel the same distance, equal to the diameter of the circle defined. By assigning the desired free flow speed on

each approach to be equal to that of the upstream roadway, a theoretical base travel time can then be readily determined for each o-d pairing, entirely independent of any base roadway condition at the site.

3.4.1 Component Validation Results

The authors denote two theoretical base travel times relative to the radius of influence defined, which are the vehicle-path travel time from the radius (BTT_R), and the proposed node-to-centroid base travel time (BTT_N). The difference between these two values for each given turn movement provides the geometric component of the delay, as provided in equation 3-3.

$$d_G = BTT_N - BTT_R \quad (3-3)$$

One consequence of universally adopting a geometric delay component will be some rather drastic shifts in individual turn-movement delays at interchange facilities. There will be significant increases in left-turn delays, and significant decreases in right-turn delays, often returning negative delay values for right-turn movements. This may raise issues for practitioners working with constraints on individual turn-movement LOS, but it more accurately represents the impact of each design alternative on experienced travel times.

Ideally, the entry and exit points used in reducing the field data would be located at the radius points selected as the theoretical limits of intersection influence. However, as field data collection pre-dated the development of this approach, the camera locations chosen require different entry/exit points to be selected. As a result, an adjustment factor (TT_A) was introduced to the travel time data to convert the observed travel times, setting TT_A equal to the difference between the base travel time for vehicle paths from the radius (BTT_R) and the base travel time observed (BTT_O) for each origin-destination pair. The adjusted individual travel times (TT_R), are equal to the observed travel time (TT_O) plus the appropriate origin-destination pair adjustment (TT_A).

The process to generate the new junction delay measure (d_J) is shown in **FIGURE 3-2** as a flowchart. The boxes with grey backgrounds indicate the additional steps implemented to work with the field data collected, with the grey arrows indicating the change in path to conduct the analysis if simulation data is being used. Substituting the travel times adjusted for the radius entry and exit points (TT_R) for the observed travel times (TT_O) in equation 3-1 and equation 3-2 above will provide the adjusted base travel time (BTT_U) and control delay components (d_C), as shown in equation 3-3 and equation 3-4, below.

$$BTT_U = \min_{o-d}(TT_{R,5\%}, BTT_R) \quad (3-3)$$

$$d_C = TT_{R,M} - BTT_U \quad (3-4)$$

Finally, the junction delay is simply equal to the control delay plus the geometric component of the delay, as shown in equation 3-5.

$$d_J = d_C + d_G \quad (3-5)$$

The aggregate average delay per vehicle for the intersection is calculated by taking the weighted average of each origin-destination pair (turn-movement), with the demand volume in the peak period used for weight.

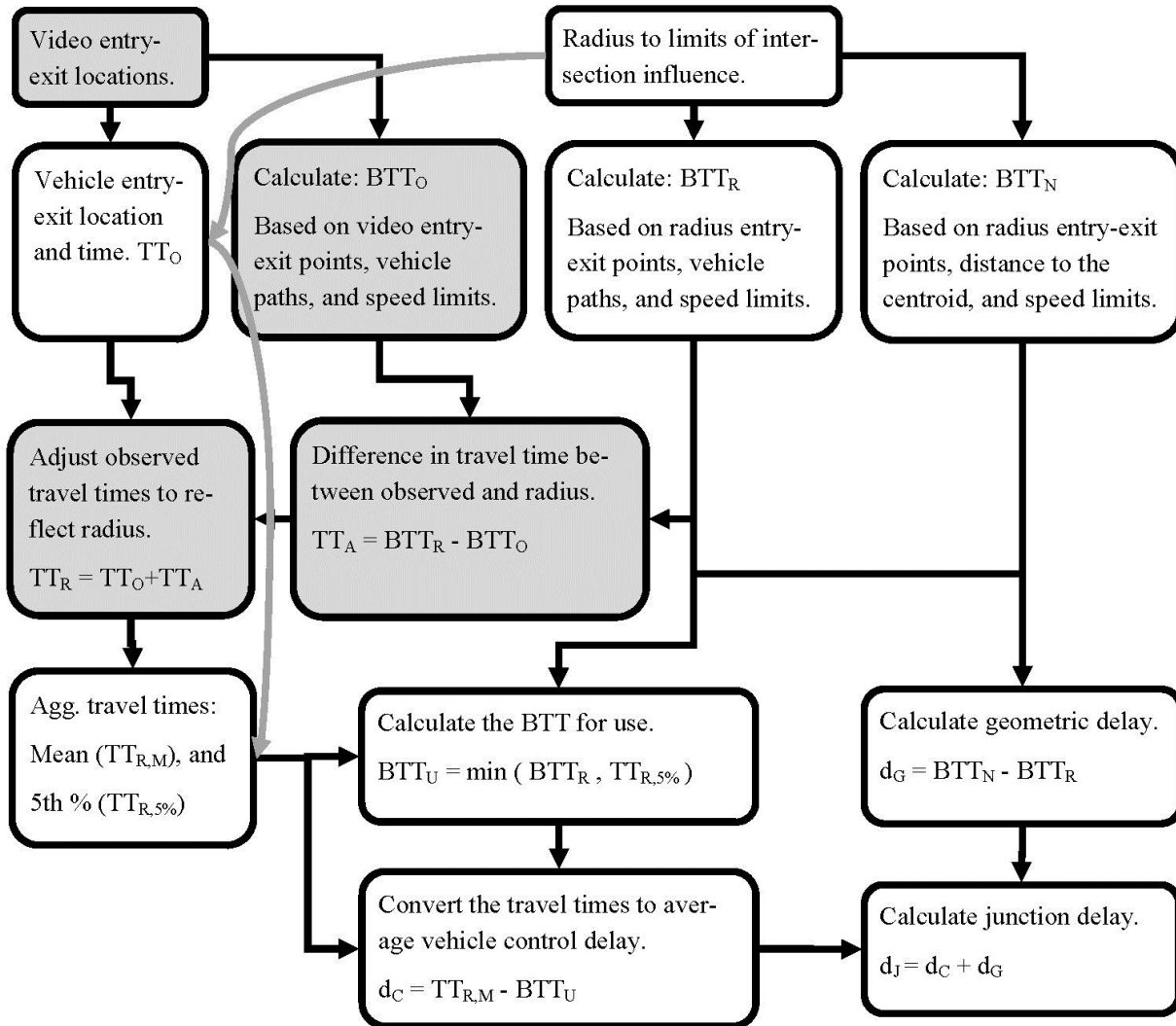


FIGURE 3-2 Flow chart to calculate delay using field data and proposed method

Applying this methodology to the three case study locations yields results as shown in the three part of **TABLE 3-3**, provided below. The overall impact of the geometric delay is low for all three designs, with the DDI losing 3.9 seconds, the DLT losing 0.4 seconds, and the RCUT gaining 0.7 seconds.

TABLE 3-3 Measures of travel time and delay by location and turn movement

(a) I-15 and SR92, Lehi, UT (DDI)

I-15 and SR92 (DDI) Measures of Base Travel Time	Northbound			Southbound			Eastbound			Westbound			Junction Average	
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right		
Vehicle Trajectory - BTT(R)	37.3	Uninterrupted Flow	10.1	38.5	Uninterrupted Flow	10.8	44.0	38.5	32.5	50.0	38.0	34.0		
Observed 5th Percentile - TT(R,5%)	24.5		11.4	34.4		10.9	41.2	34.2	21.3	49.6	27.1	26.6		
Base Travel Time - BTT(U)	24.5		10.1	34.4		10.8	41.2	34.2	21.3	49.6	27.1	26.6		
Node Method - BTT(N)	37.3		37.3	37.3		37.3	37.3	37.3	37.3	37.3	37.3	37.3		
Geometric Delay - d(G)	0.0		-27.2	1.2		-26.5	6.7	1.2	-4.8	12.7	0.7	-3.3		-3.9
Observed Mean - TT(R,M)	60.7		21.8	62.2		18.1	71.7	83.9	24.3	80.8	64.7	29.0		51.0
Control Delay - d(C)	36.2		11.7	27.8		7.3	30.5	49.7	3.0	31.2	37.6	2.4		24.6
Junction Delay - d(J)	36.2		-15.5	29.0		-19.2	37.2	50.9	-1.8	43.9	38.3	-0.9		20.7

(b) Eisenhower and Madison, Loveland, CO (DLT)

Eisenhower and Madison (DLT) Measures of Base Travel Time	Northbound			Southbound			Eastbound			Westbound			Junction Average	
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right		
Vehicle Trajectory - BTT(R)	22.9	25.6	22.5	22.9	25.6	22.5	23.4	22.3	21.3	24.0	22.4	21.0		
Observed 5th Percentile - TT(R,5%)	22.2	22.3	20.7	24.9	27.0	34.8	49.3	11.0	19.5	45.3	31.3	27.6		
Base Travel Time - BTT(U)	22.2	22.3	20.7	22.9	25.6	22.5	23.4	11.0	19.5	24.0	22.4	21.0		
Node Method - BTT(N)	24.0	25.6	24.0	24.0	25.6	24.0	24.0	22.3	24.0	24.0	22.3	24.0		
Geometric Delay - d(G)	-1.1	0.0	-1.5	-1.1	0.0	-1.5	-0.6	0.0	-2.7	0.0	0.1	-3.0		-0.4
Observed Mean - TT(R,M)	67.2	75.2	40.2	86.9	78.0	69.4	98.8	42.5	30.8	84.3	42.5	30.8		49.7
Control Delay - d(C)	45.0	52.9	19.5	64.0	52.4	46.9	75.4	31.5	11.3	60.3	20.1	9.8		28.0
Junction Delay - d(J)	43.9	52.9	18.0	62.9	52.4	45.4	74.8	31.5	8.6	60.3	20.2	6.8		27.6

(c) Ocean Hwy E. and Ploof Rd, Leland, NC (RCUT)

Ocean Hwy E. and Ploof (RCUT) Measures of Base Travel Time	Northbound			Southbound			Eastbound			Westbound			Junction Average	
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right		
Vehicle Trajectory - BTT(R)	43.1	45.8	23.7	42.3	45.4	22.6	24.6	20.4	23.1	16.1	19.4	21.8		
Observed 5th Percentile - TT(R,5%)	78.9	107.6	30.1	95.9	96.9	26.9	28.7	18.5	28.3	31.0	17.0	23.5		
Base Travel Time - BTT(U)	43.1	45.8	23.7	42.3	45.4	22.6	24.6	18.5	23.1	16.1	17.0	21.8		
Node Method - BTT(N)	24.9	29.9	24.9	24.9	29.9	24.9	24.9	19.9	24.9	24.9	19.9	24.9		
Geometric Delay - d(G)	18.2	15.9	-1.2	17.4	15.5	-2.3	-0.3	0.5	-1.8	-8.8	-0.5	-3.1		0.7
Observed Mean - TT(R,M)	126.6	132.0	52.7	131.9	124.3	50.5	59.9	21.9	29.7	71.9	29.1	27.7		39.0
Control Delay - d(C)	83.5	86.2	29.0	89.6	78.9	27.9	35.3	3.4	6.6	55.8	12.1	5.9		18.2
Junction Delay - d(J)	101.7	102.1	27.8	107.0	94.4	25.6	35.0	3.9	4.8	47.0	11.6	2.8		18.9

The negative 3.9 seconds associated with the DDI interchange is due to very large reductions in travel time experienced by the right-turn movements, as the interchange ramp system significantly cuts down on the physical distance these vehicles travel, effectively traversing one side of a triangle instead of two. There are increases to the distance traveled by left-turn movements, with slight increases to through-movements as well, but these increases are more than offset by the advantages provided to the right-turning vehicles. The ultimate junction delay (d_j) measure includes negative values for all of the right-turn movements, with the largest values that of the southbound-right movement at -19.2 seconds, and the northbound-right movement at -15.5 seconds. These would technically fall into the category of LOS A, being less than or equal to 15 seconds of delay, though to the authors knowledge there isn't currently a methodology that predicts or accommodates negative delay measures at a junction.

With a negative 0.4 seconds of geometric delay for the DLT design, the same problems occur as with the DDI, but to a lesser degree. In the case of the DLT, none of the individual turn movements experience a net negative junction delay, with geometric delays between -3 and -1.1 for the left- and right-turn movements. For this design, the intended benefit of crossing over the left-turn movement at an upstream signal is to reduce the amount of loss time per cycle by reducing the number of phases from four to three, with minimal impact on the vehicle path travel times.

The RCUT design experiences some of the largest increases in minor-street travel times due to rerouting of the through and left-turning vehicles from the minor approaches through downstream u-turn bays. The component of delay associated with geometry is estimated to be between a 15.5 second and 18.2 second increase for these movements. However, the change to the average delay per vehicle for the overall junction is only an increase of 0.7 seconds, due to slight advantages in the pathway of the major approach left-turning vehicles, and the relatively small number of vehicles making the left- and through-movements on the minor approaches. Based on the observed control delay, the signal timings had already been set to prioritize the major-movements, with the minor-street left- and through-movements operating in the LOS E range before the introduction of geometric delay, and LOS F range after its inclusion; this in comparison to an overall intersection LOS of B in both cases.

3.5 Proposed Level of Service Methodology

The introduction of alternative intersection designs to the Highway Capacity Manual may further convolute the already complex system of LOS thresholds, as previously provided in **TABLE 3-1**. As an alternative, the authors recommend an LOS relationship with junction delay that is independent of the type of installation, but is instead based on the peak-hour demand volumes. The authors propose a system tied to a metric similar to the MUTCD signal warrant methodology, examining the sum of the demand volumes on the three busiest approaches at a junction. [7] The sum of the two-way major street volume and the greater of the two volumes from the side street is approximately 1,500 vehicles/hour for the peak-hour signal warrant, so the authors have chosen this value as the threshold between the low-volume regime and the medium-

volume regime. A preliminary volume of 4,000 vehicles/hour is chosen for the threshold between the medium- and high-volume regimes, attempting to identify an approximate benchmark at which point grade-separation of vehicles becomes necessary. The resulting updated LOS-to-delay threshold volumes are provided in **TABLE 3-4**.

TABLE 3-4 Proposed LOS threshold relationship with peak hour demand volumes

Level of Service	Peak Hour Demand Volume sum of 3-highest approaches (experiencing interruption)		
	vol. \leq 1,500	1,500 < vol. \leq 4,000	4,000 < vol.
A	≤ 10	$x \leq 10$	$x \leq 15$
B	$10 < x \leq 15$	$10 < x \leq 20$	$15 < x \leq 30$
C	$15 < x \leq 25$	$20 < x \leq 35$	$30 < x \leq 55$
D	$25 < x \leq 35$	$35 < x \leq 55$	$55 < x \leq 85$
E	$35 < x \leq 50$	$55 < x \leq 80$	$85 < x \leq 120$
F	$50 < x$	$80 < x$	$120 < x$

As with much of the work associated with LOS, these delay thresholds will require significant study to work in driver’s perceptions. In spite of a lack of new data regarding driver perceptions provided, the authors wish to provide an argument for this revised system. The existing regime system creates situations where facility operation and LOS are not consistent with the perceptions of the public. Rural interchanges are often provided more for access than for mobility considerations, with high uninterrupted flow on the highway and minimal volumes from the local roadways – these facilities may operate at LOS A for their lifetimes, and a vehicle traversing them would never accept a delay of nearly a minute. In contrast, a large urban signalized intersection with three through lanes and three auxiliary turn lanes on each approach experiences extremely high volumes – it may never operate at LOS D or better, but vehicles traversing it are far more likely to accept longer delays to move past it. The suggestion of the authors is that driver perceptions of delay may be as affected by the driver’s perceptions of the scope of the problem being solved, as they are by the type of junction they are traversing, particularly with the broader embrace of alternative intersection geometries into our surface roadway network.

To investigate the proposed LOS thresholds further, the relative volume demands at the DDI, DLT, and RCUT junctions is revisited in **TABLE 3-5**, with the resulting LOS metric provided for each. Some of the problems that existed with the previous threshold system remain. Though

the demand volumes at the RCUT installation are with 10% of those experienced at the DDI location, the DDI is judged by the high-volume measure and the RCUT is judged by the medium-volume measure. However, had the north- and south-bound through vehicles on the highway been taken into account in the calculations, the DDI design would be well above the threshold chosen.

TABLE 3-5 Origin-destination traffic demands and resulting LOS threshold metric

Location (Design)	Volume Measure	Northbound			Southbound			Eastbound			Westbound			LOS Metric
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
I-15 and SR92 (DDI)	Origin-Destination	924	x	556	464	x	268	120	492	692	368	756	484	4392
	Approach	1480			732			1304			1608			
Eisenhower and Madison (DLT)	Origin-Destination	52	184	136	316	188	64	80	1524	40	132	780	340	3464
	Approach	372			568			1644			1252			
Ocean Hwy E. and Ploof (RCUT)	Origin-Destination	52	44	72	192	32	240	228	1296	84	100	1500	228	3900
	Approach	168			464			1608			1828			

The DDI is found to be in the high-volume threshold for LOS, consistent with the HCM 2010 values for signalized interchanges, such that the average junction delay of 20.7 seconds per vehicle places the observed performance of the facility in the LOS B range in both cases. The DLT and RCUT designs both fall into the medium-volume threshold for the proposed LOS-to-delay relationship, consistent with the existing signalized intersection regime. The calculated junction delay of 27.6 seconds for the DLT design places it in LOS C, with the junction delay of 18.9 seconds for the RCUT in the LOS B range. Though it is common in engineering practice to design facilities to perform at LOS D for the projected design year volumes, these volumes are estimates of the anticipated demand twenty years after completion of the facility. Since all three of these facilities were constructed within the past five years, it is theoretically consistent that their observed performance would be in the LOS B to LOS C range.

3.6 Limitations

The authors acknowledge that this research is limited in numerous ways. An overall limitation of the study is the small number of example cases examined, and the lack of conventional designs (roundabout, signalized intersection, and diamond interchange) included; the data presented makes use of a dataset collected for other purposes, and did not include conventional designs.

The authors anticipate a fair amount of pushback from the research and practice communities regarding the generation of negative values for average vehicle delay for a specific turn-movements. This result represents a large shift in the current approach to vehicle delays, and may require significant publication and exploration in the way of comparative analysis case

studies to prove the validity and value in making such a shift in perception. The authors feel that this is a valuable effort to pursue, as the proposed methodology presents a more-truthful base travel time condition.

Regarding the proposed LOS-threshold methodology, the greatest limitation is perhaps the lack of participant involvement for the proposed changes. Further research will need to be conducted including participant perceptions of facility service to validate the ideas presented by the authors, that intersection size impacts service perceptions at least as much as control type does. Ideally, this will be accomplished as a two-step process, first validating the degree to which participants' perceptions of delay correlate between field experiences and driving simulator experiences, and subsequently testing an array of geometry types and sizes using the simulator.

3.7 Conclusions

The development of a geometric component of delay to add to the existing control delay measure is imperative for the adoption of alternative intersection geometries into our standard methods of delay and level of service analysis. However, the authors are proposing to redefine the new measure as a junction delay, a unified approach that includes the geometric impacts caused by all manner of intersection and interchange designs. This method causes large impacts to the origin-destination pairings (turn movements) at both alternative intersections and at conventional interchanges, with large increases to left-turn vehicle travel times and even larger decreases to right-turn vehicle travel times. For the DDI example examined, the resulting average delay per vehicle was -19.2 seconds for the southbound-right movement, and -15.5 seconds for the northbound-right movements. These large negative values were offset by geometric delay increases to a number of other movements, but the overall impact of geometry on the travel time of vehicles at the facility was -3.9 seconds. While the inclusion of additional travel time is essential to not give designs like the restricted crossing u-turn an unfair advantage from control delay alone, the universal adoption of this approach may have a large impact on the way travel time and delay impacts which designs are chosen. The opinion of the authors is that a unified approach such as this one would be a positive change for robust analysis of alternatives in engineering practice.

The methodology proposed by the authors to provide a LOS-to-delay relationship based on demand volume instead of control type gives a unifying approach that can be applied consistently to all intersection types currently on the roadway, as well as those designs yet to be thought of. The method speaks to the fact that intersection sizes can vary greatly within a given design type, and that drivers perceptions of delay may be influenced as much by the scope of the problem being solved as by the control type being used to solve it. For the DDI, DLT, and RCUT designs examined using the new methodology, it's noteworthy that the delay regimes each design each design was judged by were consistent with those it would have been judged by using the existing methodology. However, adopting an LOS regime methodology based on demand volume would solve a number of inconsistencies in terms of comparative analysis between a variety of intersection geometries. The threshold values proposed are based on the

MUTCD signal warrant approach of considering the busiest three approaches at a junction, with values set at 1,500 vehicles/hour for the low-to-medium threshold and 4,000 vehicles/hour for the medium-to-high threshold. These values were set based on engineering judgement by the authors, and will require significant participant involvement in future studies to calibrate.

3.8 Acknowledgements

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Chapter 4. Validation of Simulation for Alternative Intersections

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Abstract

Although access to traffic microsimulation software is far from universal within the consulting engineering industry, this software is currently a necessity for the analysis of alternative intersection designs. To meet client needs, many consultants will attempt to model these designs using widely accessible macroscopic traffic analysis software, such as Synchro or HCS. This paper uses field data from three alternative intersection designs to validate the use of microsimulation, with comparative analysis also provided from macroscopic software to demonstrate the degree to which results differ.

Peak-period field data from three alternative intersections are analyzed herein, including a diverging diamond interchange, a displaced left-turn intersection, and a restricted crossing u-turn intersection. Three software applications are used for comparative analysis, including INTEGRATION and VISSIM for traffic microsimulation analysis, and the HCM analysis as generated by Synchro traffic analysis software. Summary findings are that running uncalibrated analysis with either simulation package resulted in average intersection delay to within 10 seconds of the observed data for each design, with HCM-based Synchro analysis competitive for the DDI, nearly 25-seconds off for the RCUT, and entirely invalid for the DLT. Until such a time as new methodologies for alternative intersection/interchange analysis are implemented within HCS and Synchro software, it is recommended that alternative intersection analysis be conducted solely using microsimulation software.

4.1 Introduction

The simulation of traffic facilities is an essential tool to better understand the functionality of our transportation surface roadway network. Indeed, the underpinning concepts developed by Webster that allow traffic engineers to design signal timings come from an early computer simulation. [116] This simulation allowed Webster to model the relationship between traffic volumes, signal timings, and the resulting average delay per vehicle experienced at a signalized intersection. While traffic simulation itself is now based on individual driver behaviors and variability, the aggregate average delay per vehicle remains the key measure of effectiveness to determine the level of service provided by a signalized facility. [5] Within municipal applications of traffic engineering, local and regional laws governing development fees and approvals are today often based on the maintenance of a proscribed intersection level of service, imparting great design significance on the average delay measure. While the delay relationships are well-known for conventional signalized intersections, mounting pressure on our transportation infrastructure is forcing the industry to embrace new solutions, despite a lack of full understanding of their functionality.

As new and innovative facility designs are embraced by the industry, such as the modern roundabout in the past ten years, our understanding of the relationships between traffic demand, capacity, and delay have been tested. Research shows that not only does the unsignalized roundabout geometry result in a very different capacity-to-delay relationship, but that the relationship is impacted by the location of the facility, with different behaviors experienced in different parts of the world. [19], [117] Partially due to the success of intersection alternatives like the roundabout, a bevy of new innovative solutions collectively referred to as alternative intersections and interchanges are garnering greater industry interest. [4] Some of these designs have been embraced either regionally or nationally, at least to a small degree, but many of them lack field implementation. Predicting their performance in terms of the average vehicle delay measure is of critical importance in increasing the implementation of these designs in the field. [114], [118]

Widely used software applications for traffic engineering, such as Synchro or Highway Capacity Software (HCS), use macroscopic models generated from observed capacity-to-delay relationships for conventional intersections to predict aggregate delay for a given demand-geometry condition. Traffic microsimulation tools such as VISSIM and INTEGRATION, on the other hand, instead examine the travel experience of individual vehicles traversing the road network defined. Each vehicle is given a desired path and desired speed, and must then navigate the roadway while hampered by both the signalized infrastructure and the other vehicles on the road, which impart delay to it. Examining the difference in actual travel time and desired travel time provides the delay for each individual, and the aggregate results can provide turn-movement and total facility delays. This becomes a critical analysis for alternative designs, many of which reroute vehicles to travel through multiple coordinated traffic signals as part of the facility.

Anecdotally, practitioners without access to simulation software packages model alternative designs as piecemeal interactions given demand volumes and signal timings on an individual lane basis, combining multiple delay locations as needed to produce aggregate results. The cost for procuring the simulation software and training staff to use it may be prohibitive when compared to the loss in accuracy of avoiding simulation, but there currently exists no literature on the degree to which these findings are incorrect. The purpose of this paper is to use field data to assess the validity of both micro- and macroscopic models when applied to alternative intersection and interchange designs.

4.2 Background

As an investigation of design-level analysis tools, additional background information must be provided regarding traffic microsimulation models, alternative intersection and interchange geometries, and the potential impact of these designs on their microsimulation analysis.

4.2.1 Traffic Microsimulation Model Calibration

A number of papers from the early 2000s tackle the subject of calibrating microscopic models, and provide some useful considerations for the current study. Hourdakis et al calibrated models for a freeway facility focused on volume, speed, and occupancy, attempting to match flow-rates and queues for a highway on-ramp. These measures were found to not be a useful approach for intersection modeling, where O-D demand volumes and travel times are among the most observable inputs. [119] Looking at a test-bed area of 12 urban intersections in a network, Sacks et al. used a measure of stop-time per vehicle for calibration, examining individual vehicle events at each junction rather than tracking vehicles through the network for travel time calculations. [120] Adding to the difficulty of tracking vehicles through a network, Rakha et al. noted that there are financial and legal constraints which hamper the use of field data. [121] The reality on the ground is that practitioners rarely have access to the data necessary for calibration of their simulation models, often lack the expertise to conduct it, and almost always lack the budget to do so. As such, the results included within this study seek to ascertain the quality of the various methodologies using a basic set of assumptions without calibration.

4.2.2 Alternative Intersection and Interchange Geometries

Designs that fall into the category of alternative intersections and interchanges work by rerouting some of the turning movements made at the junction, often reducing and separating conflict points, with theoretical benefits to both safety and mobility. Conflict points represent each position where the paths of two vehicles cross, with potential collision locations identified as crossing, merging, or diverging conflicts. Any reduction in the number of potential conflicts, especially of the most dangerous crossing type, provides proportional benefit to intersection safety. This reduction in conflict points at these designs often comes with a reduction in the number of signal phases at the junction, providing a reduction in delay to through movements on the major arterial, while often increasing travel time and travel distance for secondary movements. Depending on the degree of benefit to the through movement on the major arterial

and the degree of degradation to the secondary movements, the overall average delay may be more or less than that experienced by an equivalent conventional signalized intersection.

The most comprehensive resource at the moment is the Alternative Intersections / Interchanges Informational Report (AIIR). [4] The primary focus of this document covers the displaced left-turn, median u-turn, restricted crossing u-turn, and quadrant roadway intersection designs, as well as the diverging diamond (also known as the double crossover diamond) interchange. This report provides detailed information on existing applications, typical geometric design and signal locations, as well as potential signal phasing for each design. Recommended procedure to analysis of these designs is to use a critical-lane analysis method for preliminary analysis, following up with comparative microsimulation analysis of preferred alternatives. From a research perspective, this is essentially the order that the authors have pursued in research publications.

4.2.3 Analysis of Alternative Geometries

This research is part of an ongoing series of papers by the authors on the operational analysis of alternative intersections and interchanges. While the majority of the authors recent related publications are on planning-level analysis of these designs [122]–[125], additional work has included a comparative analysis study of roundabout and through-about designs [126]. This previous literature examines low-fidelity estimation analysis methods, validating the results using the higher-fidelity simulation tools. However, previous efforts have not specifically validated the use of simulation software against field data for these designs, prompting the current work.

If an alternative geometry is modeled, the degree to which it is accurate relative to reality is a function of how closely the simulated behavior of individual drivers matches that observed in the field. In the case of the modern roundabout, the simulation parameters of note are that of gap acceptance and follow-up headway, which can be field verified and calibrated as needed. [19] Applying this idea to other alternative designs, we may examine if and how the behavior of drivers changes when traversing them, but ultimately, we must determine whether the total aggregate delay measures are accurate. For the work included herein, the authors are more interested to validate the bare-bones analysis methods as an accurate framework for analysis, and are less interested in the ability of various software applications to calibrate and match field conditions for a particular site on a particular day.

4.3 Purpose

Due to the small number of alternative intersection and interchange designs implemented in the field, and the diverse site constraints experienced by each implemented facility, it is not at this time feasible to pursue large-scale studies into the operational measures of effectiveness for this category of design using field data. By validating the aggregate measure of effectiveness from microsimulation results against observed field data for a few test cases, it is the intention of the authors to demonstrate that the study of operational performance of alternative intersections can

be robustly pursued with simulation. As such, the purpose of this study is to validate the degree to which traffic microsimulation software packages are able to provide accurate delay estimates when modeling alternative intersection and interchange designs. Macroscopic software applications are also used to assess the state of the practice for traffic engineering. The authors feel that this effort will be of value both to researchers and practitioners, providing assurance that comparative analysis conducted on these designs provides reliable results.

4.4 Evaluation Approach

Video data collected at multiple intersection and interchange locations is used to provide observations of in-situ driver behavior. Signal timings and turning-movement counts are taken from the video data for use in generating robust simulated facilities. Travel times for individual vehicles traversing each facility are generated using observations from the video data, and the aggregate travel-time (delay) measures are compared against the equivalent results of simulation.

4.4.1 Field Data Collection

Three alternative designs were chosen for conducting this study, including the diverging diamond interchange (DDI), the displaced left-turn intersection (DLT), and the restricted crossing u-turn intersection (RCUT) designs. The DDI selected is located at the junction of Interstate-15 and West Main Street in American Fork, Utah. The DLT selected is located at the junction of Eisenhower Boulevard and Madison Street in Loveland, Colorado. The RCUT selected is located at the intersection of Ocean Highway E. and Ploof Road in Leland, North Carolina. Field data collection was conducted in late May and early June of 2013. Video data using multiple GoPro cameras was collected, covering both AM and PM peak periods with simultaneous feeds covering each direction of traffic. In some cases, such as with the DDI installation, as many as 12 simultaneous video feeds were collected during two hours each peak period. On the following page, **FIGURE 4-1** provides the geometry for each site analyzed, as seen in the aerial images.

With each video feed synchronized to the others and tied back to the clock-time, turning movement counts in 15-minute intervals are conducted using the video data. Signal timings, where possible, are generated from the video feeds with direct line-of-sight to the traffic lights. When no direct line-of-sight is available to the light controlling a given turning movement, the onset of green is observed using the movements of vehicles at the beginning of queue, assuming a one-second offset to account for driver reaction time. The details of the phase diagram, including partially and fully actuated movements, green extension times, minimum and maximum green times, cycle lengths, offsets, etc., are generated using a combination of observed traffic signal behavior, and use of industry best practices such as from the Signal Timing Handbook.

4.4.2 Analysis Methodology

While the authors do not intend to calibrate the simulation models analyzed herein, they do seek to represent as accurately as possible the ground truth conditions as observed in the field. To this

end, the intersections are modeled as accurately as possible to match the design geometry as seen in aerial photogrammetry. Traffic demand volumes are observed from video data collected during the peak period, with individual vehicles tracks from entrance to exit through the site to generate origin-destination travel time observations. The video data additionally provides insight into the signal phasing as applied in the field, with analysis conducted using the in situ signal phasing and timings, rather than optimized for the specific volume demands observed. By coordinating the video times from the various feeds, signal offsets can be calculated for those designs with multiple signalized intersections operated from a central master controller.



(a) Junction of Interstate 15 and State Route 92, Lehi, Utah (DDI)



(b) Junction of Eisenhower Boulevard and Madison Avenue, Loveland, Colorado (DLT)



(c) Junction of Ocean Highway East and Ploof Road, Leland, North Carolina (RCUT)

FIGURE 4-1 Aerial location maps for data collection sites

4.4.3 Software Application Parameters and Input

One note that may be of interest to practitioners and researchers is that the authors purposefully used out-of-date versions of both VISSIM and Synchro for the results presented herein. The versions used include release 2.40 of INTEGRATION dating to 2012, release 4.30 of VISSIM dating to 2007, and version 6 of Synchro dating to 2003. The understanding of the authors is that these versions represent the most recent iterations of each program not requiring either a networked license or an ongoing subscription fee, and as such would be consistent with the software employed by the majority of practicing engineers facing budget constraints.

In defining the separation between model calibration and model creation, the authors choose to take any readily observable information, such as the roadway geometry, the speed limits, the signal timings, and the demand volumes, to be part of the model creation. One note on the difference between the macroscopic HCM method and simulation is that INTEGRATION and VISSIM have built in variance in each simulation run. To collect a sufficient amount of data, both simulation software application were run for ten seeds of twenty-five minutes each, with fifteen minutes of data used from each run, with vehicles scheduled to enter the network between five and twenty minutes during the simulation.

The geometric input for Synchro requires a node-link layout, based on overlaying centerlines of roadways onto an aerial background image which has been appropriately scaled. The various designs posed different sets of problems in modeling in this link-node format, with the DDI effectively modeled as a network of one-way streets, the DLT looking similar to a pinwheel with the left-turn bays looking like the blades, and the RCUT looking similar to a diamond interchange, without a bridge crossing the middle. The key for HCM modeling is to match each demand flow with the appropriate number of lanes and signal timing, at which point individual movements may be modeled separately without regard to the overall junction. Signal phasing and timing must be explicitly defined, but once that's done the report is quickly generated.

The logic for geometric input in INTEGRATION is similar to that of Synchro, but in this case there isn't a graphical interface, with the nodes entered based on coordinate locations, and links defined by which nodes they connect and how long they are, a process aided by the use of overlaying a node-link network on an aerial image in a CADD drafting program. Some of the designs proved challenging to model while maintaining minimum link lengths, with the displaced left-turn design requiring a couple of iterations to solve. The desired speed and other vehicle behavior parameters are set in the link file. For consistency with HCM 2000 recommendations, the basic saturated flow value is set to 1900. Jam density is set to 150 veh/km to be consistent with VISSIM default values. The authors defined free flow speed and the speed at capacity, as the posted speed limit and half of this value for each link, respectively, consistent with prior research findings for arterial driver behavior. [127]

The VISSIM input is the most graphically-oriented of the three, with the network defined as a series of lane segments connected to other lane segments, and stop-bar locations defined explicitly by location on each lane approach. When defining origin-destination flows the

software highlights the default set of links by which vehicles will traverse to their goal, allowing users to validate the geometric connections. One problem for the analysis of alternative intersections in this case is that users must first have a near-fully realized geometric design before the simulation can be created. Of the three, the graphical interface of VISSIM makes it the most versatile (easiest to use) in dealing with the unconventional lane configurations encountered with alternative designs. However, this advantage would become a liability in the case of modeling large networks, where modelling each connection would be time consuming.

4.5 Research Results

The information provided as research results represents the data obtained both from the observed field conditions and from the modeling efforts performed for this study. For interpretations on the data, as well as judgements and conclusions drawn, please see the analysis section of the paper, which follows.

4.5.1 Observed Signal Phasing and Timing

An essential component of modeling the in situ conditions required accurately understanding the operations of the signalized control being implemented in the field. In many cases, the traffic signal lights themselves were visible in the videos. In cases when none of the cameras were able to view the light, the start of green was used as one-second prior to the first movement of vehicles at the stop bar. The recorded fifteen-minute period of data is shown in **FIGURE 4-2**, with the equivalent pre-timed signal timings shown in **TABLE 4-1**.

TABLE 4-1 Pretimed-equivalent signal timings

Data Source		Cycle	Green	Yellow	All-Red	Green Start	
Diverging Diamond	West	Southbound Left	75	39	3	3	4
		Eastbound Through		23	3	3	50
		Westbound Through		43	3	3	0
	East	Northbound Left	90	41	3	3	4
		Eastbound Through		45	3	3	0
		Westbound Through		31	3	3	51
Displaced Left-turn	Cross	North/South Left	140	20	3	3	0
		South/North Through		108	3	3	26
	Main	North/South Left	140	27	3	3	8
		North/South Through		27	3	3	17
		East/West Left		16	3	3	50
		East/West Through		64	3	3	72
Restricted Crossing U-turn	East U	Eastbound U-turn	100	10	3	3	0
		Westbound Through		78	3	3	16
	North	SB Right / EB Left	100	21	3	3	31
		Westbound Through		67	3	3	58
	South	NB Right / WB Left	100	10	3	3	3
		Eastbound Through		78	3	3	19
	West U	Westbound U-turn	100	18	3	3	44
		Eastbound Through		70	3	3	68

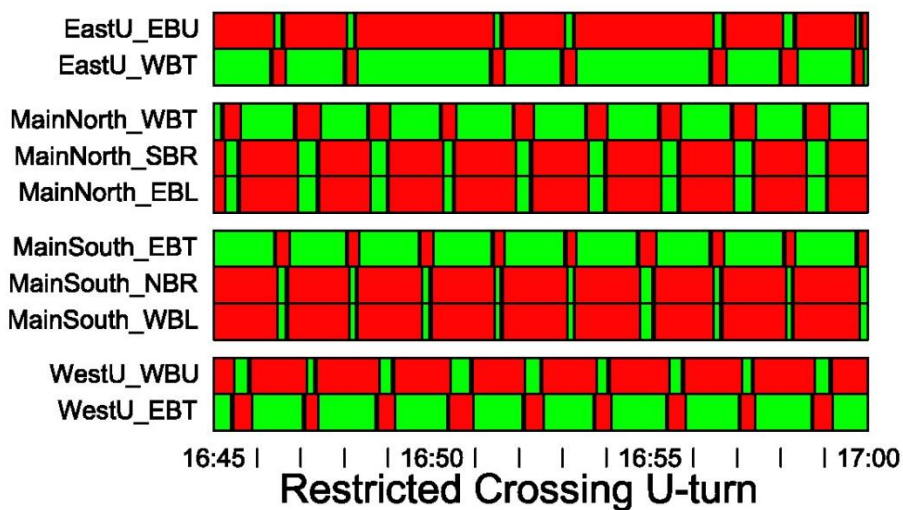
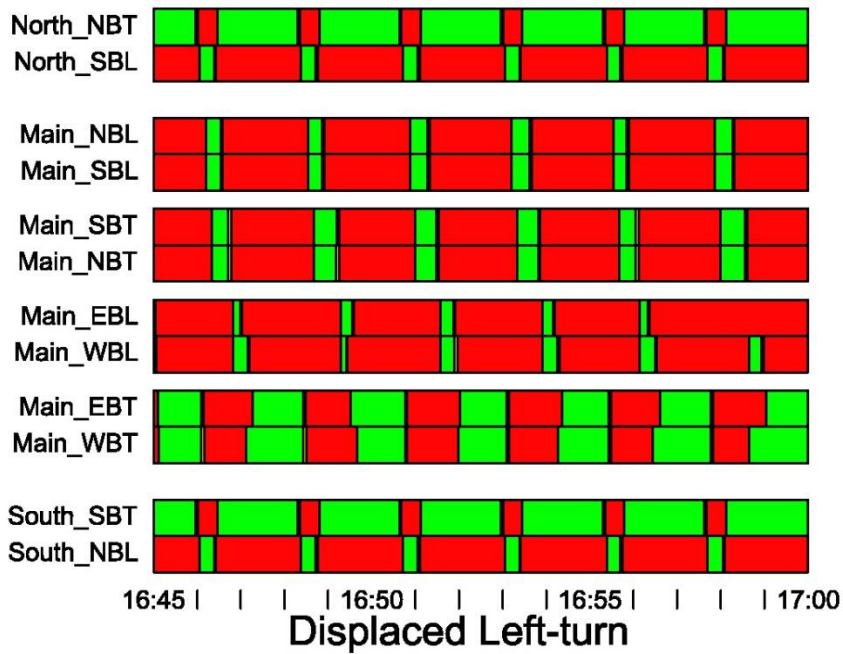
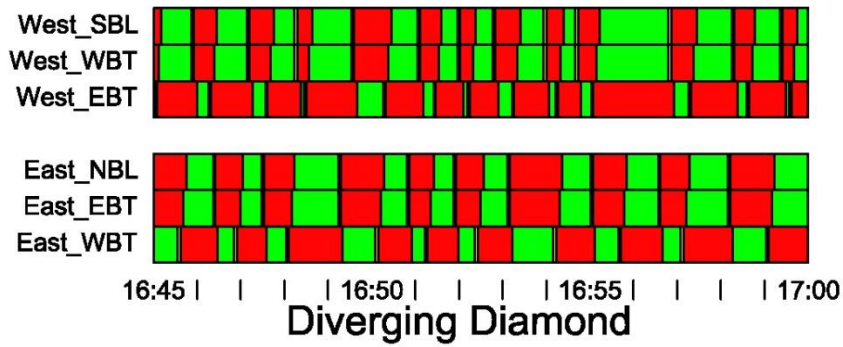


FIGURE 4-2 Observed peak period signal operations for DDI, DLT, and RCUT facilities

4.5.2 Green Band Progression

By examining the signal phasing of multiple signalized locations within a given design, the internal green-band progression at the junction may be observed.

For the diverging diamond interchange, it was found that the eastern signal completed 12 cycles, while the western signal completed 15 cycles, indicating that these two locations are operating entirely independent from one another. For both signal locations, the cycle lengths themselves were found to be irregular.

The observed traffic signal and green-band progression information for the DLT northbound and southbound left-turn movements is shown in **FIGURE 4-3**. In this case, a well-defined signal phasing scheme is observed, with considerations for progression for all turning movements encountering multiple signalized control points.

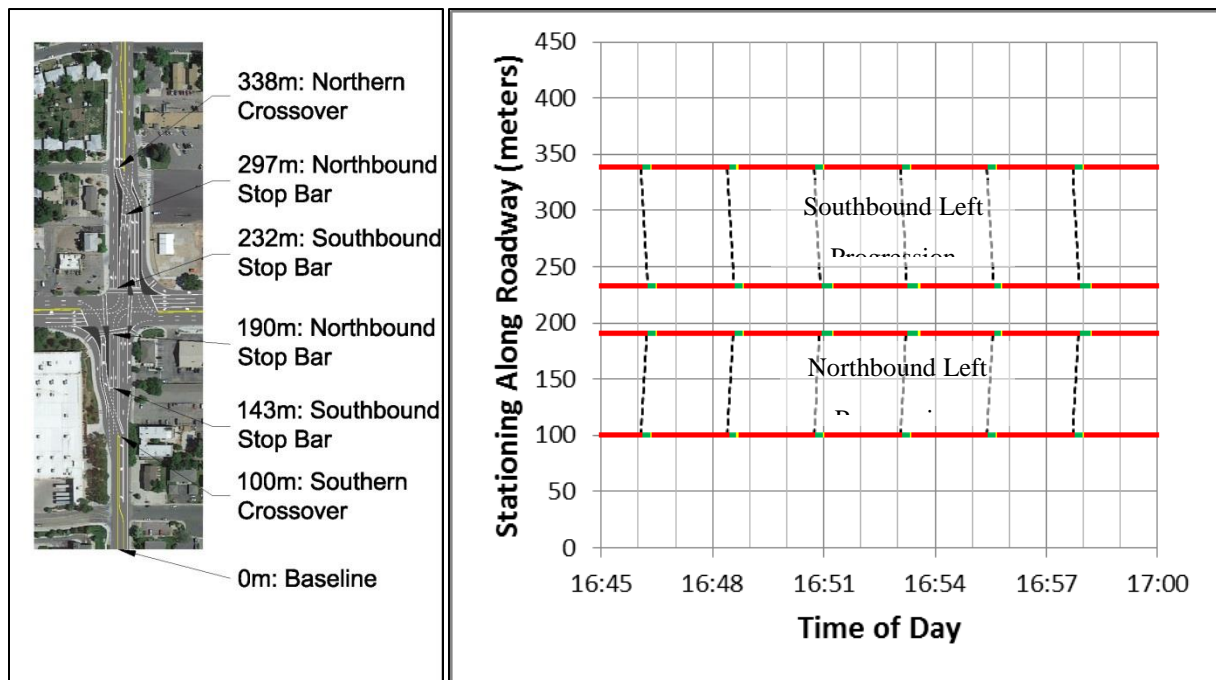


FIGURE 4-3 Displaced left-turn green band platoon progression

The observed traffic signal and green-band progression information for the RCUT site are provided in **FIGURE 4-4**. In this case, a well-defined signal phasing scheme is observed, though there are questions about the progression provided by the observed phasing scheme.

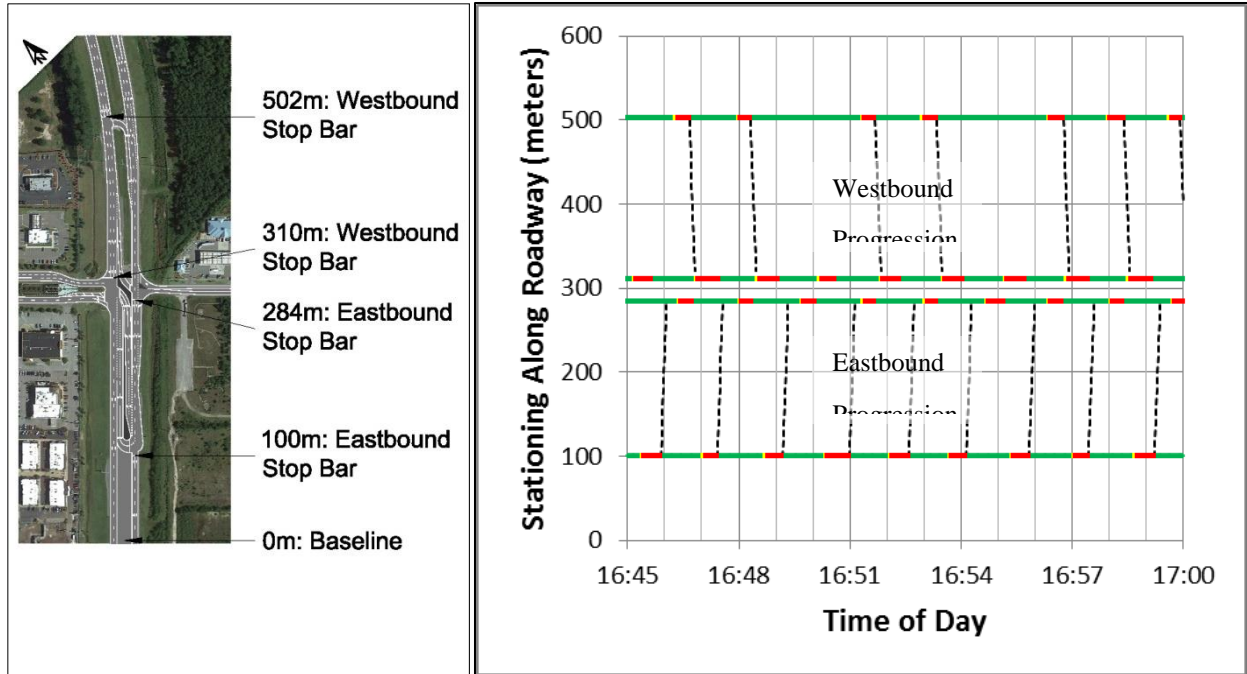


FIGURE 4-4 Restricted crossing u-turn green band platoon progression

4.5.3 Observed Peak Period Traffic Volumes

Video data collection at each site included a total of four hours of traffic, covering both two hours in the AM peak period and two hours in the PM peak period. Due to the time-consuming process of tracking individual vehicles through the network to obtain origin-destination travel times, it was determined that the fifteen minute period between 4:45 and 5:00 PM would be used for analysis. Turning movement counts as shown in **TABLE 4-2** are based on all vehicles entering the network during the fifteen-minute period recorded, multiplied by four to obtain hourly flow-rates.

TABLE 4-2 Origin-destination traffic demands by location

Location (Design)	Northbound			Southbound			Eastbound			Westbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
I-15 and SR92 (DDI)	924	x	556	464	x	268	120	492	692	368	756	484
Eisenhower and Madison (DLT)	52	184	136	316	188	64	80	1524	40	132	780	340
Ocean Hwy E. and Ploof (RCUT)	52	44	72	192	32	240	228	1296	84	100	1500	228

4.5.4 Aggregate Travel Time and Delay Measures

Using a combination of the video input feeds and the roadway geometry, a location along each approach and exit from the junction was chosen as the limits of the network for the sake of travel time calculations. Time was logged for each vehicle as it crossed the threshold entering the network, and subsequently as it crossed the corresponding threshold exiting the network, with the collected travel times then aggregated by turn movement and by overall intersection. The aggregate intersection average delay per vehicle for each of the four cases, field observation, VISSIM, INTEGRATION, and HCM method, are shown in **TABLE 4-3**.

The measure of delay for the HCM style report comes directly from the report, and is not associated with a travel-time measure. However, the delay values indicated for the observed field data, VISSIM, and INTEGRATION results are based on subtracting a base travel time value from the individual vehicle travel times observed, with the base travel time values being defined by turn movement. The minimum of two values is taken as the base travel time value for a given turn movement, the first being the theoretical travel time needed to traverse the given distance at the posted speed limit, and the second being the 5th percentile travel time observed from the field data for a vehicle completing the movement. Whatever value is determined as the base travel time for the field data is then applied to the observed travel times from VISSIM and INTEGRATION simulation to determine their respective delay measures. In this way, it is possible, and indeed observed, that the fastest vehicles observed both in the field data and in the simulations result in negative delay values. These negative values represent drivers exceeding the speed limit, and their values are left intact in calculating the aggregate delay observations.

TABLE 4-3 Average delay per vehicle

Data Source	DDI	DLT	RCUT
Field Data Observation	24.6	28.5	24.8
VISSIM	28.3	24.6	26.9
INTEGRATION	31.0	33.7	26.9
Synchro (HCM Report)	22.0	158.6	49.5

The observed travel times for the individual turn-movements is shown in **FIGURE 4-5** for the DDI, **FIGURE 4-6** for the DLT, and **FIGURE 4-7** for the RCUT designs, respectively. The box plots indicate the mean average vehicle delay with a diamond shape, and use the box-plot format for communicating the distribution of delay experienced by the vehicles in the dataset. The box-plot format shown uses the 5th and 95th percentile values as the whiskers, with the 25th and 75th percentiles as the upper and lower limits of the box itself. A line within the box indicates the median observed value. The four data sources are shown as black for field data, blue for VISSIM, red for INTEGRATION, and green for HCM methodology, respectively. The values used to generate the plots are shown to the right, aligned with each box plot.

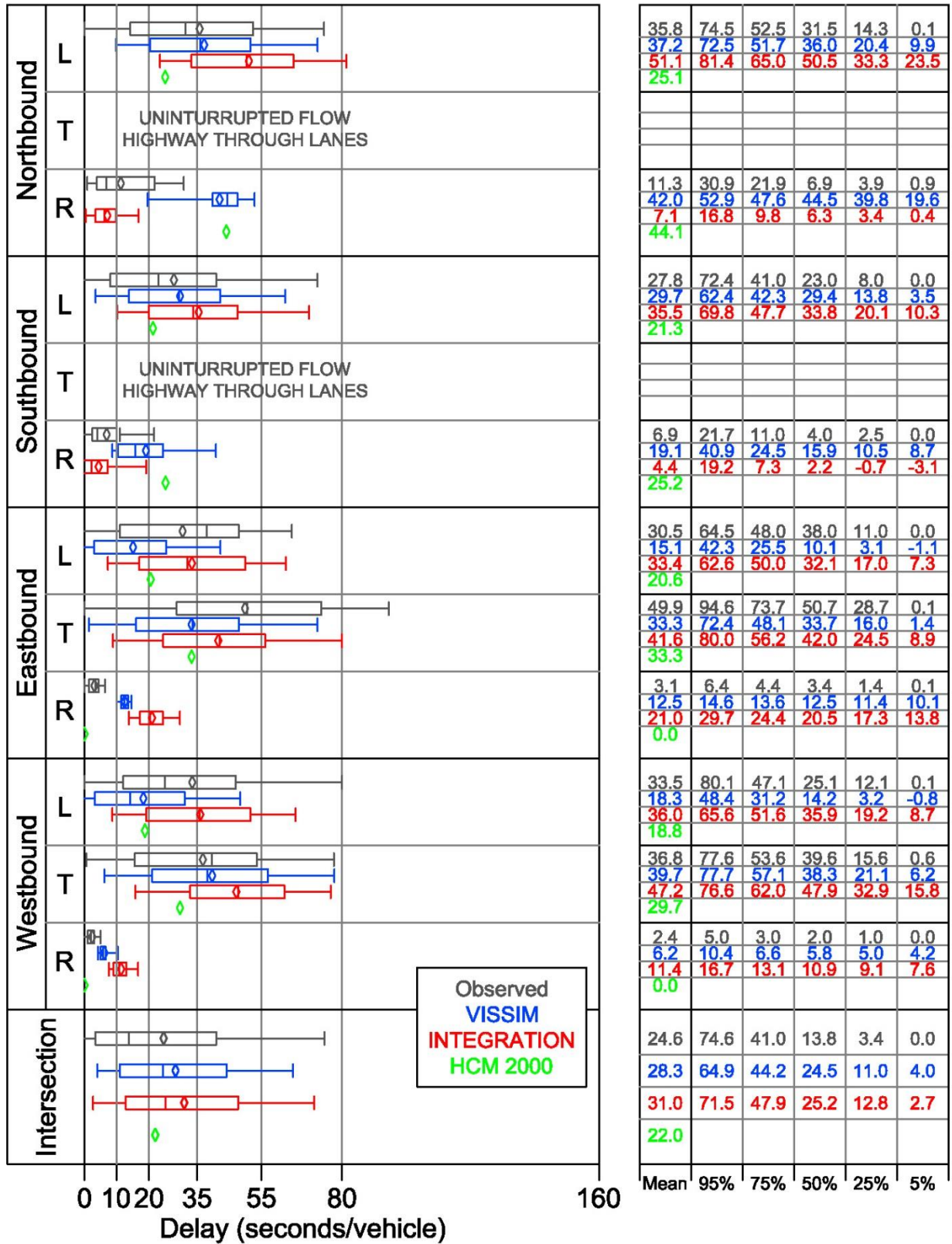


FIGURE 4-5 Delay observations for diverging diamond interchange

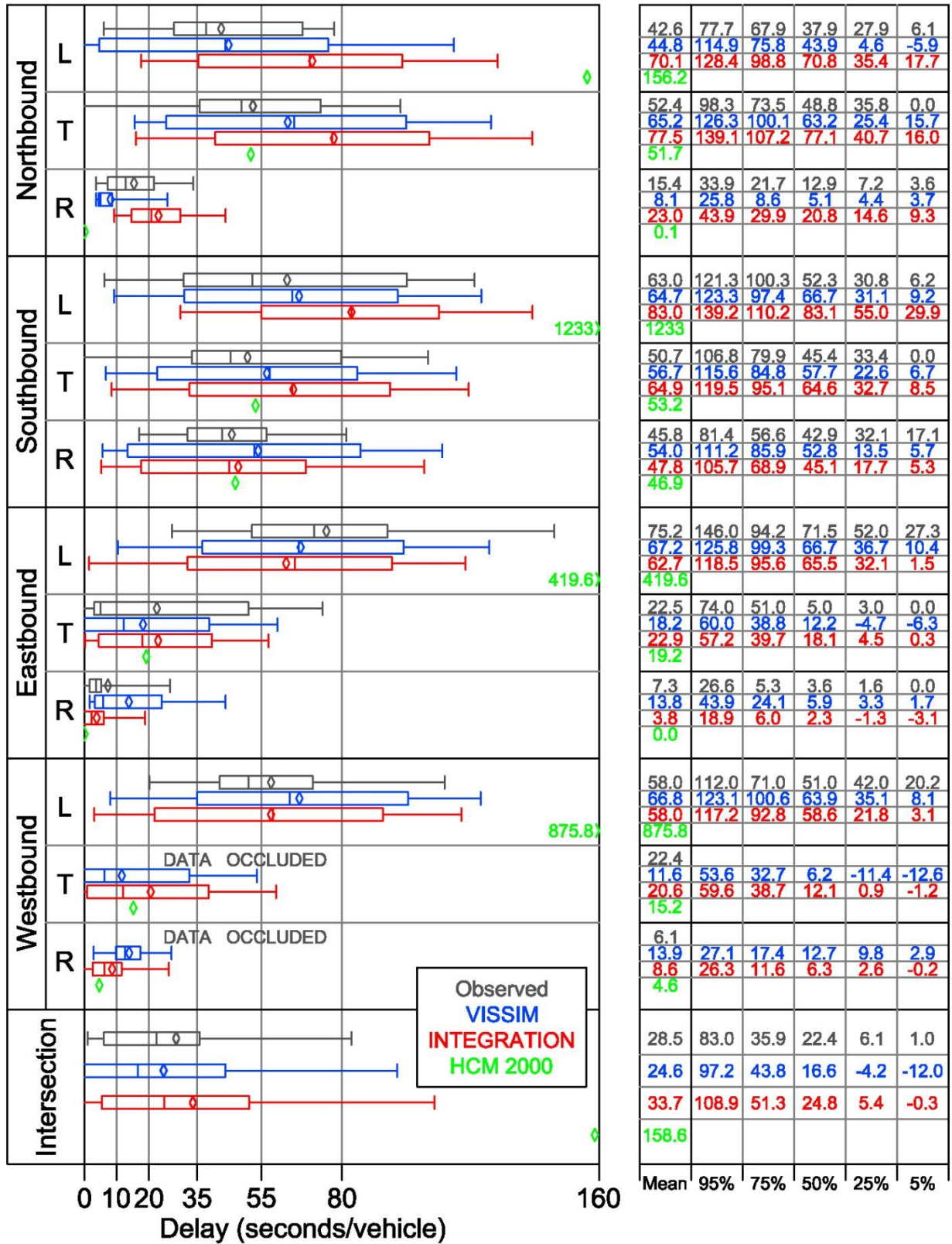


FIGURE 4-6 Delay observations for displaced left-turn intersection

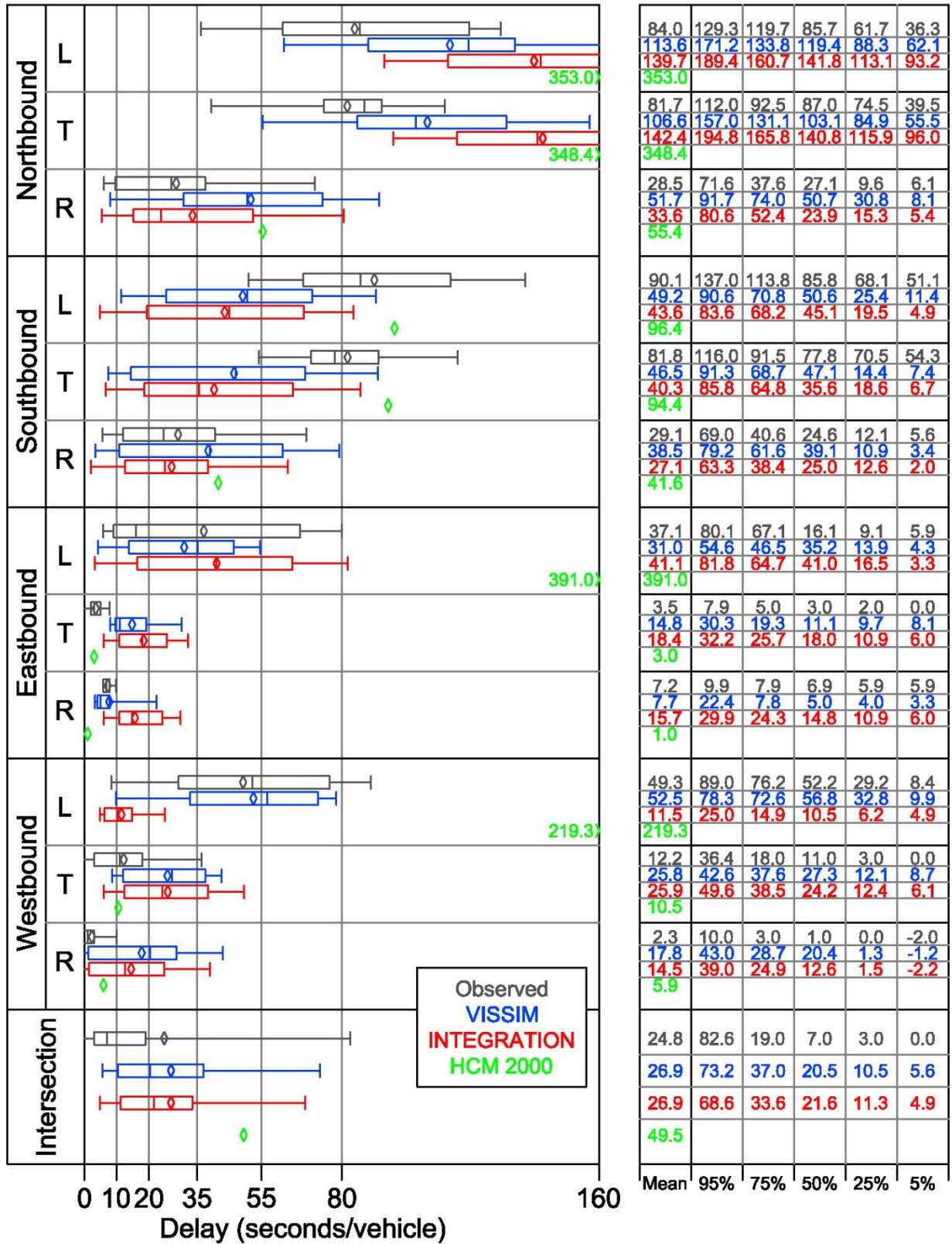


FIGURE 4-7 Delay observations for the restricted crossing u-turn intersection

4.6 Analysis of Results

4.6.1 Analysis of Signal Timing and Platoon Progression

The lack of coordination between the eastern and western junctions of the DDI is worth some discussion. In the majority of applications for actuated (non-pre-timed) signals, the minor movements are actuated to time-out or skip when lacking demand, with the additional time provided to the major movement, maintaining a constant cycle length. In this case, it appears as though both the eastbound and westbound movements are designed to gap-out, such that the cycle itself is allowed to end early transitioning to the next cycle without a consistent offset time between the two signal locations. On further investigation, we can see that the northbound left and the westbound through demand volumes are approximately equal, so neither one can be prioritized for progression in the westbound direction. Likewise, the demand flowrates at the western intersection show very similar values for the eastbound through and southbound left volumes, so progression in the eastbound direction cannot be prioritized for one or the other. As a result, the benefit for providing consistent signal cycle lengths is lost,

At the displaced left-turn intersection, the decision to provide the majority of the green time (108 seconds) at the crossover locations to the through movements is interesting, as the channelized right-turn bays ensure that through vehicles only occur at this location during the east/west left-turn phase and the north-south through phase (a total of 49 seconds including the clearance interval between the phases). Vehicles traveling the north/south left-turn movements experience all of their delay in one location, before the crossover. This assumes that the offset of green start times are set correctly in the field, which **FIGURE 4-3** previously indicated they had been. One benefit of this decision is that there is no chance of queue spillback from the left-turn bays at the main intersection back through the crossover, but consequentially, the queue lengths for the left-turn bays beyond the crossover locations must be significantly longer to not generate their own spillback effects on the north/south bound approaches.

The restricted crossing intersection has the greatest potential to provide through progression when built in series along an arterial. Because the timing of the north-south disconnect at the central location of the design, the eastbound and westbound green bands can be independently coordinated with adjacent intersections, providing progression concurrently in both directions along the roadway. The distance between the stop-bar at the u-turn location and the stop-bar at the main intersection for both the eastbound and westbound directions for the intersection observed is approximately 190 meters, a travel time of approximately 9.5 seconds based on the posted speed-limit of 45 miles per hour. However, the observed offsets for start of green during the peak period are approximately 45 seconds for the westbound-through movement, and 50 seconds for the eastbound-through movement. As a result, the existing operation of the signal consistently has the highest-volume through movements on the arterial leaving at the start of green from the u-turn location only to arrive at the main intersection at the end of the green phase, as indicated in **FIGURE 4-4**, imparting unnecessary delay to these critical movements. The lack of signal progression in the observed condition will not impact the simulation results,

because the same signal offsets and progression are maintained for that analysis, but it may impact the results of the macroscopic analysis, which does not automatically take into account progression considerations between nodes.

4.6.2 Analysis of Delay Measures

Of great importance when assessing the validity of various analysis methods to model a given design is to consider the individual turn movement behaviors modeled. In some cases, designers may be seeking a design with an overall level of service of C or better, but they are constrained to maintain every individual movement at the junction to a level of service of D or better. An analysis which correctly predicts the overall average delay, while significantly under-estimating some movements and over-estimating other movements, may appear to be a valid analysis tool on the surface, but may then unfairly criticize a given design for flaws that aren't there. That said, comparing the results of the various methods against the observed behavior can provide valuable information for researchers and practitioners on the biases of these tools.

All three analysis methodologies provided sufficiently accurate predictions for the overall average intersection delay for the DDI design. The observed average delay was 24.6 seconds, with Synchro predicting the smallest delay at 22.0 seconds, VISSIM predicting 28.3 seconds, and INTEGRATION predicting 31.0 seconds; consistently in the LOS C range. Examining some of the details from **FIGURE 4-5**, all observed and predicted movement delays are operating at LOS D or better, with the vast majority of predicted timings falling into the same LOS regime as the observed behavior. One observation of note is that the lane utilization on the east/west through movements was much more evenly distributed in the simulation than it was observed in the field, which didn't greatly impact the results herein, but may add significant delays to the observed condition at demand volumes closer to capacity. The authors postulate that the results of all analysis methods so closely matched the field observations because the behavior of the DDI design is more like that of two one-way streets, with the geometric design of each individual crossing location very closely matching normal roadway conditions.

The results provided for the DLT design provide for the most cautionary example of using macroscopic analysis to predict alternative design behaviors. The data from **FIGURE 4-6** shows how the HCM analysis correctly predicted the vehicle delays for through movements at the intersection, but erroneously overestimated the left-turn delays to a very high degree, despite being programmed with the exact signal timings observed in the field. As a result, the Synchro analysis predicted an overall average delay of 158.6 seconds per vehicle, in contrast to the 28.5 seconds observed. Both of the microsimulation results are within an acceptable margin of error, with 24.6 seconds for VISSIM and 33.7 seconds for INTEGRATION. The individual turn-movement predictions from simulation are fairly consistent with the observed values, though INTEGRATION is predicting northbound and southbound delays that are higher than expected.

The aggregate average delay predicted by the simulation applications of 26.9 (for both) were very close to the observed delay of 24.8 for the RCUT design, with the HCM methodology prediction of 49.5 seconds being twice that of the observed condition. Examining the individual

movements from **FIGURE 4-7**, we see that the observed delay for northbound and southbound movements in the field is approximately equal, while the simulated conditions incurs significantly higher delays on the northbound approach, offset by significantly lower delays on the southbound approach, this in spite of the signal timings being taken from the observed condition. As with the DLT design, the HCM methodology primarily fails on the left-turn movements, with delay estimates of nearly 400 seconds per vehicle for these movements heavily impacting the aggregate average. In both cases, an adjustment to the percentage of green-time allocated to the turn movements may solve the problem, however, this would require modifying the field observed inputs to better match the resulting field observed delay, which defeats the purpose of accurately modeling the design.

4.7 Limitations

The research presented herein is limited in a number of different ways, which may impact the robustness of the conclusions reached, but which the authors wish to state as known issues.

The designs chosen for review here were carefully chosen, but in no-way represent a definitive collection of alternative intersection designs. Many additional types of alternative designs, such as the quadrant roadway, the median u-turn intersection, and others could have been included. Also, the designs shown are not ubiquitous geometries for the design categories they represent, but instead are individual iterations of a given style of design.

The example locations were chosen specifically because there is ongoing research to incorporate them into future versions of the highway capacity manual. While the authors recognize that the development of macroscopic techniques applied to these designs may reduce the utility of the research presented herein, we feel that the current research provides significant contribution at the present time, and that it may serve as a foundation for future work to examine the validity of these new procedures.

The fifteen-minute peak period analyzed provides a sense of the average delay experienced in the field, but multiple days of data, with significantly more observations would be preferable given unlimited budget and unlimited time. As an individual run of a given simulation scenario provides an indication of the true result, so too does the individual peak-period observation collected in the field.

The authors previously mentioned the age of the software applications employed for this study. While we feel that there's a valid argument to be made for using these older iterations of the various software applications, there may be an equally valid argument for conducting the analysis with up-to-date software. Additionally, the software applications used are in no way exhaustive of the possible software applications for use in traffic simulation or macroscopic analysis, and incorporating other software applications into the research may provide a more nuanced understanding of the problems encountered in analyzing these designs.

4.8 Conclusions

The purpose of this paper as stated is to assess the validity of micro- and macroscopic tools currently employed to analyze the operational performance of alternative intersection designs. Based on the analysis conducted herein, the authors find that microsimulation applications provide robust results for all types of intersection geometries, predicting delay measures for both average intersection delay and individual turn-movement delays with sufficient accuracy for design purposes. The macroscopic HCM methodology was found to accurately predict delays associated with the DDI design, but was unable to predict delays for either the DLT or RCUT designs. Primary failures for HCM methodology predictions came from the left-turn movements for the at-grade designs, with macroscopic analysis predicting oversaturated flow conditions when using observed signal timing plans, despite microsimulation accurately modeling these same movements with identical inputs.

The authors recommend two expansions of the current research. First, the inclusion of additional alternative intersection designs, such as the jughandle, the median u-turn, and if possible a field implementation of the quadrant roadway design. Second, future research should include analysis of the ongoing research developing HCM methodologies for the DDI, DLT, RCUT, and median u-turn designs, once this information is released.

4.9 Acknowledgements

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Chapter 5. Sample Comparative Analysis of Alternative Geometries

Citation for original publication:

Sangster, J. D., Rakha, H. A., & Al-Kaisy, A. (2015). Comparative Analysis of the Through-about, Roundabout, and Conventional Signalized Intersection Designs. Transportation Research Board, 94th Annual Meeting of the. Washington, D.C.

Abstract

The modern roundabout has become a very popular alternative intersection design within the United States. As this design becomes a more standard solution in keeping with European practice, we are likely to encounter more frequent occurrences of the through-about intersection, a related design. This design, while not well documented in the literature, occurs on the spectrum of alternative intersection designs.

This paper examines the operational aspects of the through-about intersection design. Comparative analysis of overall intersection delay is conducted using the INTEGRATION traffic microsimulation software package, with data for the conventional intersection verified using the HCM methodology embedded within the macroscopic-based Synchro traffic software. Secondary measures of effectiveness are discussed, including safety, cost, and fuel consumption.

The analysis of this design shows that the through-about may not be successful as a direct competitor for either the modern roundabout design or for the conventional signalized intersection. Instead, this design should be considered as a site-specific solution to achieve effective safety and access considerations where a high-volume arterial intersects with one or more low-volume roads, potentially with large skew angles.

5.1 Introduction

The topic of alternative intersections is becoming one of increasing interest in the United States. With the ongoing integration of the modern roundabout design into the transportation engineering industry, more professionals and researchers are taking note of potential alternatives to the conventional signalized intersection design. There remain few options for analyzing these alternatives short of full micro-simulation, which may be preventing these designs from experiencing more widespread adoption by the industry. A design alternative related to the modern roundabout is the through-about, which includes a circulatory road with signalized through lanes bisecting it to allow for greater throughput on a major arterial.

The purpose of this paper is to assess the value of the through-about intersection design as an alternative option to a conventional signalized intersection design from operational perspective. Analysis considers one major arterial crossing with one minor arterial at a ninety-degree angle, with alternatives including a conventional signalized intersection, a modern roundabout, and a through-about intersection design. Sensitivity analysis is conducted on total volume, percentage of left-turns on the major arterial, and the volume split between the major and minor arterial.

The primary measures of effectiveness (MOE) in traffic operations at intersections include the average delay experienced by all vehicles as they traverse a given facility, measured in seconds, followed by the percentage of potential capacity being utilized. Once operational requirements are established, secondary measures of effectiveness move beyond operations and include an assessment of the safety of a design, access it may or may not provide to adjacent property, expected cost of construction and maintenance, and fuel consumption experienced by vehicles traversing it. This paper will focus primarily on measures of vehicle delay, but will touch upon other MOEs as the data allows.

5.2 Through-About Geometry

The through-about design, sometimes known as the “hamburger” intersection, consists of a circulatory road pierced by a major roadway, with through lanes crossing through the circulator. As shown in **FIGURE 5-1**, traffic in the major roadway maintains through movement using signalized intersection(s), while all turning movements from the major roadway and all traffic movements from the minor crossing roadway utilize the circulator. This configuration allows the signal control to operate with a two-phase signal plan, thus reducing lost time. Left-turns are prohibited from within the middle section of the through lanes, forcing left-turning vehicles from the major approach to use the circulator to access their destination.

There are two mentions of the through-about intersection in recent literature. The first is in the Alternative Intersections and Interchanges Informational Report (AIIR) [4], and the second from a TRB conference paper on a proposed new design, the Reduced Conflict Intersection (RCI) [128]. The material included in the AIIR references a 2004 report from Iowa State [106], though upon reviewing this source material it was found that no reference to either the through-about or the hamburger intersection is made therein. The later publication on the RCI provides more detail on the through-about design than did the AIIR, but it also lists the Iowa publication as its source material. Further review indicates that the RCI paper combined information

provided in the AIIR with information found on the “roundabout” article on Wikipedia [129]. None of the information provided about the design on the Wikipedia page is referenced to an external source.

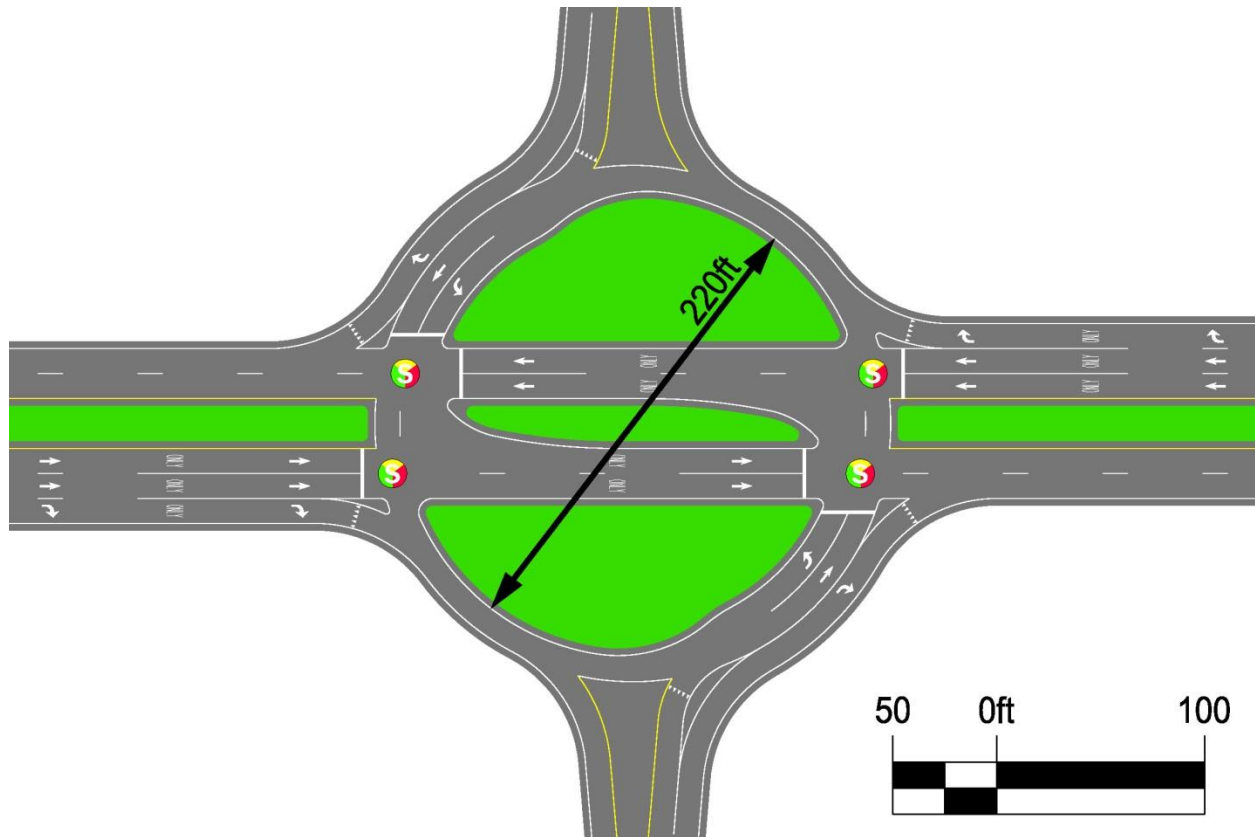


FIGURE 5-1 Example through-about geometry

5.2.1 Existing Implementations

Despite a lack of literature on the design, a number of locations exist where it has been implemented. Without citation, some of the literature cites these designs as being popular solutions in both the United Kingdom and Spain. Within the United States, there are currently three examples documented, though the authors believe that all of these designs date from a time when rotaries were popular, and are not associated with the recent proliferation of the modern roundabout. The differentiation between the designs is well documented in the current state-of-the-practice document: Roundabouts: An Informational Guide (second edition) [19]. The older rotary design is differentiated from the modern roundabout by a larger diameter, often in excess of 300 feet. This large diameter was often set based on the length of weaving section required between intersection legs, and resulted in higher speeds of travel and greater safety concerns than its modern counterpart. Modern roundabouts are designed to maintain lower circulating speeds, with an inscribed diameter between 90 and 180 feet for single lane roundabouts, and 150 to 300 feet for multilane roundabouts. The smaller radii of these designs force drivers to slow their speed to comfortably traverse the curve of the road, with the slower speeds achieved reducing

both the frequency and the severity of crashes at the site. Aerial images of the domestic example locations can be seen below in **FIGURE 5-2**. These designs are not true circles, but are oblong in shape, with locations that include Fairfax, Virginia with a 200/230 foot inscribed diameter, Everett, Massachusetts with a 400/450 foot inscribed diameter, and Cherry Hill, New Jersey with a 230/330 foot inscribed diameter. The diameter dimension along the major arterial may be governed by the need to provide adequate queuing space.



(a) Fairfax and Old Lee, Fairfax, VA



(b) Revere Beach and Mystic View, Everett, MA



(c) Kaighn Ave and Church Rd, Cherry Hill, NJ

FIGURE 5-2 Existing domestic implementations of the through-about

Aspects that these locations have in common are the intersection of a major arterial with high through vehicular volumes, and skewed approaches on minor streets. While the Virginia location features only two minor approaches heavily skewed from the primary arterial, the Massachusetts location includes three minor approaches and the New Jersey location includes four. The accommodation of pedestrians and bicyclists are handled differently at each location. In Fairfax, VA, at-grade crossings are provided in all four directions, with north-south movements accommodated on the outer edge of the circulator, and east-west movements accommodated within the green space. The Everett, MA location accommodates pedestrians and bicycles with grade-separated facilities, maintaining traffic throughput. No accommodation is

provided at the Cherry Hill, NJ location. In general bicycle and pedestrian accommodations are a site-specific concern, and the analysis of their interaction is not explored herein.

Information was provided by a traffic engineer working for the city of Fairfax, VA regarding the history of the through-about in their city [130]. The single through-about intersection located in Fairfax is the last remaining of a number of different locations that have been reconstructed over the years due to safety concerns. Many of these resulted from the initial construction of rotaries or traffic circles in the 40's and 50's, with through lanes subsequently added in later years to accommodate growing traffic volumes in the area. Safety concerns arose due to frequent accidents, largely due to weaving movements with business access points in close vicinity to the intersection. The remaining location is operating within acceptable mobility and safety measures, and the cost to reconfigure the junction would not be justified.

5.3 Evaluation Approach

Though a number of the alternative intersection designs are being integrated into future iterations of the Highway Capacity Manual, currently the two analysis methods available for practitioners are a planning level capacity-based analysis using FHWA's CAP-X program [131] and full traffic microsimulation. Many of these designs exhibit benefits to safety or access, but in practice these become secondary considerations, examined only after capacity and delay measures are satisfied. The opposite also holds true, that many of these designs have benefits to mobility but the right-of-way necessary to accommodate large intersection footprints or wide medians is not always available or cost effective. In evaluating the through-about design, our primary concern will be measures of delay, with secondary measures of effectiveness investigated as appropriate and available.

5.3.1 Analysis Methodology

Two software applications were employed to validate the findings included in this paper: the macroscopic based Synchro software, and the microsimulation based INTEGRATION software package. The standardized methodology from the Highway Capacity Manual [132] was used to validate the findings of microsimulation for the conventional signalized intersection using HCM-2000 methodologies as implemented in Synchro (version 6). Microsimulation results are an amalgamation of numerous simulation runs, varying the random seeds to achieve significant confidence in the values presented. Each volume/geometry combination was run for approximately 20 hours of simulated time, resulting in 95% confidence intervals on average delay of less than +/- 0.5 seconds for undersaturated traffic conditions.

The use of microsimulation software packages provides the opportunity to optimize signal timings in real time, however, in the interest of obtaining consistent and repeatable results the authors developed signal timing schemes for each volume combination and geometry. Critical lane analysis was used to develop both the natural cycle lengths, using Webster's method, and the signal splits for the various phase movements, in accordance with the Traffic Signal Timing Manual [9]. The natural cycle length was rounded to the nearest ten seconds, and allowed to vary between 60 and 140 seconds for the four-phase conventional intersection, and between 40

and 120 seconds for the two-phase through-about design. Signal timings were treated as pre-timed non-actuated signals, with a constant timing plan for the duration of the simulation. The modern roundabout design was analyzed as an unsignalized intersection under yield control.

5.3.2 Measures of Operational Effectiveness

While control delay is the operational standard by which conventional intersections are measured against each other, alternative intersections often include displaced or rerouted vehicle movements with increased travel length. The preferred methodology used in the literature to provide comparisons between alternative intersections is to hold the limits of the network size standard across each alternative design, and provide comparisons for total travel time [4]. As the delay measure is so critical for transportation engineering practitioners, the authors of this study chose to define a traffic network with origin-destination nodes located equidistant from the centroid of the intersection. In this case, each design was modeled with start/end points located 0.4 km away from the centroid of the intersection. Setting the desired free-flow speed to 40 km/h (approximately 25 mi/h), each of the twelve origin-destination movements travels 0.8 km, with a base theoretical travel time of 72 seconds. Using the average total travel time of vehicles in the network as the output of simulation, a measure of delay can then be used that incorporates both control delay as well as geometric delay for alternative designs.

5.3.3 Selection of Alternative Geometries for Comparison

The analysis conducted herein seeks to identify the potential of the through-about intersection design to serve as an alternative solution to a conventional signalized intersection or a modern roundabout design. Decisions about intersection geometry were made to provide as equal as possible a comparison between the alternative designs. A visual representation of the geometries simulated is shown below in **FIGURE 5-3**.



(a) Conventional



(b) Roundabout



(c) Through-about

FIGURE 5-3 Geometry of (a) conventional, (b) roundabout, and (c) through-about intersections

5.3.4 Definition of Origin-Destination Demands for Comparative Analysis

Three separate sensitivity analyses are presented herein. The first set of volume combinations varies the total volume on all approaches, while holding other factors constant. The second set varies the percentage of major approach vehicles turning left. The third set varies the percentage of total traffic on the major approach. The volume combinations used are based on engineering judgment.

In conducting sensitivity analysis on the magnitude of volume demands, the total volume on all approaches ranges from 1,200 vehicles (passenger car equivalents) to 4,200 vehicles, with a step size of 600 vehicles. The directional split between the major and minor approaches is 70%/30%. The major approach turn-movement demands are split with 25% left-turning movements, 65% through movements, and 10% right-turning movements. The minor approach turn-movement demands are split with 15% left-turning movements, 70% through movements, and 15% right-turning movements.

In conducting sensitivity analysis on the percentage of left-turning vehicles, the total volume demand on all approaches is held constant at 3,000 vehicles. The minor approach experiences the same turn-movement demand splits for the left-turn analysis as it does with the volume sensitivity analysis. The major approach turn-movement demands hold right-turning movements at 10%, while varying the left/through movements as 10%/80%, 20%/70%, 25%/65%, 30%/60%, and 40%/50%.

In conducting sensitivity analysis on the split between the major and minor approaches, the total volume demand on all approaches is again held constant at 3,000 vehicles. The major and minor approach turn-movement demands experience the same splits as in the volume magnitude analysis. The split between the major/minor approaches varies as 90%/10%, 80%/20%, 70%/30%, 60%/40%, and 50%/50%.

A summary of the volume scenarios used can be seen in **TABLE 5-1**.

TABLE 5-1 Volume scenarios used in sensitivity analysis of (a) volume magnitude, (b) left-turn percentage, and (c) major/minor split

(a) Volume Magnitude									
Volume Magnitude	Total Volume	Major				Minor			
		split	Left	Thru	Right	split	Left	Thru	Right
	1,200	70	25	65	10	30	15	70	15
	1,800	70	25	65	10	30	15	70	15
	2,400	70	25	65	10	30	15	70	15
	3,000	70	25	65	10	30	15	70	15
	3,600	70	25	65	10	30	15	70	15
	4,200	70	25	65	10	30	15	70	15

(b) Left-turn Percentage									
Left-turn Percentage	Total Volume	Major				Minor			
		split	Left	Thru	Right	split	Left	Thru	Right
	3,000	70	10	80	10	30	15	70	15
	3,000	70	20	70	10	30	15	70	15
	3,000	70	25	65	10	30	15	70	15
	3,000	70	30	60	10	30	15	70	15
	3,000	70	40	50	10	30	15	70	15

(c) Major/Minor Split									
Major/Minor Split	Total Volume	Major				Minor			
		split	Left	Thru	Right	split	Left	Thru	Right
	3,000	50	25	65	10	50	15	70	15
	3,000	60	25	65	10	40	15	70	15
	3,000	70	25	65	10	30	15	70	15
	3,000	80	25	65	10	20	15	70	15
	3,000	90	25	65	10	10	15	70	15

5.4 Research Results

For the volume combinations examined, in general, the roundabout was found to be the preferred design, performing significantly better than the other alternatives investigated, with the conventional intersection displaying slightly better operational performance than the through-about design at higher volumes. Results of the three sensitivity analyses are found in **FIGURES 5-4 to 5-6**.

The two-lane roundabout alternative in **FIGURE 5-4** is seen to impart significantly less average delay than the other alternatives prior to 3,600 total vehicles on all approaches, after which the roundabout design experiences oversaturated conditions. The results of microsimulation of the conventional signalized intersection track along with the anticipated results calculated using the macroscopic HCM methodology, but the microsimulation predicts an average vehicle delay of a few seconds more in each case. At lower volume thresholds, the through-about intersection performs slightly better than the conventional signalized intersection, with equal performance in the mid-range volumes and increasingly poorer performance in the high-volume regime.

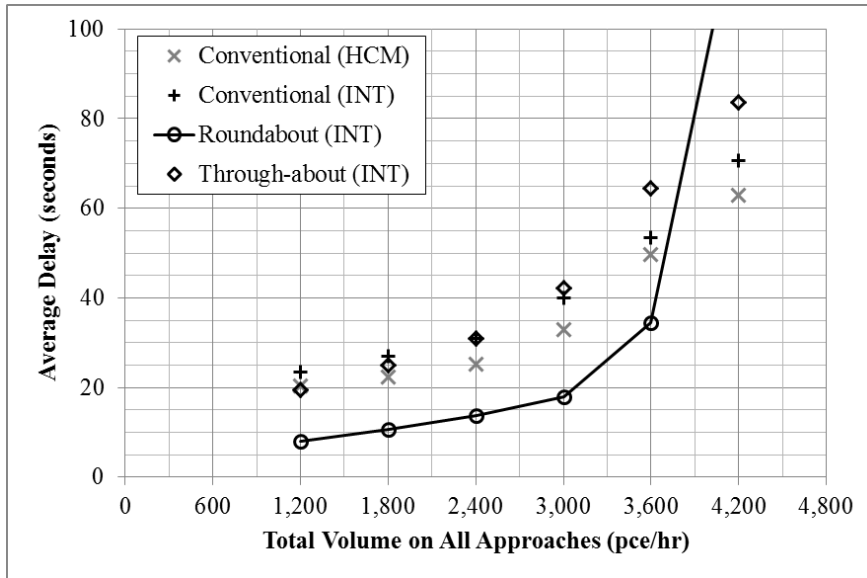


FIGURE 5-4 Average delay as a function of total volume on all approaches

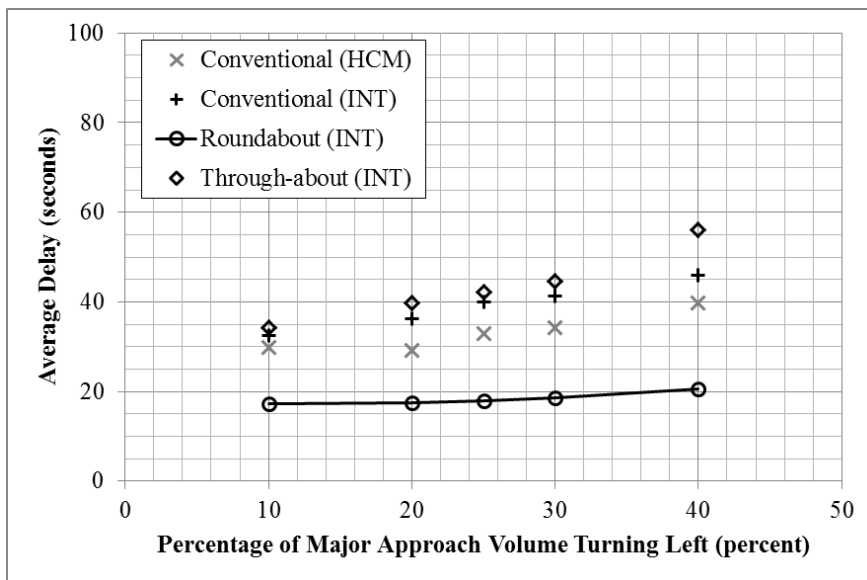


FIGURE 5-5 Average delay as a function of percent left-turning vehicles on major approach

The results shown in **FIGURE 5-5** examine the sensitivity of each design to the percentage of left-turning vehicles. The values displayed at 25% left-turns on the major approach in **FIGURE 5-5** are the base results included at the 3,000 total vehicles level in **FIGURE 5-4**. The findings of this research indicate that at this level of volume magnitude, the operational effectiveness of the roundabout design is largely unaffected by the percentage of left-turning vehicles. As the percentage of left-turning vehicles increases, both the conventional signalized intersection and the through-about design degrade in operational effectiveness, though the conventional intersection is seen to be less sensitive to the effects of left-turning vehicles than is the through-about.

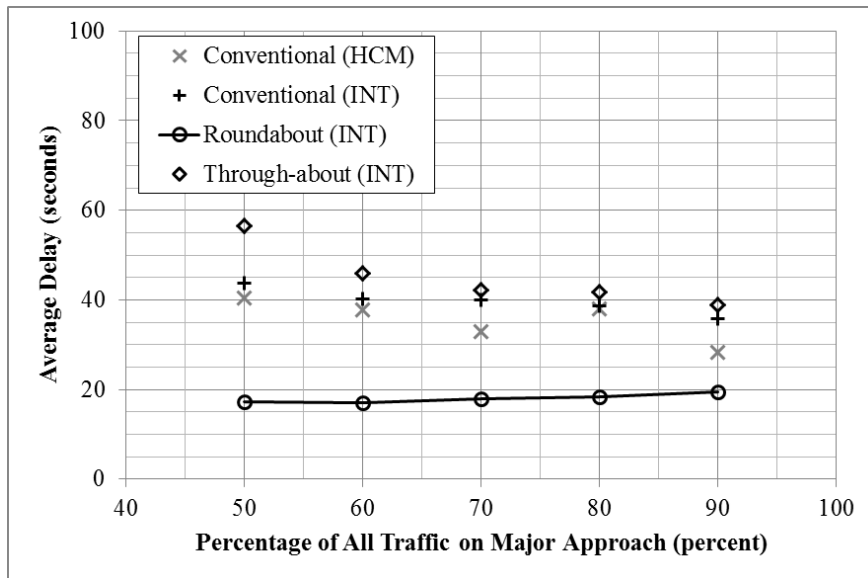


FIGURE 5-6 Average delay as a function of percent of total volume on major approach

The results shown in **FIGURE 5-6** examine the sensitivity of each design regarding the degree to which the major arterial experiences more traffic than the minor arterial. The findings of this research indicate that at this level of volume magnitude, the operational effectiveness of the roundabout design is largely unaffected by major/minor approach split. As the percentage of traffic on the major approach increases, both the conventional signalized intersection and the through-about design improve in operational effectiveness, though the through-about design slightly underperforms the conventional signalized intersection, and neither of these designs comes close to the operational effectiveness of the roundabout.

5.5 Analysis Of Results

Reviewing the three sensitivity analyses for the through-about design, there does not appear to be a niche condition under which this design is preferable, based on the volume combinations tested herein. Anecdotal evidence suggests that the existing through-about intersections in use domestically were constructed as a modification to an existing traffic circle, as through-vehicle traffic demands on the main arterial grew too large to be accommodated on the circulator [130]. This may suggest that specific volume combinations with high through-traffic volume on the major arterial, and low volume demands on the major approach turning movements and minor approaches may yield beneficial results.

While overall average delay experienced by vehicles at the through-about design appears to be greater than that of either the roundabout or the conventional signalized intersection, the reduction from four phases to two may reap benefits for this design on the average delay of through-movement vehicles on the major arterial. There are some cases where access is necessary for turning movements and minor approaches, but the maintenance of throughput on the major arterial takes precedence over the overall performance experienced by all vehicles at the intersection. As such, **FIGURE 5-7** seeks to determine if there's an advantage to the average

delay experienced by only the major approach through-movement vehicles when examining the results of the total volume sensitivity analysis. The results of this through-movement delay analysis indicate that the advantage of the modern roundabout design tails off as it approaches capacity before the conventional intersection or the through-about design, and that the through-about design consistently serves through-movements better than its conventional intersection counterpart. A question for designers might be the degree to which the through movements on the major arterial might take precedence over the total intersection operations at a given location.

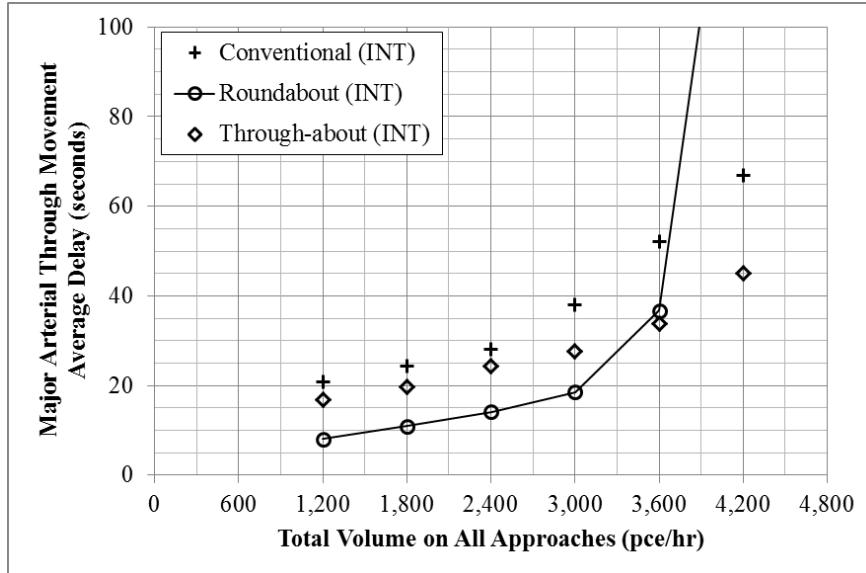


FIGURE 5-7 Through-movement delay on major arterial

5.5.1 Secondary Analysis Metrics

Though delay is the primary measure of effectiveness used in the comparison of potential improvements for an intersection facility, many other considerations come into play before a final selection is made. No increase in mobility can be justified if the safety of the roadway users is compromised by its construction, designs must be shown to be as safe as the other alternatives under consideration. The cost of each alternative may become a governing factor if the difference is significant; this is particularly true when there is insufficient right-of-way to construct a given design, and additional land must be purchased. Fuel consumption, while a secondary consideration, is also becoming a common metric for evaluation. A final metric that becomes important for the through-about design is the number of minor approaches intersecting at the location, and the relative skew of those approaches. The study results lead the authors to believe that the through-about may serve as an optimal solution in cases where a high-volume major arterial is joined by more than two low-volume approaches, and/or those approaches meet at a highly skewed angle.

One of the primary considerations which must be met when proposing facility improvements is that the safety of the location must not be compromised by any improvement constructed. Though the modern roundabout design provides significant safety improvements over the

conventional signalized intersection design, these improvements are arrived at due to the reduction in the total number of conflict points encountered by vehicles traversing the facility [8], and the reduction in severity of the existing conflicts. The modern roundabout design features only merging and diverging conflicts, with the majority of incidents involving property damage with little in the way of injuries or fatalities. The conventional signalized intersection has higher rates of accidents, injuries, and fatalities, due to the larger number of conflict points, and the fact that a number of these conflicts are the most dangerous crossing type of conflicts. An analysis of the conflict points for the multi-lane designs analyzed in this study can be seen in **FIGURE 5-8**. The conventional signalized intersection includes 48 total conflict points, of which 30 are the most-dangerous crossing conflicts. The dual-lane roundabout retains 16 crossing conflicts out of a total of 32 points of conflict. The through-about design shows a slight reduction from the conventional signalized intersection, with 40 total conflict points including 28 crossing conflicts.

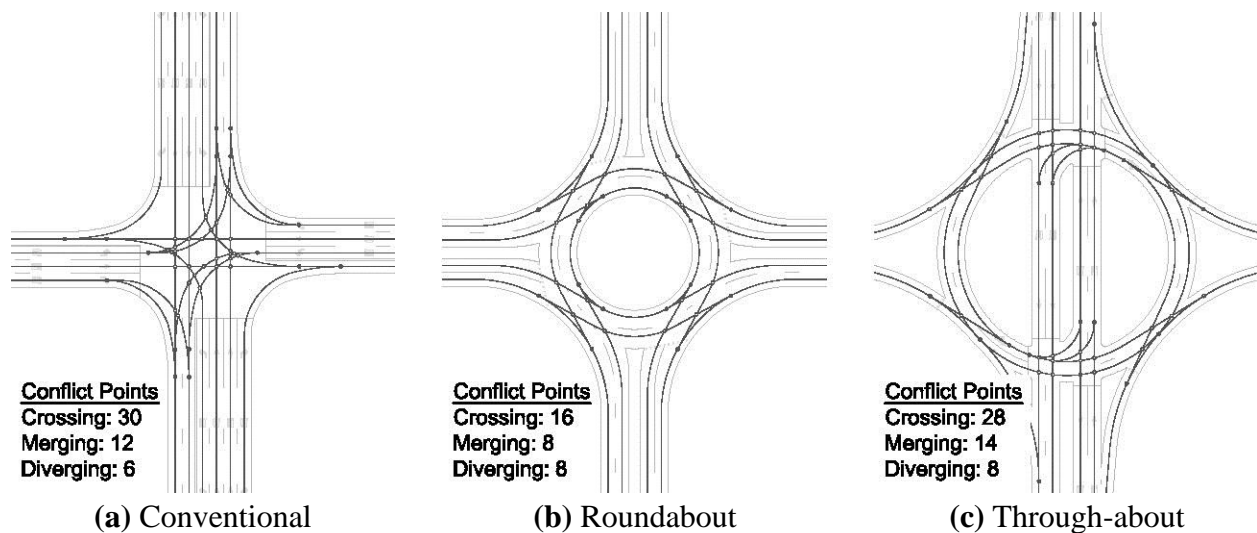


FIGURE 5-8 Conflicts created by (a) conventional signalized, (b) roundabout, and (c) through-about intersection designs

Four factors impacting cost for the alternative designs examined herein include the cost of purchasing right-of-way, the cost of constructing the footprint of the roadway geometry, the cost of constructing signalization and corresponding sensors, and the cost of maintenance for the signalization. These factors can be summarized as shown below in **TABLE 5-2**. Although the indicator variables shown are not directly correlated to the overall cost, they give a clear indication that right-of-way is an issue for the through-about design at the central intersection location, and that the modern roundabout design can be expected to incur the lowest total cost for construction. The total asphalt footprint is measured for a 1 kilometer diameter around the centroid of the intersection design, and includes a small paved shoulder beyond the edge of the travelway. These values are not taken to be constant for these designs, but are based on the

geometries used in this study and are meant to be indicative of the relative costs that can be expected.

TABLE 5-2 Cost indicators for alternatives analysis

Design	Diameter of Central Area (m)	Width of Major Approach (m)	Width of Minor Approach (m)	Total Asphalt Footprint (m ²)	Signal Masts
Conventional Signal	42.2	24.0	16.8	25,367	4
Roundabout	54.4	19.8	19.8	24,900	0
Through-about	70.8	20.0	12.8	27,364	6

Fuel consumption is becoming a more common secondary measure of effectiveness, and is likely to continue increasing in importance along with the cost of energy. The INTEGRATION traffic microsimulation software package interfaces with the VT-Micro fuel consumption model to provide measures of fuel consumption for the alternatives analyzed [133]. Making use of the total volume sensitivity analysis, **FIGURE 5-9** shows the relative fuel consumption of the alternative designs for the various volume combinations examined. The results of the fuel consumption analysis indicate that the modern roundabout outperforms the conventional and through-about designs until it approaches capacity, and that fuel consumption for the conventional intersection is consistently better than that for the through-about design, though not to a great degree.

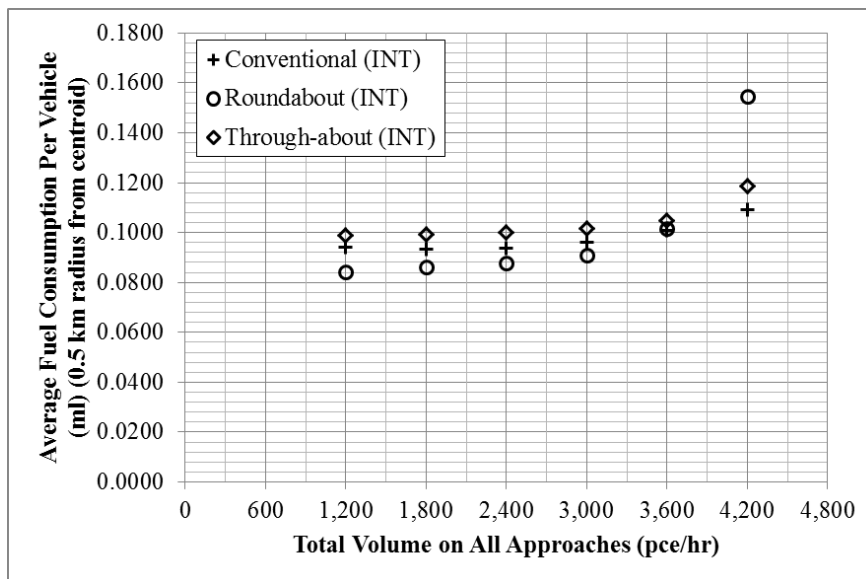


FIGURE 5-9 Fuel consumption measures from volume magnitude sensitivity analysis

The existing implementations of the through-about design domestically all have minor approaches with significant skew angles as they meet the major arterial. Two of the three locations additionally have more than two minor approach spokes at the junction, with the circulator road providing access to multiple points along the circumference. Combined with the microsimulation results of delay measures, the existing implementations as previously shown in **FIGURE 5-2** would indicate that this design is useful as a solution to a unique geometric condition, and not necessarily useful as a direct competitor to other alternative intersection designs.

5.6 Limitations of the Current Research

The authors believe that there exist specific conditions under which this design becomes the preferred choice for construction, but that the conditions necessary for this may be more related to site-specific space constraints than superiority in operational analysis. As an example, all of the domestic implementations of this design exist with large skew angles on the minor approaches, and two of the three locations have more than two minor approaches coming together at the junction. Additionally, there may be a set of turning-movement volumes which indicate the through-about as the appropriate choice from an operational perspective, but the volume combinations tested herein did not include such a scenario.

5.7 Conclusions and Recommendations for Future Research

Three sensitivity analyses were conducted for the research presented herein, with a primary focus on the average delay of vehicles. The first analysis on total volume of all approaches indicates that the roundabout intersection design outperforms the conventional signal and through-about designs up until the roundabout reaches capacity, at which point the conventional signalized intersection has a slight advantage over the through-about design. The second analysis on the percentage of left-turning vehicles indicates that the through-about design is more susceptible to performance degradation at high percentages of left-turning vehicles than is the conventional signalized design, but that the modern roundabout design outperforms both by a significant margin, and is relatively insensitive to the percentage of left-turning vehicles. The third analysis on the major/minor approach split resulted in very similar findings to the second - the through-about design is more susceptible to performance degradation at high levels of minor-approach traffic volumes than is the conventional signalized design, but that the modern roundabout design outperforms both by a significant margin, and is relatively insensitive to higher volumes of minor-street traffic.

Though the through-about intersection design was not able to show significant improvements over the alternatives in terms of overall intersection delay, a number of secondary measures of effectiveness were investigated. From a safety perspective the vehicle-path conflicts produced by the through-about intersection are anticipated to be on-par with those of a conventional signalized intersection, with the modern roundabout experiencing significant improvements to safety over both. From a cost perspective, the conventional signalized intersection is anticipated to need the least amount of right-of-way for construction, with the roundabout expected to have

the lowest long-term costs due to the lack of signals and corresponding maintenance – the through-about intersection requires significantly more right-of-way, and is likely to be considered only on the occasion of retrofitting an existing traffic circle or rotary installation. In terms of fuel consumption, the vehicle trajectories associated with the roundabout design provided significant benefits over either the conventional signalized intersection or the through-about design. Finally, it was discussed that a primary benefit of the through-about is its ability to safely accommodate a larger number of minor street approaches than the other designs, as well as approaches that have large skew angles.

The authors propose that future analysis of this alternative intersection design may be conducted using the Monte Carlo method of experiment design, randomly sampling from a wide variety of volume combinations to seek out the outlier conditions under which this design outperforms the competing geometries. Unfortunately, this type of analysis requires a widely-expanded set of volume combinations to achieve significant variety from the sample population, with a corresponding increase in computer processing time for traffic microsimulation software. The potential exists that sub-regimes of traffic may be pre-identified as potentially benefited by this design, with higher weights provided to volume combinations in this regime when selecting the random samples.

5.8 Acknowledgements

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Chapter 6. Conclusions from Planning-Level Analysis

Citation for original publication:

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Abstract

Capacity Analysis for the Planning of Junctions (CAP-X) is a planning-level tool for the comparative analysis of intersection designs developed by the Federal Highway Administration. This software is freely available to practitioners, but the practical impact of this tool remains smaller than its potential since it has not been officially documented in the literature, nor has it been embraced by any of the standards organizations such as the Highway Capacity and Quality of Service committee.

The research included in this paper seeks to document the formulation of equations for the critical sum method employed by the tool, and to demonstrate the operational limitations of a variety of intersection designs as predicted by this software tool. The authors find that the tool, while requiring validation, provides a highly functional planning-level analysis. In terms of highest capacity reached before failure conditions are experienced, the roundabout was found to be the least robust of the designs, with the restricted crossing u-turn performing slightly better, followed by comparable performance between the conventional signalized intersection, median u-turn, and Jughandle designs, with the displaced left-turn design consistently performing the best. Future research is recommended to validate the comparative analysis of intersections against the results of a higher-fidelity analysis methodology.

6.1 Introduction

Signalized intersections have been on our surface transportation network since the 1800s, but alternatives to the conventional signalized intersection have been slow to develop, and even slower to be embraced. The recent widespread adoption of roundabouts as a viable intersection alternative within the United States suggests that there may be support within the transportation engineering community to embrace a wider selection of options in intersection design.

Alternative intersections are a class of intersections which is generally designed to increase capacity and safety by reducing and separating the number of theoretical vehicle-path conflict points. By rerouting vehicles through alternate paths, these designs are able to reduce the number of signal phases at an intersection to increase arterial throughput. A corresponding negative impact is experienced as additional travel time for the rerouted vehicles. The most comprehensive report to date on the topic of alternative intersections is Federal Highway's Alternative Intersections/Interchanges Informational Report (AIIR) [4]. This document recognizes the time and money required to accurately model these alternative designs for comparative analysis, and advocates the application of a simplified capacity analysis to narrow the field of potential alternatives.

A number of different analysis methodologies have been used in the past for conducting comparative analysis on these designs. The critical sum method involves an analysis of capacity and is particularly useful during the planning/preliminary engineering stage of design. The Federal Highway Administration (FHWA) has been providing spreadsheet-based tools for the analysis of alternative intersections since 2009. Capacity Analysis for Planning of Junctions (CAP-X) [131] is the current version of the tool (Version 1.2, dated November 2011 used herein), replacing the Alternative Intersection Selection Tool (AIST) [87]. As part of the literature review process for this paper, the authors contacted FHWA requesting information on publications relating to the CAP-X tool, and were informed that no formal documentation existed at this time [134].

6.1.1 Purpose and Scope

The primary purpose of this paper is to examine implications for at-grade intersection design that result from the analysis of a large set of volume combinations run through the critical-sum methodology utilized by the CAP-X tool. The analysis herein is confined to at-grade intersections, as the authors' experience is that interchange improvement projects generally have analysis budgets sufficient to simulate alternative designs. Intersection improvement projects generally do not, relying on algorithmic analysis such as that documented in the Highway Capacity Manual [5]. While traffic simulation software is proficient at generating comparative analysis for these alternative designs, algorithmic analysis does not yet incorporate them.

Because the CAP-X tool is deterministic, a given set of volumes will always yield the same result for a defined geometric condition. As a result, it may be possible to run a large number of volume combinations through the application, yielding results related to the performance of each design, and its relative performance compared to other equivalent designs.

There remain many questions regarding the validity of the results produced by CAP-X, and the critical sum method in general. The authors intend to work toward answering these questions in a future study by conducting cross examination between the results of CAP-X and the results of microsimulation. As the results of microsimulation are highly dependent on the quality of the geometric design, as well as the signal system optimization, careful consideration is going into the design of this study.

6.1.2 Overview of Traffic Operations Analysis Methodologies

When analyzing the operations of traffic on a road network, two common methods of understanding the functionality of a facility are to examine the delay experienced by drivers at the location, or the capacity of the roadway in comparison to the corresponding demand experienced.

Delay is the standard by which specific intersections are typically deemed to be adequate or in failure. Usually provided in units of seconds per vehicle, it measures the amount of additional time that a given vehicle will take to travel from point A to point B, in comparison to if there had been no traffic control at a location, and no other vehicles on the road. Calculating the average delay manually is a fairly rigorous process, and it is standard practice to rely on software packages to generate these values, based on equations generated from research and incorporated into the HCM.

As an alternative measure of operations, capacity is related to delay but instead refers to the theoretical volume of traffic that can pass along a roadway, or through an intersection, during a given period of time. Capacity is usually presented in units of vehicles per hour, or vehicles per lane per hour. In contrast to calculating average delay, calculations of capacity at an intersection can be completed quickly without the aid of software, providing a planning-level analysis of design sufficiency.

6.1.3 Capacity Analysis for Planning of Junctions (CAP-X)

CAP-X utilizes the critical sum method [84] which calculates capacity at an intersection location by examining the combination of critical movements at the facility. The idea behind the calculation is that while a traffic signal services every turning movement demand at an intersection, only some of those demands are governing the signal timing, while the others just happen to occur simultaneously [84]. For example, on a commuter route with a high volume of traffic during the morning peak in the eastbound direction and a low volume in the westbound, the light remains green in both directions while the eastbound direction exhibits constant flow and the westbound direction only has a trickle of vehicles after the initial queue dispersal. It would not be possible to increase the capacity at the intersection with additional lanes on the approach with a low volume, because the high-volume approach is governing at the time. The critical sum method determines which movement is governing at any given time during a signal cycle, and sums the demands for each sequential critical movement, determining a total value for the demand at the intersection, in units of vehicles per hour per lane (veh/ln-hr).

The CAP-X software assesses six different intersection designs and five different interchange designs. The intersection designs included are the conventional signalized intersection, the quadrant roadway design, the displaced left-turn design, the restricted-crossing u-turn design, the median u-turn design, and the roundabout. The interchange designs include the traditional diamond interchange, the partial cloverleaf interchange (type A4), the displaced left-turn interchange, the double-crossover diamond interchange, and the single point urban interchange. Because the software allows for differentiation between the primary alignments of the various designs (east-west or north-south), in all there are twenty different intersection options and ten different interchange options.

The first step in conducting analysis with the CAP-X tool is to enter in the origin-destination demand volumes into the “Input Worksheet.” These take the form of vehicle/hour turning movement counts, with additional information provided including the percentage of trucks for each of the four directional approaches, the adjustment factors for saturated flow of u-turns, left-turns, and right-turns. The recommended default values for these parameters are 2% trucks, with adjustment factors of 0.80 for u-turns, 0.95 for left-turns, and 0.85 for right turns. The user may also choose to enter a failure-level value for the Critical Sum, with a default value of 1,600 veh/ln-hr. Comparative analysis within CAP-X relies on an assumption that all intersections reach failure at the same critical sum value, a simplification which may not hold true, given that a main feature of alternative intersections is the reduction of phases and associated loss-time.

With the turning movement (O-D) values entered, along with the truck percentages and saturation flow rate modifications, the next step is to visit each of the alternative designs in turn, which is computed on its own tab within the spreadsheet. The authors have observed that this software works well from two different approaches: either a practitioner may choose to enter an equivalent number of approach/auxiliary lanes for each design and perform comparative analysis across “equal” options, or they may instead choose to increase the size of each design until a desired level of service is reached, comparing the relative costs of “unequal” designs that perform equally well.

With the number of lanes for each turning movement group identified, the input portion of the CAP-X software application is complete, and the next step is to go to the “Results Worksheet,” where measures of relative demand/capacity are provided in a color-coded summary fashion. These results are based on capacity, and are intended to be predictive only of which intersections will or will not be in failure, not of which intersection will out-perform the others in terms of average intersection delay. Additionally, these values are representative only of the most critical location at an intersection or interchange, and are not indicating an average condition across the facility. Recognizing which intersection design options provide promising results, additional analysis can be performed on a subset of the total designs, resulting in cost-effective alternatives analysis.

6.2 Evaluation Approach

A number of decisions are required from a design of experiments perspective before an evaluation can be completed. These include: defining a method to generate sets of Origin-

Destination volumes; selecting alternative geometries for operational analysis and comparison; validating the equations used within the CAP-X software program; checking the applicability of the Critical Sum value as an indicator variable for average delay;

6.2.1 Definition of Origin-Destination Demands for Comparative Analysis

A relatively simple method was developed to systematically generate volume combinations for analysis, as shown in **TABLE 6-1**.

TABLE 6-1 Generation of volume combinations for analysis

Parameter	Units	Minimum	Maximum	Step	Values	Randomization
Major Street 2-way Volume	pceph	600	7000	200	33	$-100+[200*\text{Rand}(0,1)]$
Split Percentage on Major Street	%	0.5	0.7	0.05	5	$-0.025+[0.05*\text{Rand}(0,1)]$
Turn Percentage on Major Street	%	0.1	0.3	0.05	5	$-0.025+[0.05*\text{Rand}(0,1)]$
Minor Street 2-way Volume	pceph	600	6000	200	28*	$-100+[200*\text{Rand}(0,1)]$
Split Percentage on Minor Street	%	0.5	0.7	0.05	5	$-0.025+[0.05*\text{Rand}(0,1)]$
Turn Percentage on Minor Street	%	0.1	0.3	0.05	5	$-0.025+[0.05*\text{Rand}(0,1)]$

To limit the duplication of combinations, 2-way volumes on the minor road were held to levels at or below the 2-way volume of the major road, reducing the total number of potential volume combinations to 335,000.

6.2.2 Selection of Alternative Geometries for Comparison

Analyzing all possible volume combinations against all possible intersection and interchange designs would generate more information than could be communicated within this paper. Instead, the authors have chosen to focus on intersection designs, looking primarily at the volume regime where small/simple conventional intersections are beginning to break down, requiring the construction of additional through and/or auxiliary lanes. The specific intersection designs analyzed include the conventional signalized intersection, the roundabout, the jughandle, the median u-turn, the restricted crossing u-turn, and the displaced left-turn. Three different sizes of intersection were considered for each of the six designs, with an effort to make each design an “equivalent” option within a given size regime, as shown in **FIGURES 6-1 to 6-3**. By examining horizontally across intersection design size, and vertically across equivalent alternative designs, an understanding can be reached regarding potential intersection improvements, as design volumes are expected to exceed a given current geometric condition.

Careful consideration was paid to the number of auxiliary turn lanes across design types. In some cases two designs may not appear to be equivalent, because the auxiliary lanes from one approach have been reallocated as a u-turn bay, or an additional right-turn bay on a different approach.

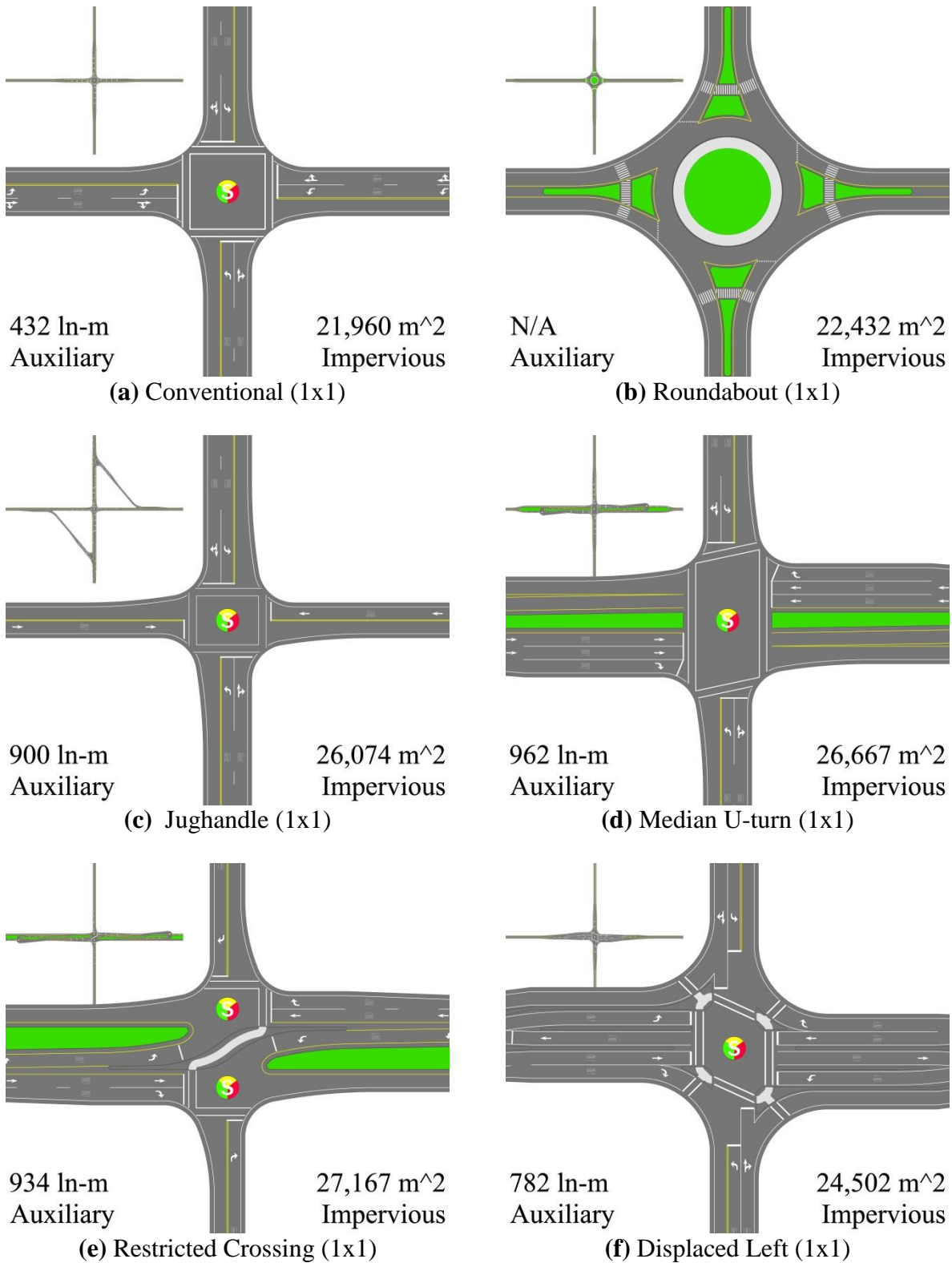


FIGURE 6-1 Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major road and one lane approaches on the minor roads

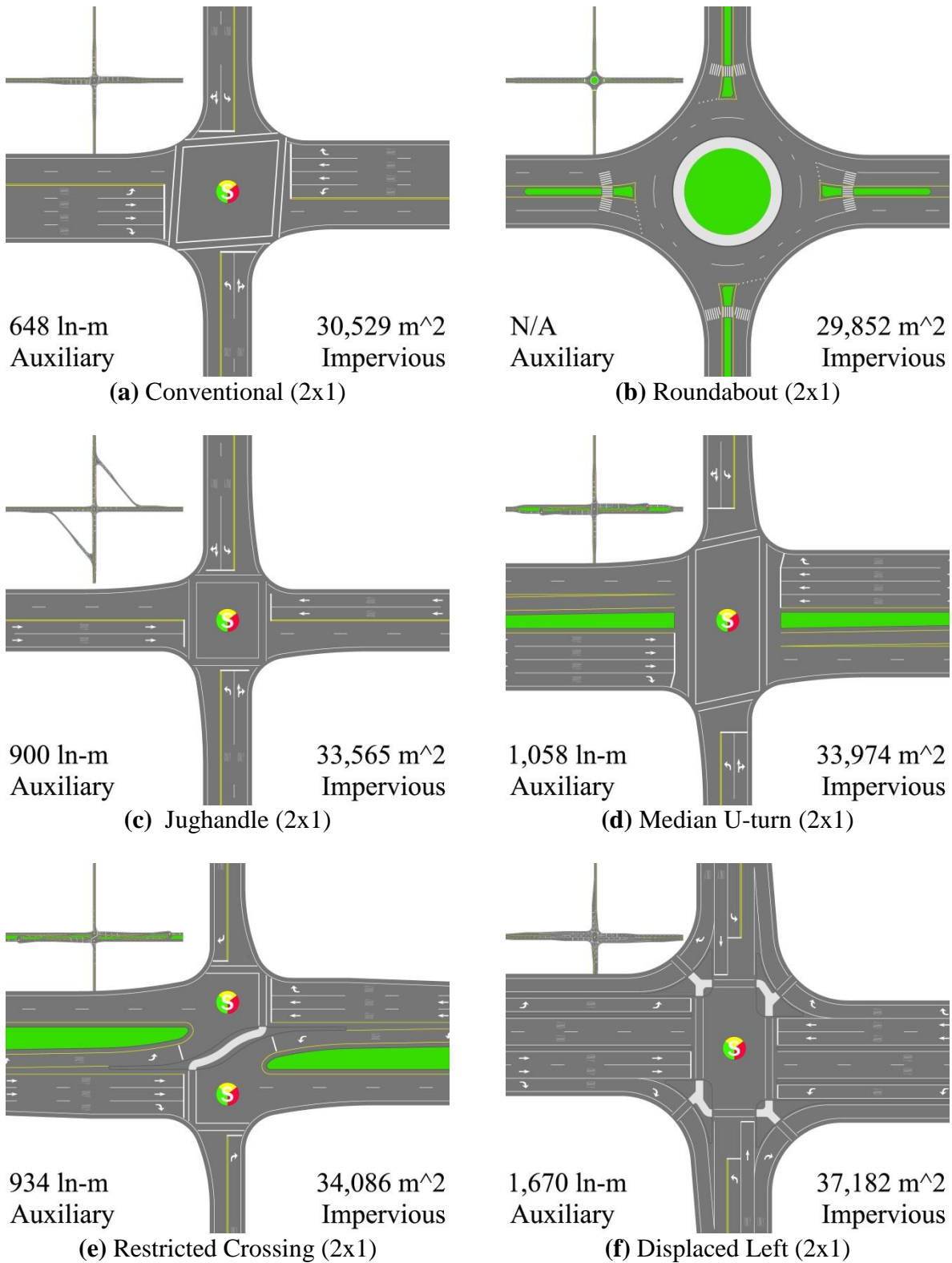


FIGURE 6-2 Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major road and one lane approaches on the minor roads

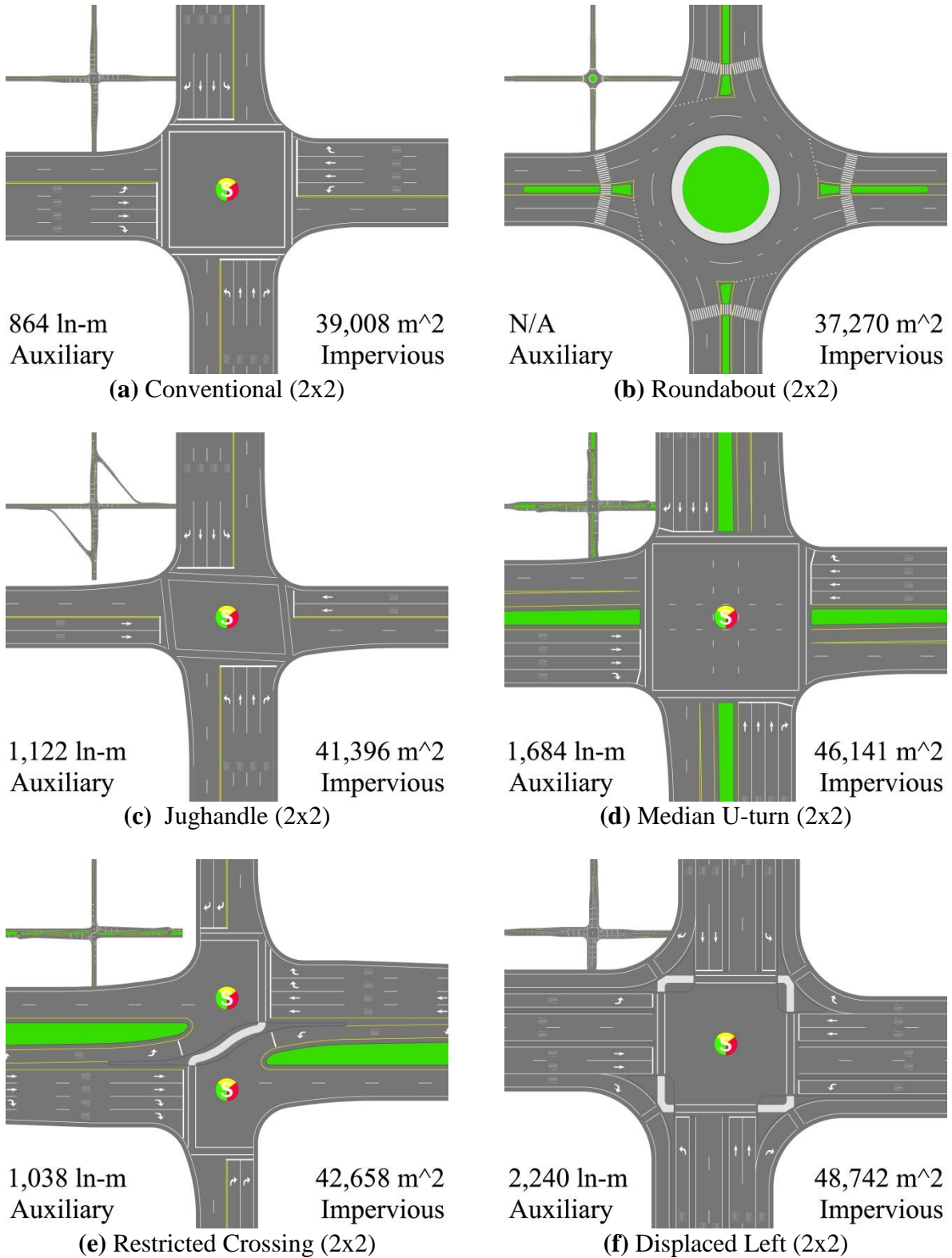


FIGURE 6-3 Lane configurations for typical intersection designs (a) through (f), as labeled, with two lane approaches on the major and minor roads

A detailed explanation of the geometry of each alternative design is outside of the scope of this paper, and readers are directed to the pertinent FHWA “Tech Brief” documents on each design, which are both informative and succinct [24], [27], [38], [46].

6.2.3 Documentation of Equation Formulations

The purpose of using capacity analysis instead of delay analysis is that capacity calculations can be completed at a preliminary design phase, without consideration of specific signal timings. Because this analysis occurs at a preliminary design stage, the equations used are necessarily based on the assumptions of the designer, and may vary from analysis to analysis.

In the case of this analysis, the desire for consistency in assumptions among the disparate designs has taken precedence over the desire to be compliant with the formulations presented in the CAP-X software program. For each formulation used herein, the O-D volumes used were taken to be passenger car equivalents per hour, incorporating percentage trucks on the front-end. Relative to saturation flow rate adjustments, turn movement factors of 0.80 for u-turns, 0.95 for left-turns, and 0.85 for right turns was used; additionally, a lane utilization factor of 0.95 was introduced both for multiple through lanes, and for multiple turn lanes.

The formulation of the conventional signalized intersection assumes that the left-turn phases are operating in protected mode, (an assumption common both to critical sum analysis and to the operations of alternative intersections). Additionally, an assumption is made that the transition time from opposing left-turn movements to adjacent through movements is allowed to vary – for example, while the eastbound left (EBL) and westbound left (WBL) movements are assumed to begin at the same time, the switch from EBL to westbound through (WBT) is not assumed to occur concurrently with the switch from WBL to eastbound through (EBT). Based on these assumptions, the formulations for Critical Sum analysis of conventional intersections are shown in equations 6-1, 6-2, and 6-3, depending on the number of approach lanes being considered, and the resulting number/combination of lanes available for each movement grouping.

$$\text{Critical Sum}_{\text{Conventional } (1x1)} = \text{MAX}[(\text{EBL}/0.95 + \text{WBT} + \text{WBR}/0.85), (\text{WBL}/0.95 + \text{EBT} + \text{EBR}/0.85)] + \text{MAX}[(\text{SBL}/0.95 + \text{NBT} + \text{NBR}/0.85), (\text{NBL}/0.95 + \text{SBT} + \text{SBR}/0.85)] \quad (6-1)$$

$$\text{Critical Sum}_{\text{Conventional } (2x1)} = \text{MAX}[(\text{EBL}/0.95 + \text{MAX}\{\text{WBT}/2/0.95, \text{WBR}/0.85 - \text{SBL}/0.95\}), (\text{WBL}/0.95 + \text{MAX}\{\text{EBT}/2/0.95, \text{EBR}/0.85 - \text{NBL}/0.95\})] + \text{MAX}[(\text{SBL}/0.95 + \text{NBT} + \text{NBR}/0.85), (\text{NBL}/0.95 + \text{SBT} + \text{SBR}/0.85)] \quad (6-2)$$

$$\text{Critical Sum}_{\text{Conventional } (2x2)} = \text{MAX}[(\text{EBL}/0.95 + \text{MAX}\{\text{WBT}/2/0.95, \text{WBR}/0.85 - \text{SBL}/0.95\}), (\text{WBL}/0.95 + \text{MAX}\{\text{EBT}/2/0.95, \text{EBR}/0.85 - \text{NBL}/0.95\})] + \text{MAX}[(\text{SBL}/0.95 + \text{MAX}\{\text{NBT}/2, \text{NBR}/0.85 - \text{WBL}/0.95\}), (\text{NBL}/0.95 + \text{MAX}\{\text{SBT}/2 + \text{SBR}/0.85 - \text{EBL}/0.95\})] \quad (6-3)$$

The formulation of capacity analysis for roundabouts is not based on the simplified critical sum method, which is calibrated for conventional turning movements, but is instead based on the equations and parameters presented in the most recent NCHRP report on roundabouts [19].

These equations are consistent with the approach taken by the CAP-X software, and can be found as equations 6-4, 6-5, and 6-6 for the three sizes of intersection considered, respectively.

$$\text{Critical Sum}_{\text{Roundabout } (1x1)} = 1800 * \text{MAX}\{(\text{EBL} + \text{EBT} + \text{EBR})/[\text{1130} * \text{EXP}(-0.001 * \{\text{SBT} + \text{SBL} + \text{WBL}\})], (\text{WBL} + \text{WBT} + \text{WBR})/[\text{1130} * \text{EXP}(-0.001 * \{\text{NBT} + \text{NBL} + \text{EBL}\})], (\text{SBL} + \text{SBT} + \text{SBR})/[\text{1130} * \text{EXP}(-0.001 * \{\text{WBT} + \text{WBL} + \text{NBL}\})], (\text{NBL} + \text{NBT} + \text{NBR})/[\text{1130} * \text{EXP}(-0.001 * \{\text{EBT} + \text{EBL} + \text{SBL}\})]\} \quad (6-4)$$

$$\text{Critical Sum}_{\text{Roundabout } (2x1)} = 1800 * \text{MAX}\{(\text{EBL} + \text{EBT} + \text{EBR})/2/[\text{1130} * \text{EXP}(-0.001 * \{\text{SBT} + \text{SBL} + \text{WBL}\})], (\text{WBL} + \text{WBT} + \text{WBR})/2/[\text{1130} * \text{EXP}(-0.001 * \{\text{NBT} + \text{NBL} + \text{EBL}\})], (\text{SBL} + \text{SBT} + \text{SBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{WBT} + \text{WBL} + \text{NBL}\})], (\text{NBL} + \text{NBT} + \text{NBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{EBT} + \text{EBL} + \text{SBL}\})]\} \quad (6-5)$$

$$\text{Critical Sum}_{\text{Roundabout } (2x2)} = 1800 * \text{MAX}\{(\text{EBL} + \text{EBT}/2)/[\text{1130} * \text{EXP}(-0.00075 * \{\text{SBT} + \text{SBL} + \text{WBL}\})], (\text{EBT}/2 + \text{EBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{SBT} + \text{SBL} + \text{WBL}\})], (\text{WBL} + \text{WBT}/2)/[\text{1130} * \text{EXP}(-0.00075 * \{\text{NBT} + \text{NBL} + \text{EBL}\})], (\text{WBT}/2 + \text{WBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{NBT} + \text{NBL} + \text{EBL}\})], (\text{SBL} + \text{SBT}/2)/[\text{1130} * \text{EXP}(-0.00075 * \{\text{WBT} + \text{WBL} + \text{NBL}\})], (\text{SBT}/2 + \text{SBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{WBT} + \text{WBL} + \text{NBL}\})], (\text{NBL} + \text{NBT}/2)/[\text{1130} * \text{EXP}(-0.00075 * \{\text{EBT} + \text{EBL} + \text{SBL}\})], (\text{NBT}/2 + \text{NBR})/[\text{1130} * \text{EXP}(-0.0007 * \{\text{EBT} + \text{EBL} + \text{SBL}\})]\} \quad (6-6)$$

The formulation of capacity equations for the Jughandle design takes the same set of assumptions discussed for the conventional intersection equations, and applies them to the modified geometry for this design. For the three different sizes of Jughandle design analyzed, the formulations are included herein as equation 6-7, 6-8, and 6-9, respectively.

$$\text{Critical Sum}_{\text{Jughandle } (1x1)} = \text{MAX}\{\text{MAX}(\text{EBT}, \text{WBT}) + \text{MAX}[\text{NBL}/0.95 + (\text{WBL} + \text{SBT} + \text{SBR}/0.85), \text{SBL}/0.95 + (\text{EBL} + \text{NBT} + \text{NBR}/0.85)], \text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{SBT} + \text{WBL}\}, \{\text{NBL} + \text{NBT} + \text{NBR}\}), (\text{EBR}/0.85 + \text{SBT} + \text{WBL})], \text{MAX}[\text{WBL}/0.95 + \text{MAX}(\{\text{NBT} + \text{EBL}\}, \{\text{SBL} + \text{SBT} + \text{SBR}\}), (\text{WBR}/0.85 + \text{NBT} + \text{EBL})]\} \quad (6-7)$$

$$\text{Critical Sum}_{\text{Jughandle } (2x1)} = \text{MAX}\{\text{MAX}(\text{EBT}/2/0.95, \text{WBT}/2/0.95) + \text{MAX}[\text{NBL}/0.95 + (\text{WBL} + \text{SBT} + \text{SBR}/0.85), \text{SBL}/0.95 + (\text{EBL} + \text{NBT} + \text{NBR}/0.85)], \text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{SBT} + \text{WBL}\}, \{\text{NBL} + \text{NBT} + \text{NBR}\}), (\text{EBR}/0.85 + \text{SBT} + \text{WBL})], \text{MAX}[\text{WBL}/0.95 + \text{MAX}(\{\text{NBT} + \text{EBL}\}, \{\text{SBL} + \text{SBT} + \text{SBR}\}), (\text{WBR}/0.85 + \text{NBT} + \text{EBL})]\} \quad (6-8)$$

$$\text{Critical Sum}_{\text{Jughandle } (2x2)} = \text{MAX}\{\text{MAX}(\text{EBT}/2/0.95, \text{WBT}/2/0.95) + \text{MAX}[\text{NBL}/0.95 + \text{MAX}(\{\text{WBL} + \text{SBT}\}/2/0.95, \text{SBR}/0.85), \text{SBL}/0.95 + \text{MAX}(\{\text{EBL} + \text{NBT}\}/2/0.95, \text{NBR}/0.85)], \text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{SBT} + \text{WBL}\}/2/0.95, \{\text{NBL} + \text{NBT} + \text{NBR}\}/2/0.95), (\text{EBR}/0.85 + \{\text{SBT} + \text{WBL}\}/2/0.95)], \text{MAX}[\text{WBL}/0.95 + \text{MAX}(\{\text{NBT} + \text{EBL}\}/2/0.95, \{\text{SBL} + \text{SBT} + \text{SBR}\}/2/0.95), (\text{WBR}/0.85 + \{\text{NBT} + \text{EBL}\}/2/0.95)]\} \quad (6-9)$$

The formulation of capacity equations for the median u-turn design takes the same set of assumptions discussed for the conventional intersection equations, and applies them to the modified geometry for this design. For the three different sizes of median u-turn design analyzed, the formulations are included herein as equation 6-10, 6-11, and 6-12, respectively.

$$\begin{aligned} \text{Critical Sum}_{\text{Median U-turn (1x1)}} = & \text{MAX}\{\text{MAX}[(\text{EBL} + \text{EBT} + \text{SBL}), (\text{EBR} + \text{WBL})/0.85, (\text{WBL} + \\ & \text{WBT} + \text{NBL}), (\text{WBR} + \text{EBL})/0.85] + \text{MAX}[\text{SBT}, (\text{SBR} + \text{SBL})/0.85, \text{NBT}, (\text{NBR} + \text{NBL})/ \\ & 0.85], [(\text{EBL} + \text{EBT} + \text{EBR}) + (\text{WBL} + \text{SBL})/0.80], [(\text{WBL} + \text{WBT} + \text{WBR}) + (\text{EBL} + \text{NBL})/0.80]\} \end{aligned} \quad (6-10)$$

$$\begin{aligned} \text{Critical Sum}_{\text{Median U-turn (2x1)}} = & \text{MAX}\{\text{MAX}[(\text{EBL} + \text{EBT} + \text{SBL})/2/0.95, (\text{EBR} + \text{WBL})/ \\ & 0.85, (\text{WBL} + \text{WBT} + \text{NBL})/2/0.95, (\text{WBR} + \text{EBL})/0.85] + \text{MAX}[\text{SBT}, (\text{SBR} + \\ & \text{SBL})/0.85, \text{NBT}, (\text{NBR} + \text{NBL})/0.85], [(\text{EBL} + \text{EBT} + \text{EBR})/2/0.95 + (\text{WBL} + \text{SBL})/0.80], [(\text{WBL} + \\ & \text{WBT} + \text{WBR})/2/0.95 + (\text{EBL} + \text{NBL})/0.80]\} \end{aligned} \quad (6-11)$$

$$\begin{aligned} \text{Critical Sum}_{\text{Median U-turn (2x2)}} = & \text{MAX}\{\text{MAX}[(\text{EBL} + \text{EBT} + \text{SBL})/2/0.95, (\text{EBR} + \text{WBL})/ \\ & 0.85, (\text{WBL} + \text{WBT} + \text{NBL})/2/0.95, (\text{WBR} + \text{EBL})/0.85] + \text{MAX}[\text{SBT}/2/0.95, (\text{SBR} + \text{SBL})/ \\ & 0.85, \text{NBT}/2/0.95, (\text{NBR} + \text{NBL})/0.85], [(\text{EBL} + \text{EBT} + \text{EBR})/2/0.95 + (\text{WBL} + \text{SBL})/2/0.95/ \\ & 0.80], [(\text{WBL} + \text{WBT} + \text{WBR})/2/0.95 + (\text{EBL} + \text{NBL})/2/0.95/0.80]\} \end{aligned} \quad (6-12)$$

The formulation of capacity equations for the restricted crossing u-turn design takes the same set of assumptions discussed for the conventional intersection equations, and applies them to the modified geometry for this design. For the three different sizes of restricted crossing u-turn design analyzed, the formulations are included herein as equation 6-13, 6-14, and 6-15, respectively.

$$\begin{aligned} \text{Critical Sum}_{\text{Restricted Crossing U-turn (1x1)}} = & \text{MAX}\{\text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{WBT} + \text{NBL}\}, \{\text{WBR} + \\ & \text{NBT}\})/0.85], (\text{SBL} + \text{SBT} + \text{SBR})/0.85 + (\text{WBT} + \text{NBL}), \text{MAX}[\text{WBL}/0.95 + \text{MAX}(\{\text{EBT} + \\ & \text{SBL}\}, \{\text{EBR} + \text{SBT}\})/0.85], (\text{NBL} + \text{NBT} + \text{NBR})/0.85 + (\text{EBT} + \text{SBL}), [(\text{EBL} + \text{EBT} + \text{EBR}) + \\ & (\text{WBL} + \text{SBT} + \text{SBL})/0.80], [(\text{WBL} + \text{WBT} + \text{WBR}) + (\text{EBL} + \text{NBT} + \text{NBL})/0.80]\} \end{aligned} \quad (6-13)$$

$$\begin{aligned} \text{Critical Sum}_{\text{Restricted Crossing U-turn (2x1)}} = & \text{MAX}\{\text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{WBT} + \text{NBL}\})/2/ \\ & 0.95, \{\text{WBR} + \text{NBT}\}/0.85], (\text{SBL} + \text{SBT} + \text{SBR})/0.85 + (\text{WBT} + \text{NBL})/2/0.95], \text{MAX}[\text{WBL}/0.95 + \\ & \text{MAX}(\{\text{EBT} + \text{SBL}\})/2/0.95, \{\text{EBR} + \text{SBT}\}/0.85], (\text{NBL} + \text{NBT} + \text{NBR})/0.85 + (\text{EBT} + \text{SBL})/2/ \\ & 0.95], [(\text{EBL} + \text{EBT} + \text{EBR})/2/0.95 + (\text{WBL} + \text{SBT} + \text{SBL})/0.80], [(\text{WBL} + \text{WBT} + \text{WBR})/2/0.95 + \\ & (\text{EBL} + \text{NBT} + \text{NBL})/0.80]\} \end{aligned} \quad (6-14)$$

$$\begin{aligned} \text{Critical Sum}_{\text{Restricted Crossing U-turn (2x2)}} = & \text{MAX}\{\text{MAX}[\text{EBL}/0.95 + \text{MAX}(\{\text{WBT} + \text{NBL}\})/2/ \\ & 0.95, \{\text{WBR} + \text{NBT}\}/0.85], (\text{SBL} + \text{SBT} + \text{SBR})/2/0.95/0.85 + (\text{WBT} + \text{NBL})/2/0.95], \text{MAX}[\text{WBL}/ \\ & 0.95 + \text{MAX}(\{\text{EBT} + \text{SBL}\})/2/0.95, \{\text{EBR} + \text{SBT}\}/0.85], (\text{NBL} + \text{NBT} + \text{NBR})/2/0.95/0.85 + \\ & (\text{EBT} + \text{SBL})/2/0.95], [(\text{EBL} + \text{EBT} + \text{EBR})/2/0.95 + (\text{WBL} + \text{SBT} + \text{SBL})/2/0.95/0.80], [(\text{WBL} + \\ & \text{WBT} + \text{WBR})/2/0.95 + (\text{EBL} + \text{NBT} + \text{NBL})/2/0.95/0.80]\} \end{aligned} \quad (6-15)$$

The formulation of capacity equations for the displaced left-turn design takes the same set of assumptions discussed for the conventional intersection equations, and applies them to the modified geometry for this design. For the three different sizes of displaced left-turn design analyzed, the formulations are included herein as equation 6-16, 6-17, and 6-18, respectively.

$$\text{Critical Sum}_{\text{Displaced Left-turn (1x1)}} = \text{MAX}\{\text{MAX}(\text{EBL}/0.95, \text{EBT} + \text{EBR}/0.85, \text{WBL}/0.95, \text{WBT} + \text{WBR}/0.85) + \text{MAX}(\text{SBL}/0.95 + \text{NBT} + \text{NBR}/0.85, \text{NBL}/0.95 + \text{SBT} + \text{SBR}/0.85), [\text{EBL}/0.95 + (\text{NBL} + \text{WBT} + \text{SBR})], [\text{WBL}/0.95 + (\text{SBL} + \text{EBT} + \text{NBR})]\} \quad (6-16)$$

$$\text{Critical Sum}_{\text{Displaced Left-turn (2x1)}} = \text{MAX}\{\text{MAX}(\text{EBL}/0.95, \text{EBT}/2/0.95, \text{EBR}/0.85, \text{WBL}/0.95, \text{WBT}/2/0.95, \text{WBR}/0.85) + \text{MAX}(\text{SBL}/0.95 + \text{NBT} + \text{NBR}/0.85, \text{NBL}/0.95 + \text{SBT} + \text{SBR}/0.85), [\text{EBL}/0.95 + (\text{NBL} + \text{WBT} + \text{SBR})/2/0.95], [\text{WBL}/0.95 + (\text{SBL} + \text{EBT} + \text{NBR})/2/0.95]\} \quad (6-17)$$

$$\text{Critical Sum}_{\text{Displaced Left-turn (2x2)}} = \text{MAX}\{\text{MAX}(\text{EBL}/0.95, \text{EBT}/2/0.95, \text{EBR}/0.85, \text{WBL}/0.95, \text{WBT}/2/0.95, \text{WBR}/0.85) + \text{MAX}(\text{NBL}/0.95, \text{NBT}/2/0.95, \text{NBR}/0.85, \text{SBL}/0.95, \text{SBT}/2/0.95, \text{SBR}/0.85), [\text{EBL}/0.95 + (\text{NBL} + \text{WBT} + \text{SBR})/2/0.95], [\text{WBL}/0.95 + (\text{SBL} + \text{EBT} + \text{NBR})/2/0.95], [\text{NBL}/0.95 + (\text{WBL} + \text{SBT} + \text{EBR})/2/0.95], [\text{SBL}/0.95 + (\text{EBL} + \text{NBT} + \text{WBR})/2/0.95]\} \quad (6-18)$$

6.2.4 Differences in Equation Formulation with CAP-X

There is currently no publication or documentation for the CAP-X software [134], and the authors would like to caution practitioners using the tool to back-check their own assumptions against the equations built into the program. The authors herein identify a few areas where the equations presented above specifically vary from the formulation used in the CAP-X software.

The software does not apply an adjustment factor for lane utilization, perhaps with the exception of multiple-lane roundabouts where the formulation of the equation comes from an NCHRP report [19]. This may be due to an assumption that intersections under analysis will be operating near capacity, with continuous queues that will impact the standard flow rate reduction seen from lane utilization. In the context of a conventional signalized intersection with shared through/right-turn lanes, the software does not apply adjustment factors for saturation flow rate to turning movements; additionally, the software does not allow variability/overlap in the end-time of opposing left-turn phases. The roundabout analysis in CAP-X calculates the v/c ratio based on the equations presented, multiplying this ratio by 1600 to generate an equivalent critical sum value. The authors preferred to use 1800 as the conversion factor, as this is the critical value assumed when the roundabout equations were generated [19]. The CAP-X software does not provide calculations for Jughandle designs, which is unfortunate given the potential usefulness of this design within the alternatives toolbox. For the median u-turn design, the major street through flow at the main intersection neglects to include the additional volumes added by the minor-street left turn movements which have been rerouted to utilize a downstream u-turn bay and pass back through the main intersection. For the restricted crossing u-turn design, there is a typo in the CAP-X formulation, where instead of comparing the left-movement against the

opposing right movement at the main intersection, the left movement is compared against the opposite right-turn movement – for example, when the authors check $\text{MAX}[(\text{WBT}+\text{NBL}),(\text{WBR}+\text{NBT})/0.85]$ near the beginning of the formulation, the software instead checks $\text{MAX}[(\text{WBT}+\text{NBL}),(\text{EBR}+\text{SBT})/0.85]$.

Some of the difference in equation formulation between the results presented herein and the results obtained from CAP-X are due to typos, or differences in conceptual approach to the intersection functionality. Of concern is that specific differences, such as the adjustment in saturation flow on turn-movements at the conventional intersection and ultimate capacity at the roundabout, will have significant impacts on the relative performance of those intersections compared with other designs, and may lead to inappropriate choices in the selection of alternatives as part of a preliminary engineering effort. In summary, it is recommended that practitioners desiring to make use of the CAP-X software should perform their own validation checks, to ensure that the software is interpreting the functionality of the proposed design in the same way that the designers are intending.

6.2.5 Use of the Critical Sum Value as an Indicator Variable for Average Intersection Delay

With the establishment of formulas to determine the capacity measures for these intersection designs, it is necessary to confirm that there is correlation between the measures of capacity generated and the corresponding expected delay measures. To assess this correlation a subset of 400 volume combinations with critical sum measures varying from 200 to 2000 veh/ln-hr were selected for analysis on the conventional intersection design, using both the 2x1 approach design, and the 2x2 approach design. These volume combinations along with the corresponding intersection geometries were modeled using the Synchro traffic simulation software to generate equivalent average delay measures according to the HCM procedure.

6.2.6 Development of Failure Thresholds for a Given Geometry Using the Critical Sum Method

Because capacity analysis is heavily reliant upon measuring whether or not a given design is in failure (experiencing oversaturated conditions), a main objective of the current research is to determine where that failure point can be expected to occur for the various intersection designs. With an exhaustive number of different ways to represent the data, a decision was made to follow the example of the signal warrant charts provided within the Manual on Uniform Traffic Control Devices (MUTCD) [7], an example being signal warrant number 3, the peak-hour warrant, found in MUTCD as Figure 4C-3. The x-axis for this chart displays the total volume of traffic on both approaches at the major road, with the y-axis displays the higher volume approach of the two approaches on the minor street, with the distinction between the major and minor roads being made based on volume. As parameters, these are not directly related to the generation of the critical sum value, and as such there is necessarily a fair amount of scatter in the results generated.

6.3 Research Results

The results of the research effort are presented below. Beginning with an initial assessment of the feasibility of using the critical sum method as an analogous parameter to average intersection delay, the failure criteria are identified and the failure thresholds are presented for the alternative geometries analyzed.

6.3.1 Correlation of the Critical Sum Values with Average Intersection Delay

A subset of 400 out of the 335,000 total volume combinations was chosen to be validated against the average intersection delay values generated by the Highway Capacity Manual. The volume combinations selected were chosen at random from those that generated critical sum measures of demand between two hundred and two thousand vehicles/ln-hr, when applied to the larger two of the three conventional intersection designs previously described herein. No data from alternative intersections was used to check validation, as these designs are not at this time integrated into either the HCM or corresponding software packages.

Viewing the results as **FIGURE 6-4** in chart format, the x-axis presents the critical sum demand calculated and the y-axis presents the corresponding measure of average intersection delay experienced for the same volume/geometry combination. The first view of the chart displays the results encountered if saturation flow-rate modifications are not applied to the equations of the critical sum method, with the second chart displaying the results when these adjustment factors have been applied. In addition to the rightward shift of the results along the x-axis, the authors had hypothesized that the application of adjustment factors for saturation flow rate would reduce the error/variability between the value of demand calculated with the critical sum method and the value of average intersection delay calculated, but this was not found to be the case. Using raw volumes, a critical sum value of 1500 was found to approximately define the separation between LOS D and LOS E, while this value shifts to 1600 in the modified case.

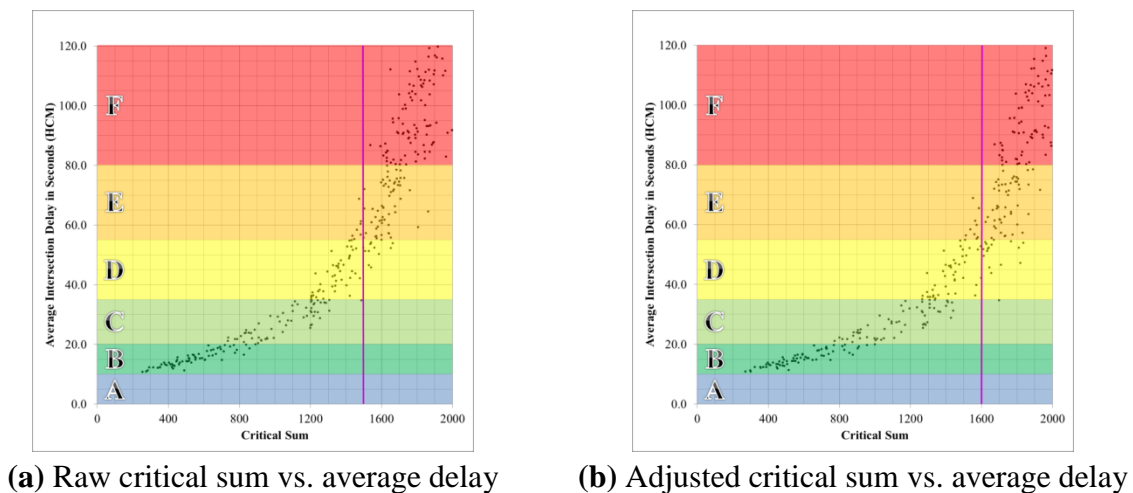
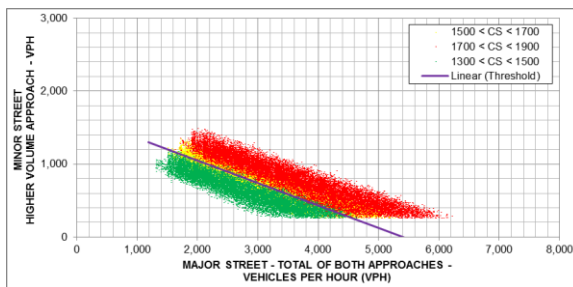


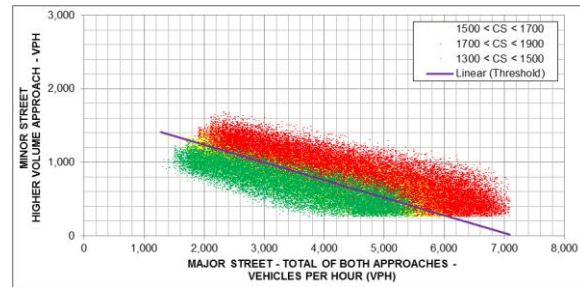
FIGURE 6-4 Correlation of critical sum values against the average intersection delay expected for a conventional intersection, (a) without and (b) with saturation flow rate adjustments

6.3.2 Failure Thresholds based on the Critical Sum Method

In applying the demand values calculated through the critical sum method to the axes of the MUTCD signal warrants, a great deal of scatter (error) was observed, as there are many contributing factors to determining the demand value beyond the summative approach volumes presented on the axes. For example, **FIGURE 6-5** demonstrates the range of volume combinations found to generate results within three ranges of critical sum demand. Values of demand, in units of vehicles/hour/lane, are assumed to indicate undersaturated conditions between 1300 and 1500. Values between 1700 and 1900 are assumed to indicate oversaturated conditions, and values between 1500 and 1700 are assumed to be at or near saturation flow.



(a) Conventional signalized intersection (2x1)



(c) Restricted crossing u-turn (2x1)

FIGURE 6-5 Sample applications of critical sum to the signal warrant framework, including a (a) conventional signalized intersection (2x1), and a (b) restricted crossing u-turn intersection (2x1)

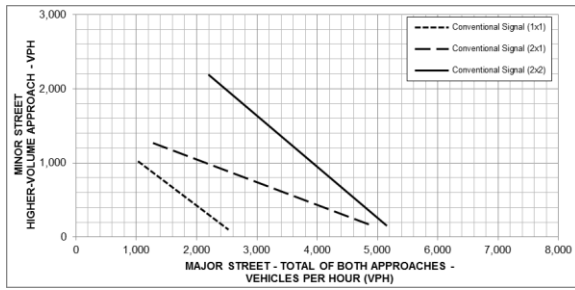
The scatter effect within the data resulted in a problem with identifying the threshold when the 1600 vehicles/hour/lane value was reached. In working with the data and examining the information represented, it was found that binning the data into groups with a range of 200 vehicles/hour/lane resulted in a clear picture of which volume combinations were reliably above and below the 1600 value. By optimizing a linear function that minimizes the number of observations in the 1300-1500 range that fall above the line, equal to the number of observations in the 1700-1900 range that fall below the line, it was possible to establish a threshold that was decidedly not less than 1600, and not greater than 1600. The resulting calibrated parameters can be seen below in **TABLE 6-2**.

TABLE 6-2 Equation parameters to generate failure thresholds, with validity measures

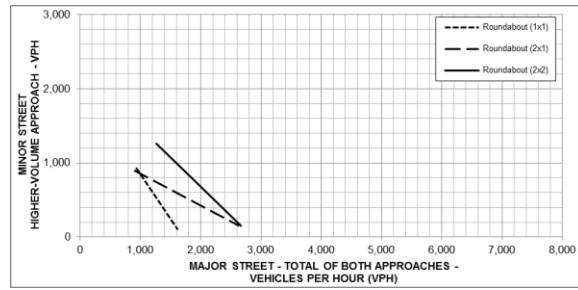
Number of lanes	Design	Hypothetical x-axis intercept (2-way flow on major road)	Hypothetical y-axis intercept (1-way flow on minor road)	Critical Sum 1300 < CS < 1500 Violating Observations / Total Observations	Critical Sum 1700 < CS < 1900 Violating Observations / Total Observations
One approach lane from each direction.	Conventional	2694	1659	101 / 4,057	101 / 5,821
	Roundabout	1707	2050	265 / 1,467	264 / 1,700
	Jughandle	3551	1624	835 / 6,263	836 / 8,914
	Median U	2523	2816	505 / 4,936	505 / 6,923
	Restricted U	2381	1521	209 / 2,871	209 / 4,251
	Displaced Left	3150	1693	640 / 5,395	639 / 7,672
Two approach lanes on the major road, one approach lane on the minor road.	Conventional	5408	1664	233 / 12,503	221 / 17,619
	Roundabout	2986	1288	333 / 2,864	333 / 3,241
	Jughandle	5239	1696	748 / 12,151	748 / 17,113
	Median U	4448	2717	1,491 / 13,137	1,491 / 17,864
	Restricted U	3900	1611	519 / 7,081	519 / 10,109
	Displaced Left	7139	1719	1,852 / 18,576	1,852 / 25,244
Two approach lanes from each direction.	Conventional	5378	3701	870 / 22,377	870 / 30,152
	Roundabout	2860	2251	703 / 4,033	703 / 4,476
	Jughandle	4799	3694	3,324 / 17,846	3,324 / 24,008
	Median U	4752	3801	3,305 / 17,846	3,305 / 24,008
	Restricted U	4458	2822	1,355 / 13,697	1,355 / 18,712
	Displaced Left	6018	5905	2,403 / 33,780	2,542 / 43,129

The information in **TABLE 6-2**, above, provides insight into the formulation of the threshold equations. The hypothetical x-axis intercept provides the maximum two-way volume along the main roadway approaches which can be accommodated by a given geometry, assuming that there is no volume of vehicles entering from the minor roadway. Alternatively, the hypothetical y-axis intercept provides the same information, except for the higher of the two minor approaches, with no volume on the major roadway. The remaining two columns in the table serve as measures both of the scatter inherent in applying this methodology to the given axes, and a check that the proscribed failure threshold is indeed a valid measure for each intersection design.

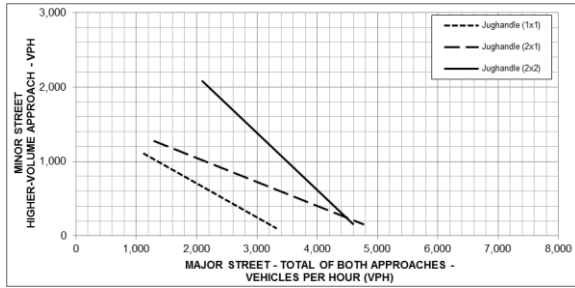
Reviewing this calibrated data in a graphical form, **FIGURE 6-6** demonstrates the calibrated failure threshold for each of the six alternative designs individually, comparing the thresholds across sizes of intersection instead of across types. The least robust geometry for each design represents a typical lane configuration for that design, given that there is only one lane of traffic on each approach coming to the junction. As a second a lane of traffic on the main roadway is added, while the minor street is kept with one lane approaching, the slope of the failure threshold line is seen to get flatter for each of the designs, as the major roadway becomes a constraining factor in fewer of the volume combinations. Finally, the failure threshold for the designs that have two lanes on all approaches can be seen to be generally parallel with the lowest level failure threshold, but shifted to the right to account for the greater levels of capacity afforded them.



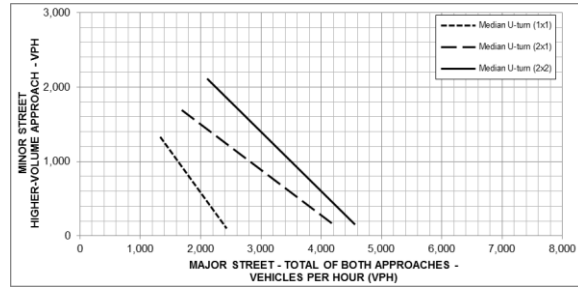
(a) Conventional Signalized Intersection



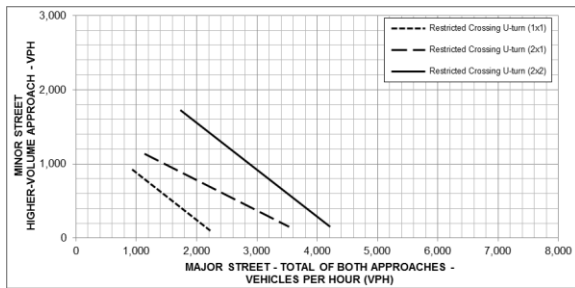
(b) Roundabout



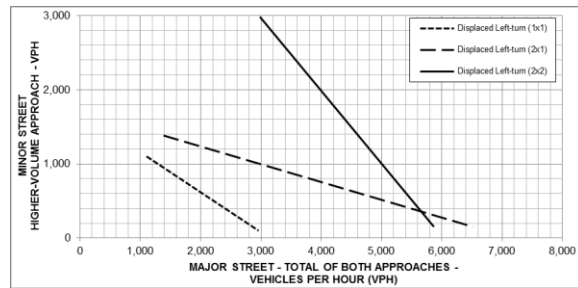
(c) Jughandle



(d) Median U-turn



(e) Restricted Crossing U-turn

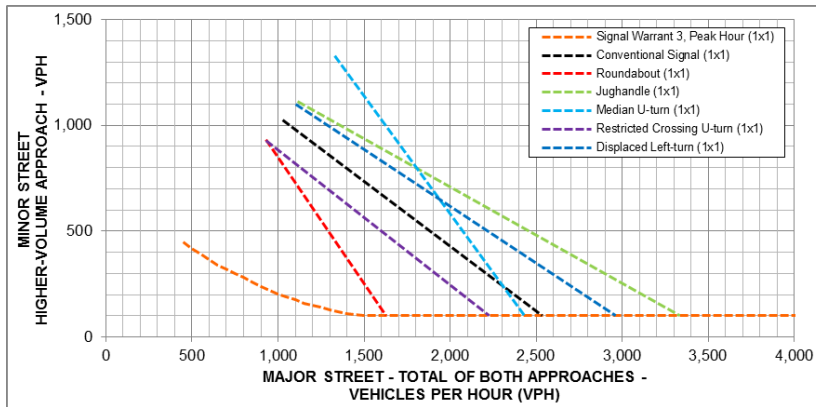


(f) Displaced Left-turn

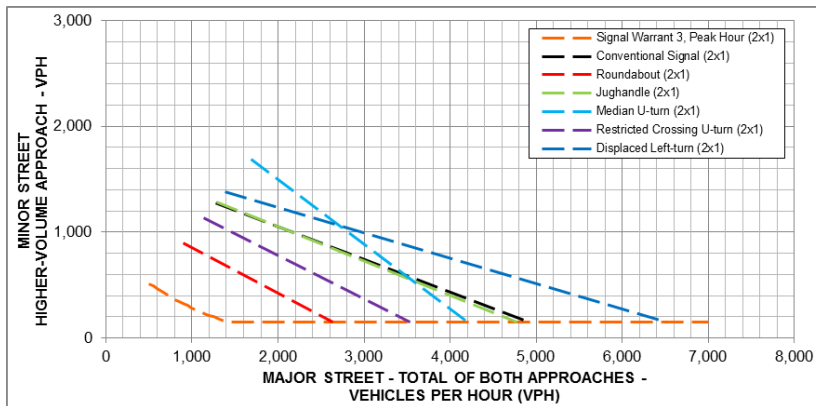
FIGURE 6-6 Maximum operational thresholds based on the critical sum method for (a) conventional signalized intersection, (b) roundabout, (c) jughandle, (d) median u-turn, (e) restricted crossing u-turn, and (f) displaced left-turn designs

6.4 Analysis Of Results

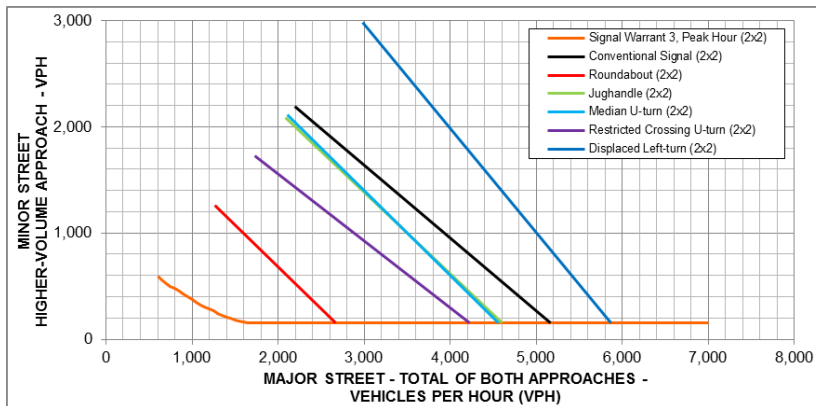
While the results provided an understanding of how capacity increases as size increases for each design, the ultimate goal for the practitioner is comparative analysis between comparable designs. Relative performance of each design within the same size category is shown in **FIGURE 6-7**. The charts indicate the failure threshold of each intersection design, the combination of major and minor street traffic that is expected to cause the design to enter into oversaturated conditions. The lines in orange indicate the peak-hour signal warrant volumes, and serves as a cut-off designation below which signalized intersection designs are not recommended.



(a) Capacity failure thresholds for one approach lane from each direction (note that scale is double that of charts (b) and (c) for readability purposes).



(b) Capacity failure thresholds for two approach lanes on the major road and one approach lane on the minor road.



(c) Capacity failure thresholds for two approach lanes in each direction.

FIGURE 6-7 Capacity failure thresholds for typical intersection designs with (a) one approach lane from each direction, (b) two approach lanes on the major roadway and one approach lane on the minor roadway, and (c) two approach lanes from each direction

Generally speaking, for the geometric designs with one approach lane from each direction, the roundabout was found to have the lowest failure threshold, followed by the restricted crossing u-turn, the conventional intersection, the median u-turn, and the displaced left-turn, with the Jughandle intersection design performing the best. The majority of failure thresholds are roughly parallel, or at least do not cross one another, with the exception of the median u-turn design, which fails more quickly than the conventional intersection with minor-road traffic approach volumes below 240 vehicles/hour/lane, and performs better than any other alternative with minor-road traffic approach volumes above 800 vehicles/hour/lane.

In the case of the intersection designs with two lanes on the major roadway approaches and one lane on the minor roadway approaches, the results are very similar, with some exceptions. Rather than outperforming all of the other intersections, the Jughandle design is now found to be functionally equivalent to a conventional intersection from a capacity standpoint. The median u-turn design continues to exhibit lower performance with low approach volumes on the minor roadway, and higher performance with higher volumes of minor roadway approach volumes.

With the addition of a second lane on the minor roadway approaches, there is no longer seen to be any crossing in switching in performance order among the alternatives. The roundabout continues to underperform the other intersections. The restricted crossing u-turn design continues to have the second lowest failure threshold. The Jughandle and median u-turn designs are now found to be functionally equivalent in terms of capacity. The conventional signalized intersection is expected to have the second highest failure threshold of the group, with the displaced left-turn intersection expected to outlast it by a fair margin.

6.5 Limitations of the Current Research

The CAP-X software tool is intended to aid practitioners in the planning stages of a project, to help narrow the field of potential alternatives on which to conduct detailed operational analysis. As such, the simplicity of use for the tool has been intentionally valued over the robustness of its results. The specific configuration of auxiliary turn lanes for a given design may not result in optimal functionality for a given set of turn-movement demands. More broadly, questions have been raised about the usefulness of this capacity-based planning tool for the analysis of alternative designs [135], given that the benchmarks for intersection performance have been set by the industry to be based on average intersection delay. By rerouting minor vehicle movements through the intersection, the capacity of the design is nearly always increased, while the benefit in travel times through the intersection, when there is a benefit, is often marginal. This research effort does not seek to corroborate the relative performance predicted in a comparative analysis against the same results observed from a more robust analysis tool such as microsimulation, but instead seeks to shed light on the very useful planning-level tool that is CAP-X. There has to-date not been any formal publication on the functionality of this software that the authors are aware of, nor has there been any documentation until now on the performance limitations of the various designs, as predicted by its capacity-based analysis methodology.

6.6 Conclusions and Recommendations for Future Research

Capacity-based software for planning-level analysis presents the potential to be very useful within a transportation engineering practitioner's set of tools. The concept of using the critical sum method, sometimes called the critical lane method, is one of the underpinning concepts at the heart of the HCM signalized intersections analysis methodology, and in its raw form serves as a productive first step towards identifying potential alternative design solutions.

A number of discrepancies were identified between the equation formulations presented within the CAP-X tool, and the assumptions for signal functionality assumed by the authors herein. Some of these errors appear to be due to typos in the software, while others are simple differences in interpretation of the design functionality. The formulations presented within CAP-X were noted to produce results that are more conservative for the conventional signalized intersection and roundabout designs, in comparison to the assumptions made herein. Practitioners are recommended to continue to use the tool provided, but ensure that the data being generated is validated against the assumptions of any given intersection geometric design.

The base method was found to be correlated fairly closely against the average delay values produced by the HCM method, at least for conventional signalized intersections. The addition of saturation flow rate adjustment factors for turning lanes was not able to reduce the amount of error observed in comparing the two measures, but it was observed to shift the correlation to the right by an appreciable amount. In defining the threshold at which an intersection ceases to function in an acceptable manner, the saturation-flow adjusted critical sum demands exhibited a transition from LOS D to LOS E at approximately 1600 vehicles/hour/lane, with an approximately equal number of observations below 1600 in LOS E, and above 1600 in LOS D.

The x- and y-axis defined for signal warrants were found to be useful conceptually for comparative analysis of intersection designs, but there was a great deal of scatter in the data produced, as these parameters do not take into account a number of factors that are important in critical sum analysis. The failure thresholds identified represent the point where a given volume combination has an equal chance of being above or below the 1600 value.

The slope of the failure thresholds for the geometric designs with one lane on each approach was observed to be generally parallel to the slope of the failure thresholds for geometric designs with two lanes on each approach. The condition with two lanes on the major roadway and one lane on the minor roadway was observed to consistently have a failure threshold with a shallower slope, indicating the intersections ability to serve higher volumes of major-street traffic flows, without improvements to service for minor-street traffic flows.

In examining the comparative analysis of failure thresholds for the various designs, the roundabout was found to consistently be the least robust, with the restricted-crossing intersection being the second-least robust. The Jughandle, median u-turn, and conventional intersections are seen to perform relatively equivalent in terms of capacity failure, while the displaced left-turn design is predicted by this method to have the highest.

There remain concerns about the use of capacity measures for comparative analysis of intersections, but that research remains for another effort to complete.

Chapter 7. Identifying the disconnect between planning and simulation analysis

Citation for original publication:

Sangster, J. D., & Rakha, H. A. (2012). Critique of the Critical Sum Method: A Case Study on the Quadrant Roadway Design. Transportation Research Board, 91st Annual Meeting of the. Washington, D.C.

Abstract

Though there is a growing body of work surrounding alternative or unconventional intersection designs, there is as yet no consensus regarding the appropriate methodology to use for performing comparative analysis of these designs. The typical software applications and measures of effectiveness used by transportation engineering consultants are not well suited to analyze these designs, which often involve the rerouting of vehicle movements. Using the Quadrant Roadway design as a case study, this paper investigates a variety of analysis methodologies, with particular focus on the Critical Sum Method which is currently being supported by the Federal Highway Administration.

Based on the case study of the Quadrant Roadway design, the Critical Sum Method (CSM) is found to be an unreliable predictor of either vehicle delay or total travel time. While the methodology does not require a great deal of time investment, it is unable to provide an accurate comparative prediction of performance, and it is recommended that CSM may be inappropriate even for preliminary analysis. The HCM methodology can provide indicative results in terms of delay, but cannot account for the additional travel time incurred by the rerouting of vehicles. Aggregate results obtained from microsimulation are recommended as the appropriate format for comparison purposes between alternative designs. Although this methodology is the most time-intensive, until a measure of effectiveness is developed which provides comparative results which are predictive of simulation results, there is no alternative analysis which is suitable.

7.1 Introduction

Over the past 25 years, many alternative intersection designs have been proposed, with the common goals of increasing capacity and safety, and with lingering driver expectancy issues. These designs generally function by reducing and separating the number of conflict points, which reduces the number of signal phases at an intersection to increase arterial throughput, all while putting drivers into new situations with unexpected movements and signal phasing patterns. To overcome the legitimate concern for drivers' reactions when encountering new traffic movements, it is important to thoroughly document when and where these designs are advantageous from a capacity perspective. The study of these alternative designs causes the lines to be blurred between practice and research, operations and theory. The analysis effort itself requires high-level microsimulation of unusual traffic flow patterns, while the results, in contrast, must focus on practice-ready measures of effectiveness if implementation of the design is to be pursued. A number of different analysis methodologies have been used in the past for conducting comparative analysis of these designs, though no analysis of the methods themselves has been conducted prior to this paper.

7.2 Unconventional Intersections

Though widespread implementation of Median U-turn facilities in Michigan and Jughandle facilities in New Jersey occurred during the 1960s, documentation of the decision/design process from the period is either not available, or does not exist. Kramer's advocacy of what is now known as the superstreet design in 1987[3] has been cited as the first paper in the modern literature to examine the benefits of unconventional intersections. With a focus primarily on improving congestion on arterials, a number of studies[10], [89], [93] have endeavored to compare a great variety of alternatives against each other to identify strong points and weak points of the designs. FHWA is actively supporting the expansion of unconventional intersections, with a number of TECH-Brief reports released on various designs[2], [26], [27], [38], [46], in addition to the seminal Alternative Intersections/Interchanges Informational Report (AIIR). As many of the unconventional designs increase the number of locations where merging/diverging occur, and some of them reroute vehicles through the intersection multiple times, it has been established that average travel times through the facility is a better measure of performance than the average delay per vehicle experienced at the central intersection node. Unfortunately, many design projects do not include the budget for an extensive microsimulation effort, and there exists a need for a low-cost methodology that is predictive of the more expensive analysis results. In applying the various analysis tools to unconventional intersections, it would be ideal to investigate all of the major alternative designs proposed. However, the scope of this research paper will be limited to a case study investigating the Quadrant Roadway design, chosen because it includes a high level of rerouting of vehicles, which some of the analysis methodologies do not account for.

7.2.1 Quadrant Roadway Intersection

First published in 2000[64], the Quadrant Roadway intersection design developed by Jonathan Reid achieves the goals of an alternative intersection: increasing safety and arterial throughput by separating conflict points and reducing the number of signal phases. For the purpose of this case study, the geometry used by the original study will be replicated for analysis. A unique aspect of the quadrant roadway design is that the three signalized intersections operate with a single controller, with each of the three signals sharing a three-phase split phasing plan. Upon observation of the simulated network, minor modifications were made to the signal phasing. As there are peculiarities with all of the unconventional designs, conclusions made based on results from the quadrant roadway design may not be applicable to all other specific intersection designs.

7.3 Analysis Methodologies

A variety of analysis methodologies are available to investigate signalized intersections, with more detailed analysis requiring significant investments in time. Some methodologies can be computed without the aid of a calculator and only provide a rough estimate of anticipated operation, while other methodologies require a great deal of calibration and validation of simulation results to provide high-fidelity information. A complication encountered when choosing a method for investigating unconventional intersections is the nature of the designs, which often modify a single signalized intersection into a system of coordinated signalized intersections. To compare the operations of a conventional design against unconventional alternatives, the increase in the number of intersection nodes invalidates a number of typical measures of effectiveness (MOEs) such as the average delay per vehicle or the v/c ratio for a given intersection.

7.3.1 Description of the Critical Sum Method

The Critical Sum Method (CSM) provides a basis for conducting capacity analysis at an intersection given the number of approach lanes and the turning movement volumes, irrespective of signal timing or lane saturation flow rate. First appearing in the literature in the 1970's[84], its intent was for the presentation of intersection operation to a layperson without first providing a background in traffic engineering. Though not in common use as an analysis tool, there are some locations, such as Maryland[86], where this procedure is used for both planning and design applications. Recently, the Federal Highway Administration placed its support behind the CSM in their publication of the Alternative Intersections/Interchanges Information Report (AIIR)[4], where they advocate use of a Microsoft Excel macro file titled the Alternative Intersections Selection Tool (AIST)[14].

7.3.2 Description of the Highway Capacity Manual Method

The Highway Capacity Manual (HCM) [132] contains the current industry standard methodology for analyzing signalized intersections. Chapter sixteen of the 2000 release of the HCM provides a detailed methodology for analyzing signalized intersections, including documentation of the theory behind the methodology and solved example problems applying the

methodology. The HCM methodology relies on a series of worksheets which reference various tables and appendices throughout the 160+ page chapter, which involves a time consuming process to work through by hand. In practice, a number of software packages are used which duplicate the procedures of the HCM, with Trafficware's Synchro software and McTrans' Highway Capacity Software being the two most commonly used within the industry. Analysis included in this report is conducted on data generated with Synchro (version 6, build 612), and is considered to be equivalent to the results produced directly from the HCM.

7.3.3 Description of the Aggregate Results of Microsimulation Methodology

From the beginning of the modern literature on unconventional intersections[88], the argument has been made that average travel time should be the primary MOE when comparing alternate designs. In an effort to concurrently increase safety and throughput, unconventional intersections reduce and separate the number of conflict points, which often results in the creation of a network of signalized or unsignalized intersections to replace a single conventional signalized intersection. By prohibiting specific movements such as left-turns at the central intersection, vehicles seeking to make minor movements are forced to reroute with increased travel times, while the primary through movements are afforded a greater percentage of green time, and a subsequent reduction in average travel times. Capacity is increased at the central intersection, but it is usually done at the expense of the minor movements. Taking a holistic view of the network in these situations is essential, rather than simply considering the delay experienced at a single node or location.

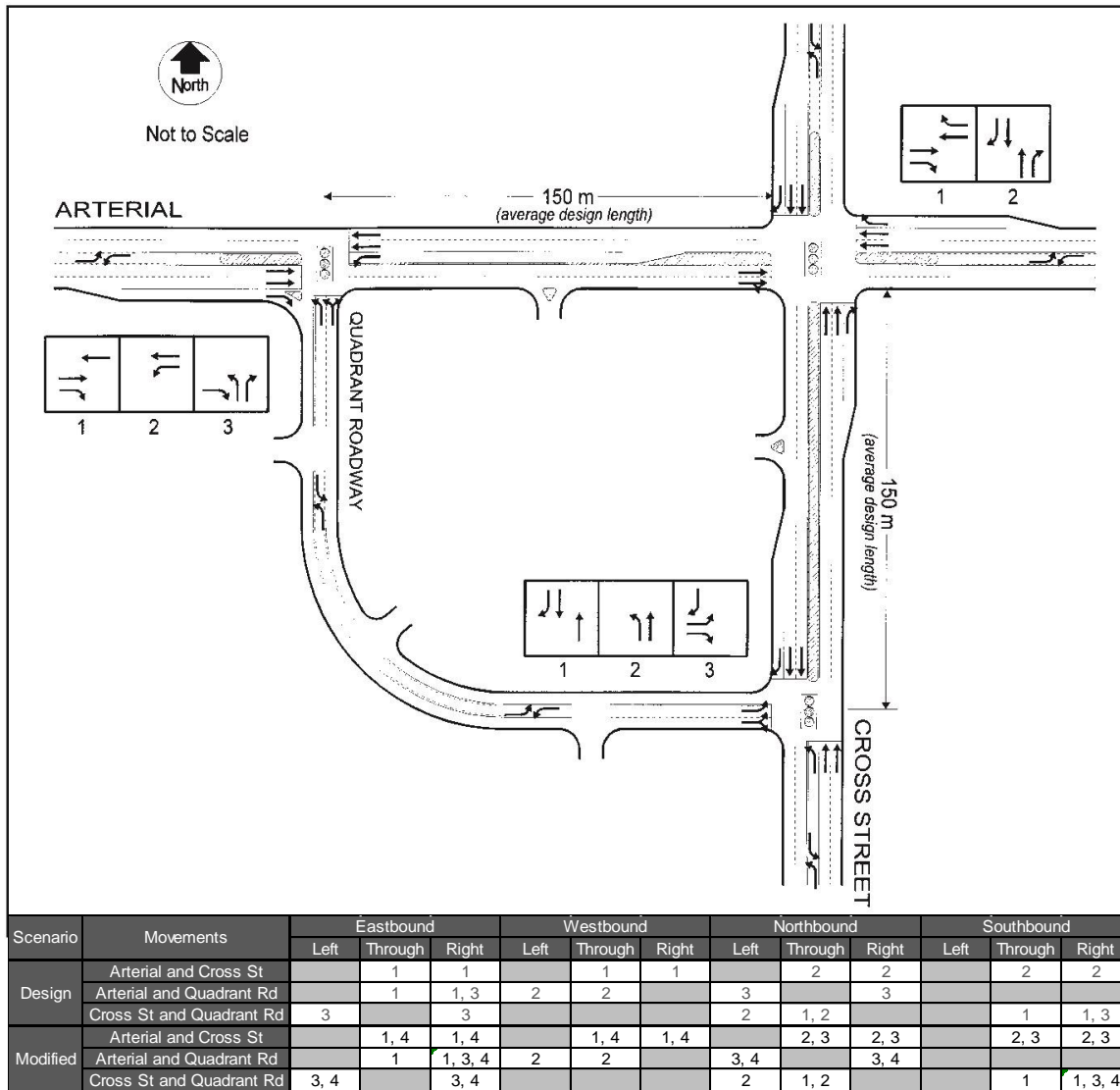
7.4 Evaluation Approach

The approach of this paper is to examine each methodology in order of increasing complexity, and then examine the durability of each approach relative to the level of effort involved in completing it. Should a simplistic approach give results that are, on the whole, predictive of the results given by complex analysis, then this simple approach should be pursued and tested for weaknesses. If, on the other hand, the results of simplistic methods are not indicative of the results obtained from methods requiring high investments in time, then the small amount of time spent to obtain the initial results is wasted, and these methodologies should be either abandoned or improved to generate better results.

7.4.1 Roadway Geometry and Vehicle Routing

As this study is not seeking to calibrate the optimal geometry of the Quadrant Roadway design, the geometry proposed by Reid will be duplicated for this analysis, as seen in **FIGURE 7-1**. One of the important factors to consider when comparing a conventional design against an unconventional alternative is to ensure that the designs be normalized with equal capacity, or number of lanes per approach to the intersection, to avoid giving one alternative an advantage by design. In the case of the quadrant roadway, there is an inequality between the conventional intersection and the main intersection replacing it, where the left-turn lanes are dropped. However, the capacity of these lanes is offset by the additional capacity provided by the spur road. Confirming the fairness of the two designs under consideration, a calculation of

impervious areas for the two roadway geometries generated for this report shows a value of 43,450 m² for the conventional intersection and 46,680 m² for the quadrant roadway, a modest



increase of 7%.

FIGURE 7-1 Geometric layout and signal phases of quadrant roadway intersection [64]

With the prohibition of left-turns at the intersection of the arterial and the cross street, vehicles which previously made this movement at the conventional intersection must be rerouted through the quadrant roadway. The rerouting of vehicles will follow the patterns shown in **FIGURE 7-2**, which come directly from Reid’s original design.

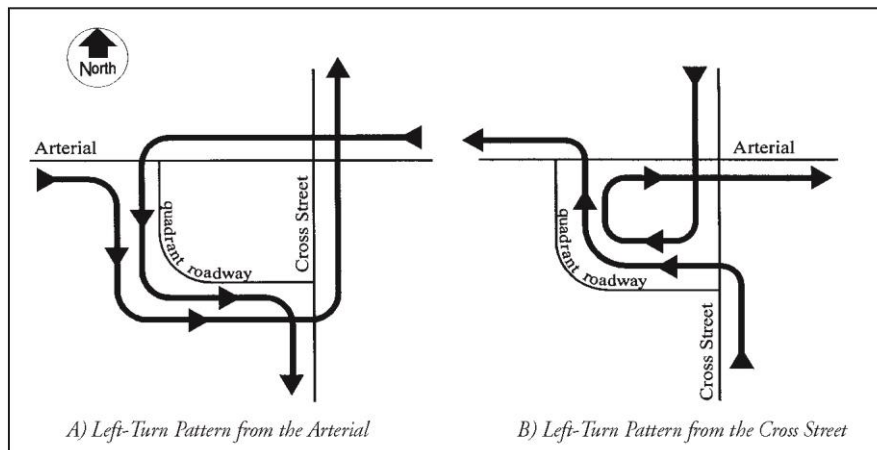


FIGURE 7-2 Pattern of left turns for quadrant roadway intersection [64]

7.4.2 Network Turning Movement Volumes

When comparing the traffic operations of an unconventional design against its conventional equivalent, it is important to conduct a comparison for a variety of volume regimes. In the absence of field collected volume data, it is commonplace to develop a factorial volume input design which iterates over a number of parameters. A total of 36 volume combinations were developed for analysis, based on the parameters, with a sample of how these parameters yield specific turning movement volumes shown in **TABLE 7-1**.

TABLE 7-1 Generation of volume combinations

(a) Parameters used to generate turning movement volumes

Parameter	λ_1	λ_2	λ_3
Major Link Two-Way Volume (vehicles/hr)	2600	2800	3000
Relative % Vol. on Minor Approach	0.7	0.8	0.9
% of Vol. Traveling Eastbound	0.55	0.65	-
% of Vol. in Each Turn Lane	0.1	0.2	-

(b) Sample turning movement volumes

Scn.	Major	Minor	Direction	Turn	INTID	E-W Street	N-S Street	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
20	2800	0.8	0.65	0.2	1	Arterial	Cross Street	364	1092	364	196	588	196	224	672	224	224	672	224
					12	Arterial Ave	Quadrant Rd		1092	728	196	812		224		224			
					11	Arterial Ave	Cross St		1316	0		784	196		1036	224		896	224
					13	Quadrant Rd	Cross St	364		560				224	896			672	224

7.4.3 Data Collection for Critical Sum Methodology

The calculation of the critical sum for each intersection is performed as a set of equations within a Microsoft Excel worksheet, according to the most simplified version of the methodology as described to operate the AIST[87], with adjustment factors set to 1. This method selects the highest vehicles/lane value among conflicting movements, summing these values across the intersection to calculate a maximum demand in terms of vehicles/hour/lane. The critical sum measure is analogous to the volume-to-capacity ratio commonly used as a measure of latent capacity in practice, though it overlooks differences in the saturation flow rate between lanes. Generally only the central node of an unconventional intersection design is used for CSM comparison purposes against its conventional alternative. A concern for the use of the CSM is the case where turning movements operate under permitted phasing, which is avoided for unconventional intersections, which are generally considered as an alternative only when left-turn volumes are high enough to operate under protected phasing within the signal timing plan. An advantage of the CSM is its independence of both roadway geometric design and signal timing plans, which are often unavailable in the planning stages of intersection design. The direct output in veh/ln/hr obtained by the CSM at the main node is used for data analysis herein.

7.4.4 Data Collection for Highway Capacity Manual Methodology

The calculation of HCM results is conducted by use of Synchro (version 6, build 612) traffic signal coordination software, a product of Trafficware Ltd. Signal phasing is based on modifications to the design by Reid, with the conventional intersection utilizing fully-actuated uncoordinated signal timing, and the quadrant roadway utilizing a single controller for the three signalized intersections, with all of the movements in the network accommodated by a 4-phase split timing plan. Turning movement volumes for the 36 alternative scenarios are loaded into the network by using the Universal Traffic Data Format (UTDF). Signal optimization is conducted through Synchro's internal signal optimization program. Reports including the HCM signalized intersection results and the signal timing plans are generated. Though there are many statistics generated from these reports, only the average vehicle delay per turning movement is used for data analysis. Prior to the formation of the network geometry for microsimulation analysis the worst-case volume scenario is identified, with predicted vehicle queue information produced. Auxiliary turn lanes are made long enough to accommodate the 95% queue length predicted by Synchro, to prevent spillback in the network modeled for microsimulation.

7.4.5 Data Collection for Aggregation of Microsimulation Results Methodology

Both the conventional intersection and the quadrant roadway network are drafted in AutoCAD to serve as a base for the creation of the VISSIM roadway network. Each of the 36 volume combinations was simulated for one hour, one run per volume combination. The output from microsimulation is in the form of aggregate network results. The eastbound and northbound approaches begin 600m away from the central intersection, with the westbound and southbound approaches beginning 450m away from the central intersection, allowing for 450m of seed length along each approach with the addition of the quadrant roadway into the network. The extent of

the impact of the signalized intersection on the approaches is 150m from the beginning of each roadway segment, which is predicted by Synchro analysis of the worst-case volume scenario to include the 95% queue length on each approach. The average delay and average travel time output values will be the variables of interest for data analysis. Screen captures of the simulation network for both the conventional and quadrant roadway designs are shown in **FIGURE 7-3**.

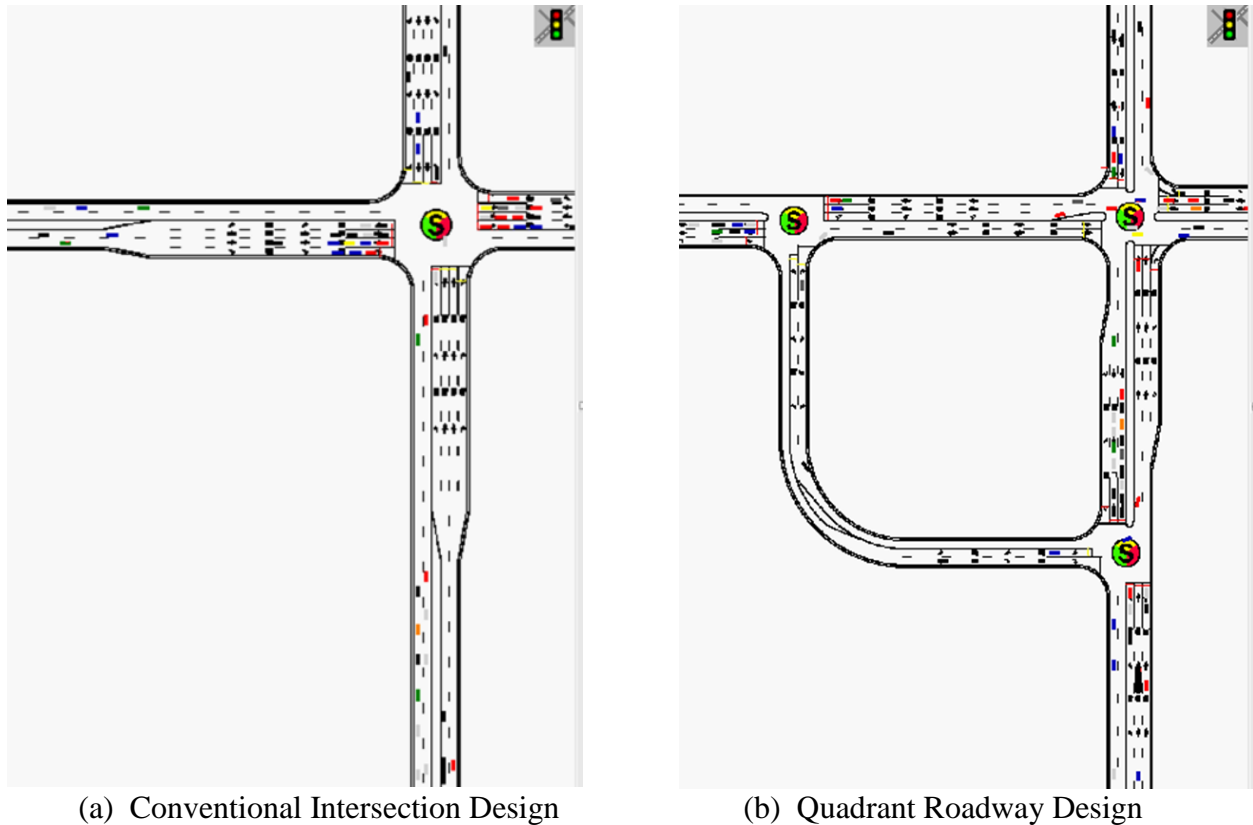


FIGURE 7-3 Screen captures of VISSIM roadway network simulation

7.5 Research Results

Research results are presented primarily in graphical form, with data used to present trends regarding the changes affected by converting a conventional intersection to a quadrant roadway design. The results of each methodology are presented individually in this section, with comparative analysis conducted in the Analysis of Results section of this report. In representing the graphical data, **FIGURES 7-4 to 7-8** show the MOE values of the conventional intersection on the x-axis, with the MOE values of quadrant roadway intersection on the y-axis, such that any data points falling below the 1:1 linear line indicate that the quadrant roadway design performs best, while data points falling above the 1:1 linear line indicate that the conventional roadway design performs best, for the given MOE.

7.5.1 Results from the Critical Sum Methodology

On average, the critical sum methodology predicts that converting a conventional intersection to a quadrant roadway will result in an increase of around 10% in terms of capacity, as seen in **FIGURE 7-4**. Examining the results by looking at specific parameters used to generate turning movement volume scenarios, it is found that the percentage of turning vehicles has the most statistically significant effect on the degree to which a quadrant roadway design is predicted to outperform a conventional design using the critical sum indicator variable. The low turn volume scenarios, with each turning movement comprising 10% of the upstream link volume (1/8 of the through volume), are predicted to produce minimal improvements compared with the improvements associated with the high volume scenarios, where each turning movement comprises 20% of the upstream link volume (1/3 of the through volume). The coefficient of determination for the predictive equation has a value of 0.812, indicating that there is a good deal of variability about the mean expected value, but that in general the results do indicate a trend consistent with the formulated equation.

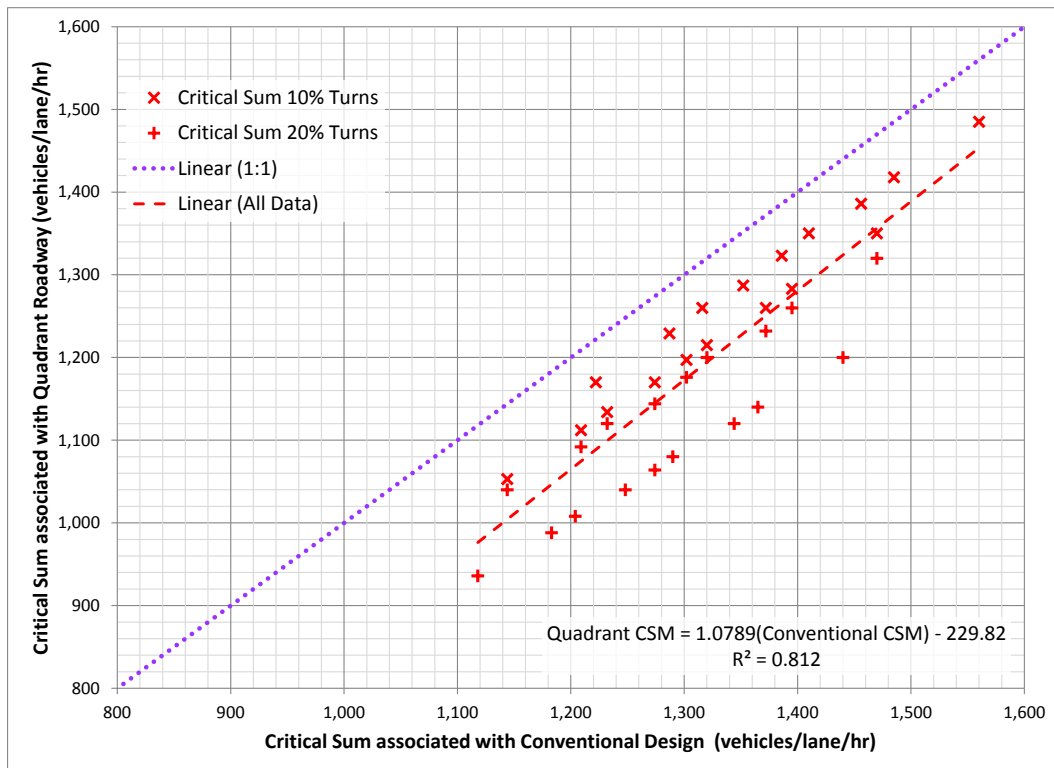


FIGURE 7-4 Change in critical sum due to the quadrant roadway design

7.5.2 Results from the Highway Capacity Manual Methodology

While the results of modeling the network with Synchro provide a great number of statistics, the primary concern is in comparing the average delay imposed on the network, which can be obtained by summing the individual movement delays at each of the three intersections of the quadrant roadway design, as seen in an example in **TABLE 7-2**. The primary limitation of this result is that it disregards the change in distance traveled for a given origin-destination pair.

TABLE 7-2 Sample Highway Capacity Manual method delay calculations

	INTID	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	Intersection
Scn.														
20	1	61.4	54.4	18.6	92.4	46.3	24.3	87.6	52.6	25.7	87.6	52.6	16.9	51.21
		62.6	57.9	40.1	72.5	25.4	11.3	59.1	28.4	22.6	88.7	38.2	24.9	43.74
	12		36.4	8.7	27.9	12.2		21.4		11.3				21.00
	11		21.5			13.2	11.3		21.1	15.3		16.4	12.7	17.90
	13	32.8		31.4				37.7	7.3			21.8	2.3	20.30

A comparison of the average delay experienced across the two networks is seen in **FIGURE 7-5**. The data indicates that the quadrant roadway reduces average delay in nearly all volume combinations investigated, with a general trend that the reduction in delay is greater as the overall delay increases. In contrast to the results predicted by the critical sum method, the HCM method predicts that the reduction in delay is independent of the percentage of turning vehicles. The coefficient of determination for the predictive equation has a value of 0.764, indicating that there is a good deal of variability about the mean expected value, but that in general the results do indicate a trend consistent with the formulated equation. The single point above 80 seconds of intersection delay for the conventional intersection appears to be an outlier for the linear relationship shown, which indicates that a regime of scenarios at higher volumes should be added to the analysis to determine whether the advantage of the quadrant roadway continues to increase as demand exceeds capacity, or whether very high levels of demand overwhelm the quadrant roadway in ways that the conventional roadway can accommodate.

A comparison of the turn-movement delay experienced across the two networks is seen in **FIGURE 7-6**. The data indicates that the quadrant roadway reduces average delay in nearly all volume combinations investigated, but that it is particularly helpful for through movements, to the detriment of some left-turn movements. As with the intersection average delay, the percentage of turning vehicles appears to be independent of the reduction in delay for specific turn-movements. The dispersion pattern of the graph indicates that it may be more difficult to predict the outcome of a specific design as the demand volumes increase beyond capacity.

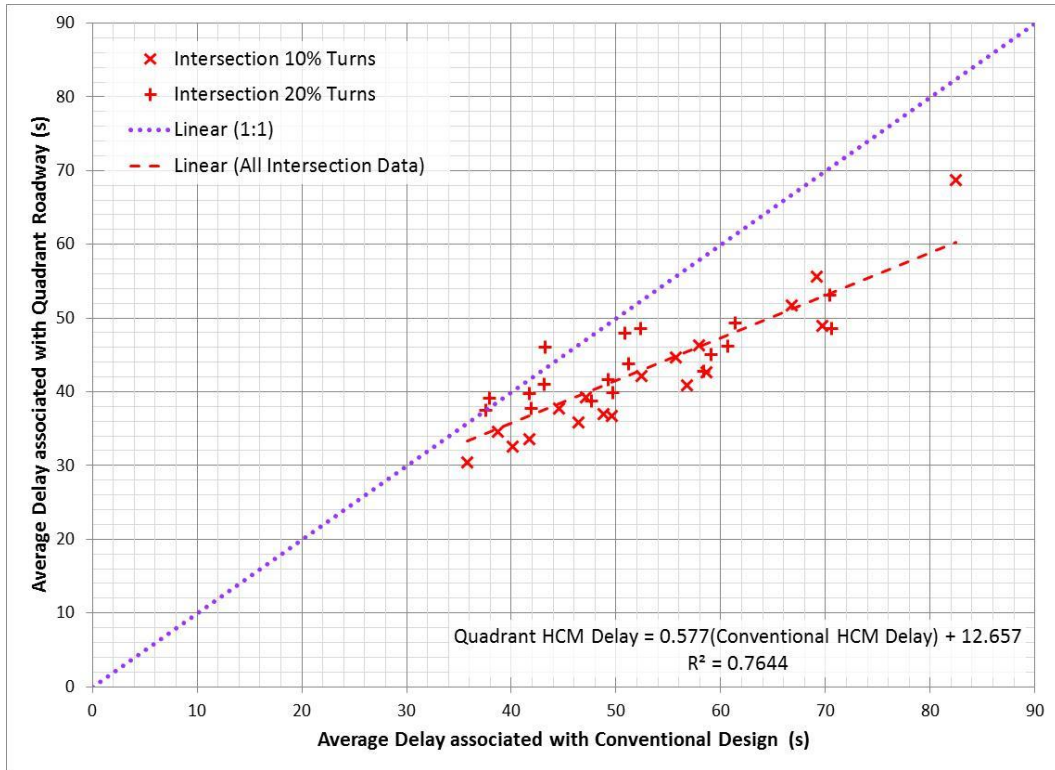


FIGURE 7-5 Change in intersection delay (HCM) due to quadrant roadway design

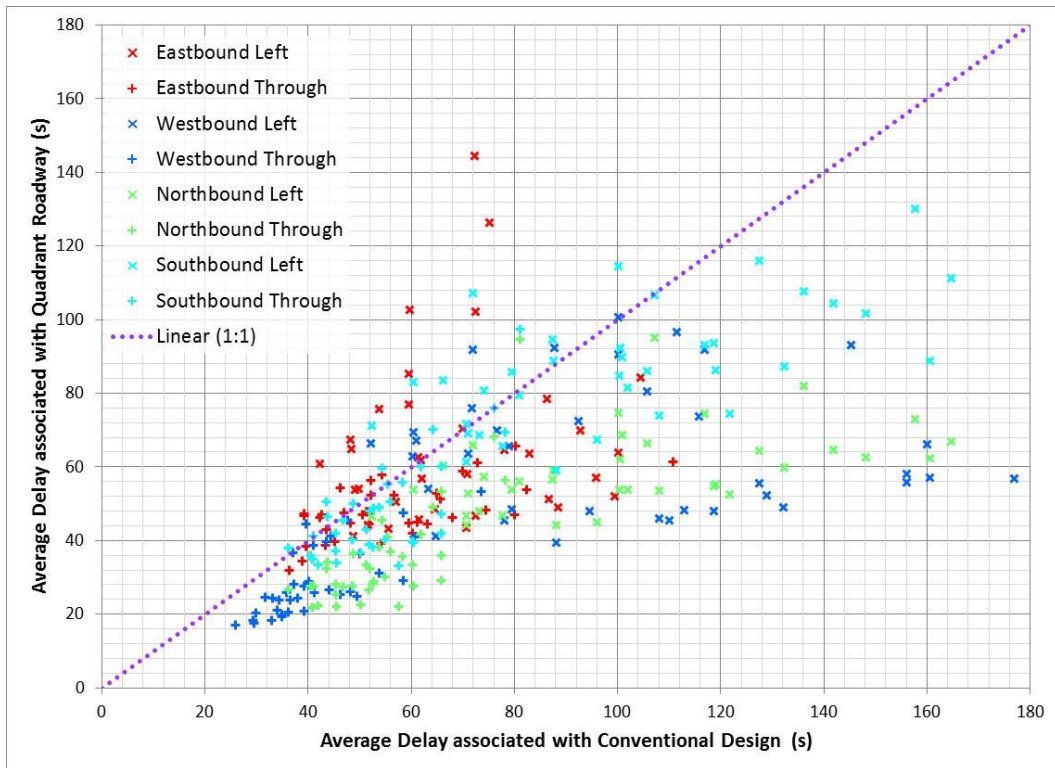


FIGURE 7-6 Change in turn-movement delay (HCM) due to quadrant roadway design

7.5.3 Results from the Aggregation of Microsimulation Results Methodology

Aggregate results were obtained for the alternative roadway networks on a network-wide basis. While the measure of average delay may be unaffected by the network method of data collection, an argument can be made that the travel-time relationship between the alternative designs is biased from the length of the roadway segments which remain unchanged, i.e., the longer the lead-in roadway lengths are the less aggregate difference there will be between designs.

A comparison of the average delay experienced across the two networks as predicted by microsimulation is seen in **FIGURE 7-7**. The data indicates that the quadrant roadway is anticipated to reduce average delay for the high-volume combinations, with a neutral effect in low-volume combinations. The coefficient of determination for the predictive equation has a value of 0.419, indicating that there is a great deal of variability about the mean expected value, and while the relationship does appear to be linear from the graph, the variability may be such that delay cannot be used as a statistically significant predictor for which alternative is better.

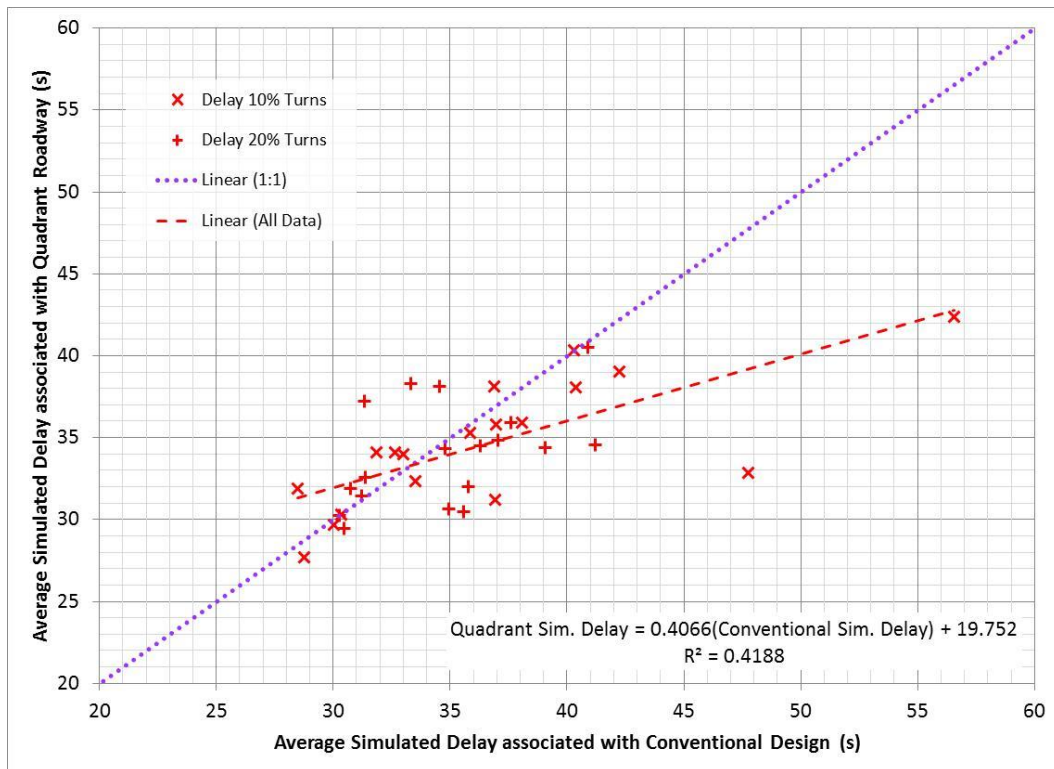


FIGURE 7-7 Change in intersection delay (simulated) due to quadrant roadway design

A comparison of the total travel time experienced across the two networks as predicted by microsimulation is seen in **FIGURE 7-8**. The data indicates that the quadrant roadway is anticipated to increase average travel times for the majority of volume combinations, with a reduction in travel time associated only with high-volume combinations. The coefficient of determination for the predictive equation has a value of 0.930, indicating that there is a small amount of variability about the mean expected value. It does appear that the percentage of turning vehicles is a significant indicator variable for increasing or decreasing total travel time. If, as it has been recommended, travel time is used as the ultimate measure of whether an unconventional intersection is appropriate or not, the quadrant roadway has a limited amount of applicability based upon the results of microsimulation analysis.

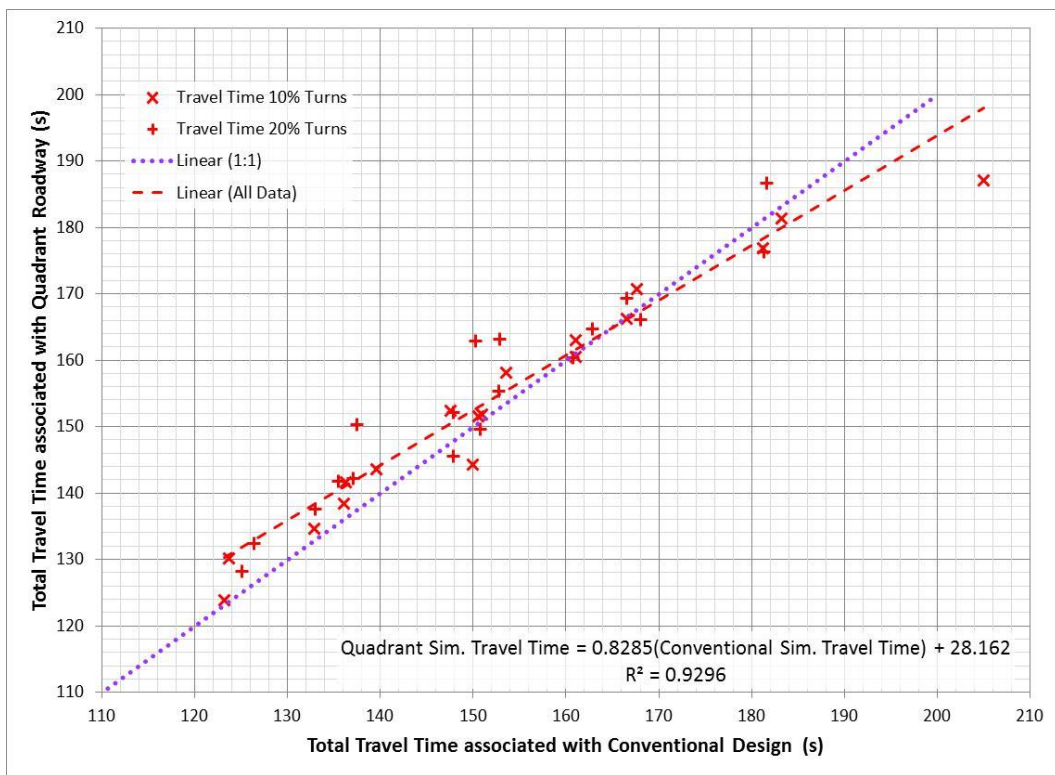


FIGURE 7-8 Change in average travel time due to quadrant roadway design

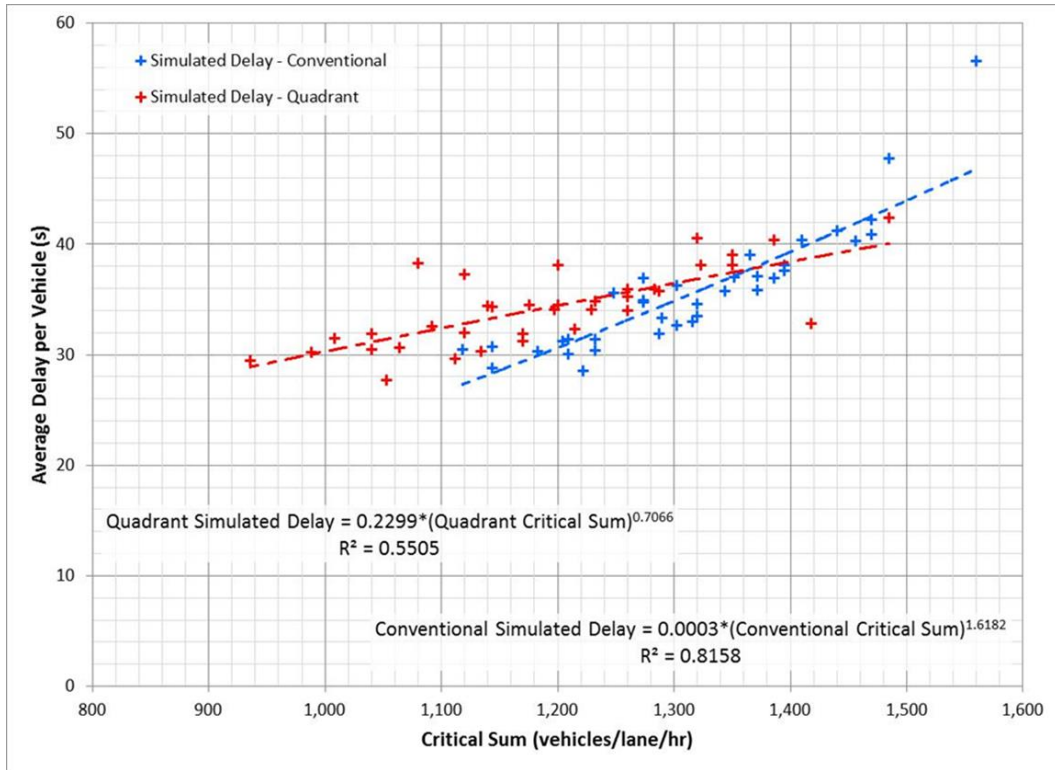
7.6 Analysis of Results

The critical sum method is found to be a decent indicator for simulated delay for a conventional intersection using a linear function, with a coefficient of determination of 0.816 as seen in **FIGURE 7-9**. In contrast to the results for the conventional intersection, the use of the critical sum method to predict the simulated delay for a quadrant roadway results in a coefficient of determination of 0.551 using a linear function, indicating a high degree of variability about the mean predicted value. In addition to the CSM's inability to predict simulated delay for the quadrant roadway design, it is seen from the figure that the relationship between CSM and simulated delay is not continuous between the conventional intersection and the quadrant roadway, meaning that direct comparisons are not appropriate. Similar results are found when CSM is compared against the simulated travel time for the conventional and quadrant roadway designs, also shown in **FIGURE 7-9**.

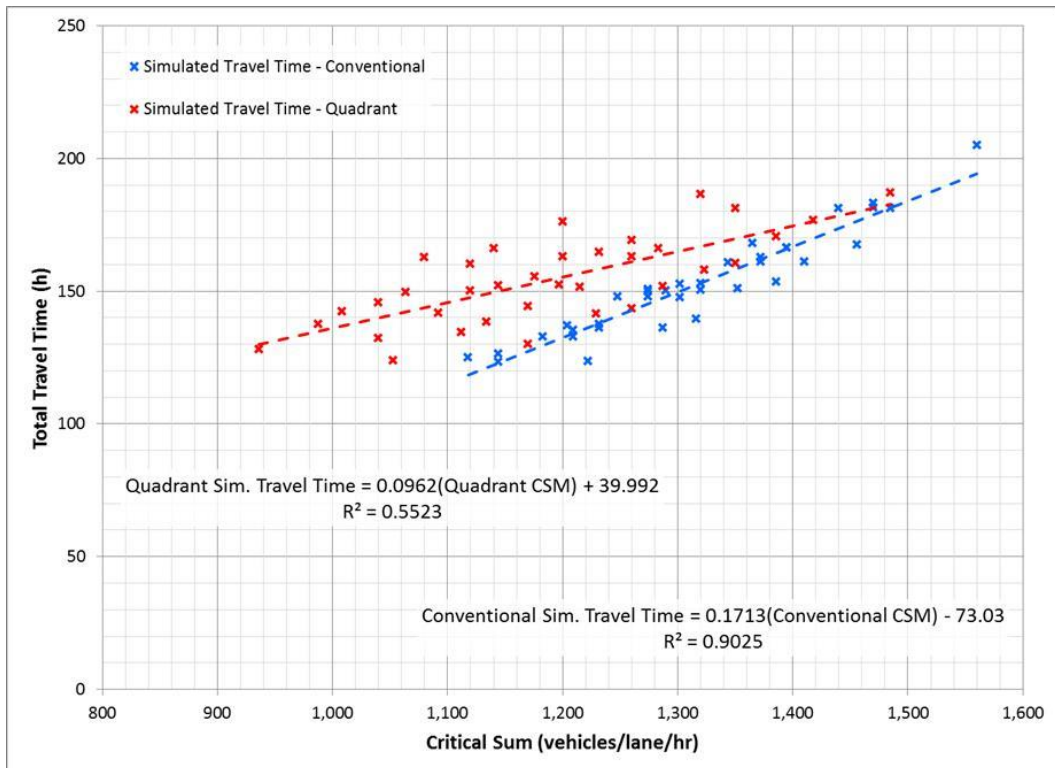
The critical sum method is a good indicator for simulated travel time for a conventional intersection using a linear function, with a coefficient of determination of 0.903. In contrast to the conventional intersection, the use of the critical sum method to predict the simulated delay for a quadrant roadway results in a coefficient of determination of 0.552 using a linear function, indicating a high degree of variability about the mean predicted value. In addition to the CSM's inability to predict simulated travel time for the quadrant roadway design, it is seen from the figure that the relationship between CSM and simulated travel time is not continuous between the conventional intersection and the quadrant roadway, meaning that direct comparisons are not appropriate.

The most important measure of critical sum as an indicator variable is to be found by tracking the change in the value of the critical sum when comparing the results of the conventional and quadrant roadway designs, and comparing it against the corresponding simulated change in either control delay or travel time, as seen in **FIGURE 7-10**. The coefficient of determination is effectively zero (0) when comparing the change in critical sum to the change in either delay or travel time, indicating that the critical sum methodology has negligible correlation with either the change in simulated delay or the change in simulated travel time.

To summarize the analysis of the research results, the critical sum method can serve as an indicator variable for HCM delay, simulated delay, and simulated travel time at a conventional intersection, but cannot accurately predict values of these measures for a quadrant roadway design, and is not useful for comparisons between a conventional intersection design and the quadrant roadway design. By extension, the author hypothesizes that the critical sum method would be inappropriate for comparisons between two alternative geometric designs when one or both of those designs involve vehicle rerouting.



(a) Critical sum method as an indicator variable for average vehicle delay



(b) Critical sum method as an indicator variable for total travel time

FIGURE 7-9 Use of critical sum as indicator variable

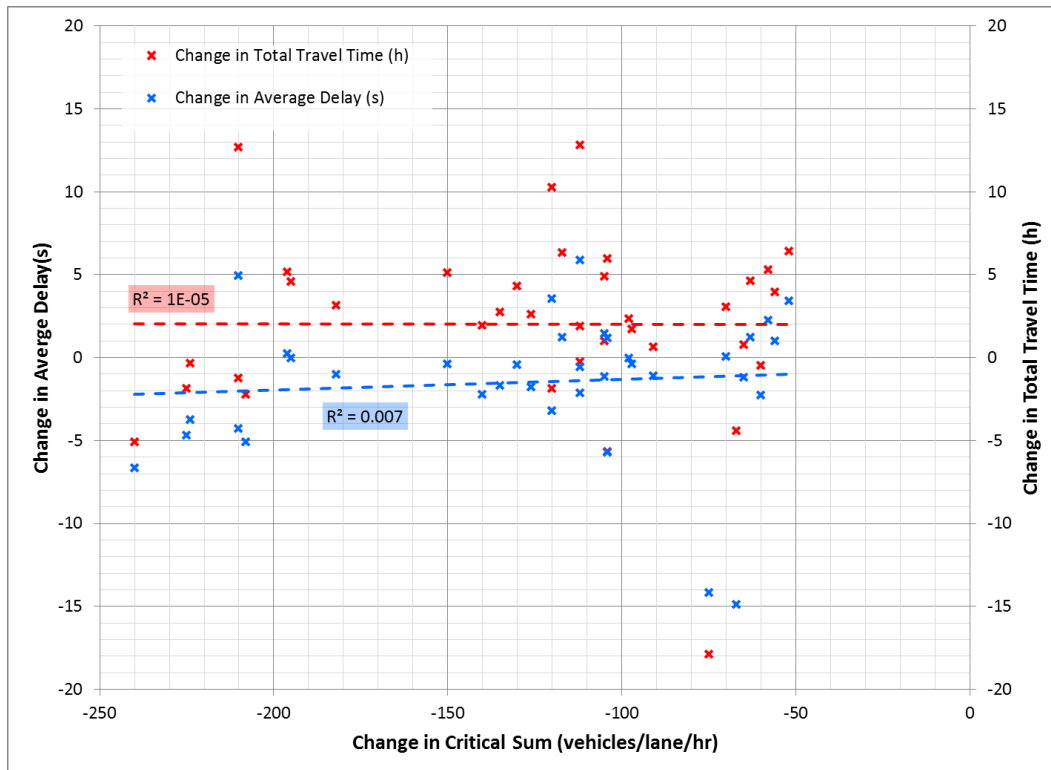


FIGURE 7-10 Change in critical sum as an indicator of change in delay or travel time

7.7 Limitations of the Current Research

The research results indicate that the critical point where the quadrant roadway design becomes the preferential design over the conventional intersection lies with a critical sum value in the range of 1,400 to 1,700, whereas the majority of volume combinations examined fall below this range. The volume combinations only consider the case where the main arterial is the east-west roadway, with the eastbound approach carrying the greatest amount of flow, to be consistent with previous studies of the quadrant roadway design. Operations are anticipated to potentially be drastically different for the given geometry if the majority of flow were to arrive from an alternate approach direction.

Currently the aggregated simulation results were generated from a single simulation run for each of the 36 volume scenarios, for a total of 72 simulation runs to investigate the conventional and quadrant roadway designs. Given the high hourly volumes examined in this study it is anticipated that high confidence intervals can be expected for relatively small error ranges, though pertinent calculations have not been included herein. While this is not ideal, it is generally consistent with the body of literature associated with the analysis of unconventional intersection designs.

Detailed geometric design was not performed on either intersection design regarding the safe turning-movement of trucks at a given design speed, which may have an effect on the operations of either alternative in simulation. Additional work needs to be conducted on the signal

operations of the alternatives within simulation, with remaining issues including the functionality of min-recall in the conventional design, and the calibration of the phase transition in the quadrant roadway design.

7.8 Conclusion and Recommendation for Future Research

Though the paper presented here has valid limitations, it definitively shows that the Critical Sum Method (CSM) is an inappropriate analysis methodology for use as an indicator variable relating conventional and unconventional intersection designs. From a consulting point of view, it has not been shown that relative values calculated from the CSM methodology is predictive of the degree to which one intersection will outperform another, in terms of either average delay or total travel time.

Future research should confirm the limitations of the Critical Sum Method, and determine whether it is universally ineffective with all types of unconventional intersections. That it is ineffective for other kinds of rerouting intersections such as the median u-turn design, the Jughandle design, or the superstreet design may seem to be a reasonable assumption based on the data presented herein, but the CSM may well serve as a useful methodology for designs that shift left-turns instead of rerouting them, such as the continuous flow intersection and the parallel flow intersection.

Given that the CSM is not applicable for preliminary analysis of unconventional intersections, future research should investigate other measures which can serve the same purpose, accurately indicating whether one design will be competitive with another. Ideally, a system of comparisons can be developed which can narrow the field of options for in-depth study that is based on reliable predictions of in-situ performance.

7.9 Acknowledgements

The author acknowledges: Dr. Joseph Hummer for his leadership in the study of unconventional intersections and interchanges, and his contagious enthusiasm for these designs; Mr. André Betit for passing on his knowledge of intersection geometric design, without which microsimulation layout would be out of reach; Dr. Montasir Abbas for accepting part of this work for course credit; and lastly, the Mid-Atlantic University Transportation Center for providing financial support in conducting this research.

Chapter 8. Capacity-based Predictions and Delay-based Results

Citation for original publication:

Sangster, J. D., & Rakha, H. A. (2015). Capacity-based Predictions and Delay-based Results: Why CAP-X is Fatally Flawed and How to Resuscitate It. Transportation Research Board, 94th Annual Meeting of the. Washington, D.C.

Abstract

Alternative intersections have unique geometries that theoretically lead to improvements in safety and arterial throughput. Compared to the modern roundabout, widespread adoption of these designs has been slow, due in part to the lack of delay-based analysis methodologies for use in preliminary engineering. Comparative analysis of these geometries previously required microsimulation of each alternative, until FHWA provided a capacity-based spreadsheet tool in 2011, the Capacity Analysis for the Planning of Junctions (CAP-X) program.

While the CAP-X tool provides a low-cost and easy to use software for the preliminary engineering stages of a project, the comparative results of this tool have yet to be shown to scale-up to delay-based comparative analysis. If the tool indicates that designs x, y, and z perform best from a capacity perspective, there is no guarantee that x, y, and z will correspondingly perform best from a delay perspective.

This paper seeks to determine whether the results from CAP-X are applicable. Results at this time are limited to a sensitivity analysis of volume combinations, examining the impact of total volume magnitude while holding the percentage of left-turning vehicles and major/minor approach splits constant. Preliminary results indicate that the CAP-X tool has significant limitations as a tool for choosing alternatives. Future research is recommended to investigate the potential for using capacity-to-delay transformation equations that are specific to each alternative, integrating these equations into CAP-X to ensure its future usefulness in the analysis of alternative intersection designs.

8.1 Introduction

The category of intersection design known as alternative intersections was known as unconventional intersections only five years ago, and five years from now may be so commonplace as to no longer hold the title alternative. These designs have come a long way since the first proposal of what is now called the restricted crossing u-turn design in the late 80's [3]. The near-universal adoption of the modern roundabout as a successful alternative intersection treatment is creating a more progressive outlook within transportation engineering practice, and other alternative designs may be ready to follow the modern roundabout's example. Sustained support by the Federal Highway Association (FHWA) for these designs has helped in normalizing them, such as with the production of the Alternative Intersections and Interchanges Informational Report [4].

These designs work by rerouting some of the turning movements made at a junction, often reducing and separating conflict points, with theoretical benefits to both safety and mobility. Conflict points represent each position where the paths of two vehicles cross, with potential collision locations identified as crossing, merging, or diverging conflicts. Any reduction in the number of potential conflicts, especially of the most dangerous crossing type, provides proportional benefit to intersection safety. This reduction in conflict points at these designs often comes with a reduction in the number of signal phases at the junction, providing a reduction in delay to through movements on the major arterial, while often increasing travel time and travel distance for secondary movements. Depending on the degree of benefit to the through movement on the major arterial and the degree of degradation to the secondary movements, the overall average delay may be more or less than that experienced by an equivalent conventional signalized intersection.

This paper summarizes the work that has been done to understand the relative performance of these designs in the past, and uses multiple analyses to examine the validity of state-of-the-practice analysis methods. The analysis of alternative intersections is currently in transition. At this time, practitioners must rely upon complete microsimulation of alternative designs to conduct comparative analysis of expected average intersection delay for a given scenario of traffic flow demands. A planning-level analysis method exists to compare measures of capacity for various designs, but this tool lacks validation (that the capacity measure is an equivalent indicator variable for delay) and has limited market penetration. Ongoing research being conducted by the Federal Highway Administration (FHWA), on which the authors previously collaborated, is developing macroscopic analysis methodologies for three alternative intersection designs to be integrated with subsequent publications of the Highway Capacity Manual (HCM) [5] and incorporated into standard software packages used by practitioners. The HCM methodology research will not provide a system for analyzing all types of alternative intersection designs, nor will it provide validation for the existing planning-level capacity analysis. The intention of the authors is to provide guidance and insight on the currently available methods, increasing their usefulness until validated delay methodologies become available for all intersection designs.

8.2 Evaluation Approach

Comparative analysis is conducted for six design alternatives using two analysis methodologies. The design alternatives are intended to be equivalent design solutions for a hypothetical junction between a minor arterial and a relatively low-volume major arterial. Traffic turning-movement demand scenarios are developed to assess the performance of each design between low-volume undersaturated conditions, and high-volume oversaturated conditions. Though there have been quite a few comparative analysis publications in the past, the primary purpose of previous publications was to assess the merits of one design over another [32], [33], [55], [79], [89], [93], [95], [96], [99], [123], [136]. The intent of the research presented herein is not to proscribe an appropriate design for a hypothetical situation, but to examine the analysis methodologies themselves for validity.

8.2.1 Selection of Geometries for Comparison

The data presented represents six intersection geometries, including the conventional signalized intersection, the modern roundabout, the median u-turn, the restricted crossing u-turn, the displaced left-turn, and the jughandle designs. Each of the six designs was fully designed in a CAD environment conforming to best practices in geometric design [137], with particular emphasis paid to realistically modelling truck turning radii. Only one configuration for each intersection design category is analyzed at this time, which generally represents the geometric condition to account for the smallest demand volumes of traffic that would justify an alternative intersection. Though the scale of each image has been allowed to vary to display the pertinent features of the designs, **FIGURE 8-1** below displays to-scale representations of the geometries analyzed. Decisions regarding u-turn and jughandle turn-out placements are based on recommendations from the literature on these designs [4], [36], as well as the projected back-of-queue locations on the turn bays. The farther away from the centroid of the intersection these are located, the greater the additional travel time experienced by some movements, however, if placed too closely the queues from this location will spill back causing even further delay.

In practice, an engineer would be working with a single, or series of, given peak hour turning-movement demand flow rates, and the lane configuration of each design alternative would be calibrated to best meet the specific needs of those demands. Once each alternative was sized to provide an adequate and equivalent projected level of operational service in terms of average delay, the engineer would then proceed to evaluate the designs based on other service measures such as safety, available right-of-way, cost, and accessibility. In contrast, the research application of these designs must set aside site-specific constraints which are essential in practice, and focus instead on providing the most accurate analysis methodologies possible for any/every condition. By providing analysis methods that give consistent and accurate predictions of the operational performance of these designs, the authors hope to give practitioners the best-possible understanding of their design options before site-specific considerations are examined.

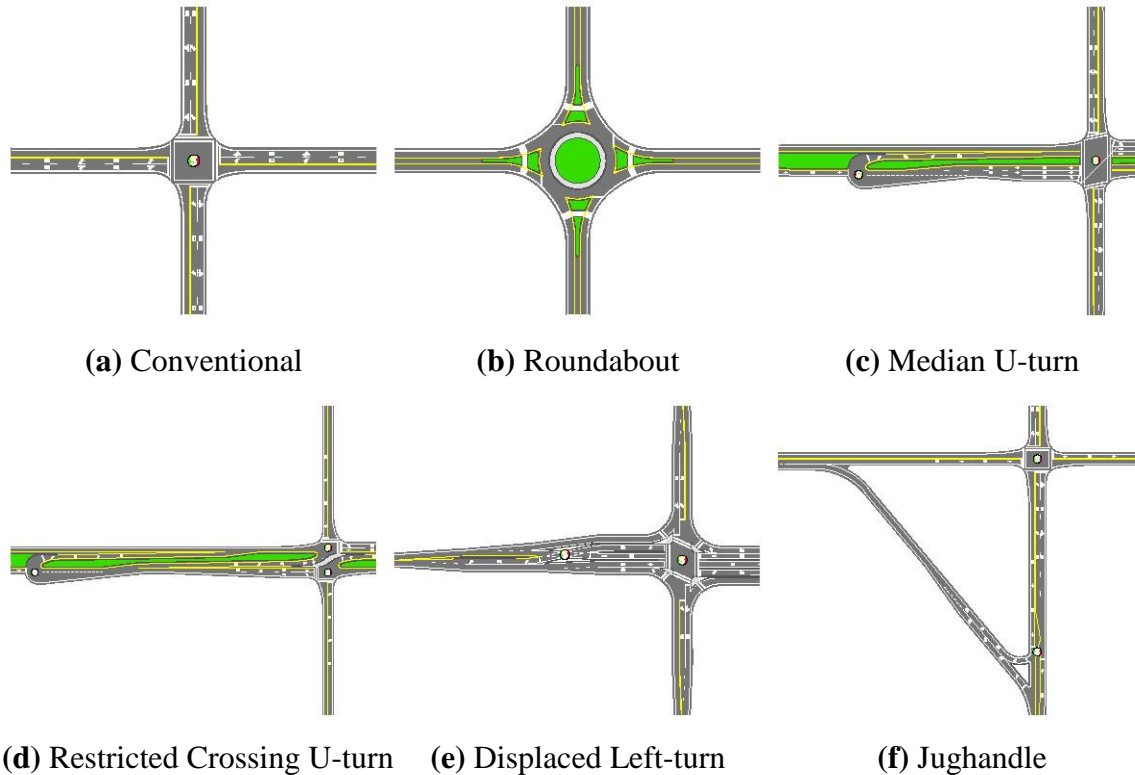


FIGURE 8-1 Geometries used for comparative analysis

While there are many considerations for what will make a set of designs “equivalent” to one another, the six designs above are intended to represent equivalent sized geometries across design alternatives. In the case of the Roundabout and the Displaced Left-turn designs, the lane configuration provided is the smallest possible design to include all of the elements necessary.

8.2.2 Origin-Destination Demands for Comparative Analysis

As a number of the designs investigated operate as a network of signalized nodes instead of a single junction location, turning-movement-count scenarios were instead treated as origin-destination scenarios. Ideally, a vast number of volume scenarios would be investigated to determine the conditions under which CAP-X does or does not provide predictive results. However, due to the processing-time requirements of running microsimulation, only a small number of combinations have been investigated at this time.

The authors determined that the most crucial factor to investigate at this time is the assumption that each of these designs “fails” at the same critical sum value. To limit the number of volume combinations examined, sensitivity analysis is conducted on the total volume of demand on all approaches, holding other parameters constant. The scenarios are based on the two-way volume on the major arterial, varying this value between 200 and 2500 in increments of 100 veh/h. The volume demands on the minor arterial are taken as two-thirds of that on the

major arterial, rounded to the nearest 50 veh/h. No directional splits are applied. The turn-movement percentages are held equal on all approaches, with 20% left-turns, 70% through movements, and 10% right-turns. The resulting volume scenarios test capacity and delay results for a wide range of saturated and unsaturated flow conditions.

8.2.3 Analysis Methodologies

The methodologies analyzed herein all relate to the operational effectiveness of the design from a mobility perspective. The primary consideration used in the design of an intersection is the average delay experienced by each vehicle as they pass through the junction. Secondary considerations include the average delay experienced by each turn-movement group (origin-destination set), and the fraction of overall capacity that a given volume demand scenario uses.

The overall intersection delay is of utmost importance, because it is the metric by which development is measured and fees are assessed. If a developer would like to construct a department store (or any other traffic generator), they must first demonstrate that their traffic generator will not impact the surrounding intersections beyond a proscribed intersection delay level of service. Often, impacts from traffic generated by a development cause a developer to pay for offsite improvements to the affected intersections to maintain existing vehicle delay levels. As a result, it is essential to practitioners to have reliable predictions of the anticipated average delay if they are to include alternative intersection designs as potential solutions. The current tools available to practitioners include CAP-X, a cost-effective planning-level tool for capacity analysis (developed from the previous AIST program)[87], [131], or high-cost microsimulation software packages for detailed analysis. Neither of these options are meeting the needs for preliminary engineering, when design alternatives are proposed.

Capacity calculation for alternative intersection designs is currently conducted through a critical sum (critical lane) analysis. This methodology was initially developed to explain signal operations to non-engineers [84], [85], and has subsequently become an underpinning concept for the HCM intersection delay models. Effectively, critical lane analysis breaks an intersection down into the component signal phase groupings, and then checks for the critical, or largest, value of demand in terms of vehicles per lane per hour for each signal phase, summing the series of critical signal phases to arrive at a total demand of vehicles per hour that are critical to the operation of the signal. Critical lane analysis can only be completed after the phase diagram has been created for the signals – the typical methodology within the U.S. is to use the NEMA standard ring and barrier design for signal phasing [9].

As the volume scenarios investigated herein include only balanced volumes from opposing directions, the dual ring structure of the ring and barrier design collapses into a single ring structure. The purpose of the control, to separate conflicting vehicle paths, remains. Diagrams with sample signal phasing (timings to scale) are shown in **FIGURE 8-2**. With turn-percentages and directional splits held constant for the volume demand scenarios, the diagrams shown may conceptually be applied to all volume combinations analyzed in this paper.

Critical lane analysis was initially conducted on the six designs for each of the twenty-four volume scenarios. Using the assumption of 1,600 as the failure threshold for the critical sum values, an approximate volume-to-capacity ratio was determined for each of the 144 volume-geometry conditions. The Webster method was applied to determine the natural cycle length in each case, with simulated cycle lengths rounded to the nearest ten seconds. To model the designs as they would be done in practice, the cycle length was constrained between 60 and 120 seconds for the conventional signal, with minimum cycle length reduced to 40 seconds for designs with fewer phases and less loss time. The green-time afforded to each turning-movement group was set based on the critical lane analysis, distributing green time proportionally while maintaining a minimum of four-seconds of green time per phase.

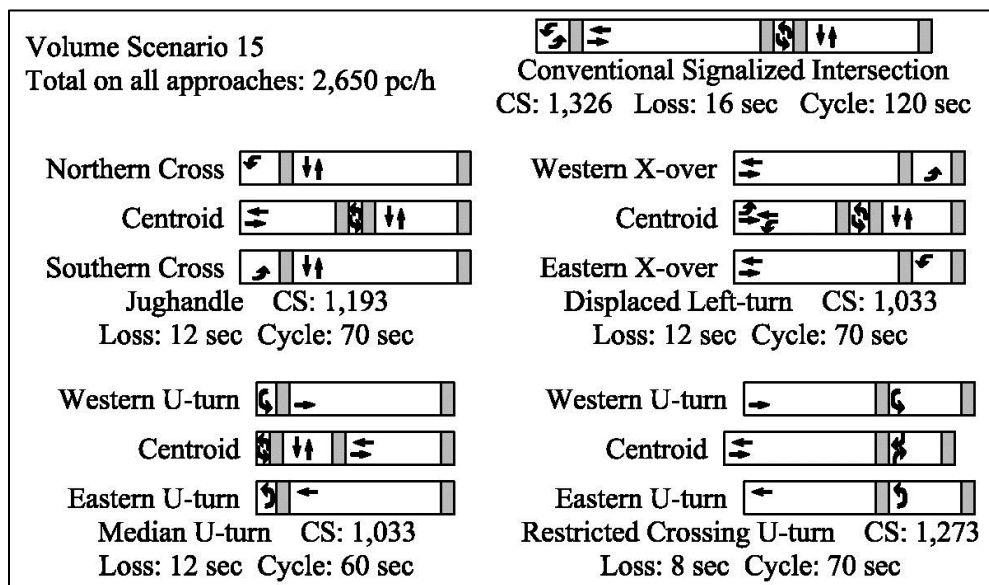


FIGURE 8-2 Sample signal phase diagrams for the simulation of alternatives

8.2.4 Microsimulation Input and Output

Typically, microsimulation of conventional signalized intersections would be conducted with undersaturated traffic volumes, collecting data in one-hour increments. There is an initial period of the simulation in which no data is collected to allow for the loading of traffic into the network. The inherent variability in microsimulation requires that multiple seeds of each volume-geometry scenario be run to achieve a sufficient amount of confidence with the values reported. Due to the inclusion of oversaturated conditions in this study, simulation time was reduced to avoid bias in the data from increasingly large queues spilling back. Each individual run lasted for thirty-five minutes of simulated time. Data was collected on a per-vehicle basis based on the expected network entry times, limited to those vehicles expected to enter the network between five minutes and 30 minutes from the beginning of the simulation. Fifty random seeds were run for each scenario, with 95% confidence intervals maintained below half a second for undersaturated conditions, and one second for conditions over but close to saturation. For

scenarios with demands far in excess of saturation the queues grew throughout the duration of the simulation, often exceeding available storage bays for turn-movements and blocking entire approaches.

On the topic of the desired output parameter, alternative intersection designs pose a problem for the standard measure of control delay, that is, the amount of time in seconds that a particular vehicle is delayed from completing their travel due to crossing through a junction. Microsimulation calculates the control delay for a vehicle as the difference between the expected velocity and the experienced velocity as it traverses a network. In the case of alternative intersections, the vehicle may encounter additional travel distance without incurring additional delay, so long as the desired velocity is met along the stretch. The current recommended practice is to create equal-sized networks and compare the average total travel time per vehicle as the output. The authors propose tying the travel time metric back to the delay metric by creating simulation networks where each origin-destination point is equidistant from the centroid of the intersection. In the absence of any intersection control, all twelve origin-destination pairs would then travel the same distance. By setting the desired velocity to be equal on all approaches, a theoretical base travel time can then be determined as the diameter of the network divided by the desired free-flow speed. In the case of the intersections modeled herein, the desired free flow speed is set to 40 km/h for all intersection designs, with the network diameter held at 1 km, generating a base travel time of 90 seconds. By subtracting this 90 second base time from the average total travel time metrics generated, a realistic measure of total delay, control and geometric, can be provided. Alternative free flow speeds could be used for individual design conditions, which would require the use of individual base travel-time measures.

While it is best to incorporate real-world data where possible, it is outside of the scope of this study to calibrate the simulation results back to observed data which may or may not exist for some of the design geometries.

8.3 Research Results from Previous Work

The work presented in this paper is an extension of previous work conducted by the authors. Initially investigating the question of how capacity and delay are related, a paper was presented in 2011 examining the relationship of capacity and delay in comparing the quadrant roadway design to a conventional signalized intersection design [124]. This initial effort examined relative differences in capacity between the two designs compared to relative differences in travel time. The data from this study found no correlation between the relative difference in critical sum and the relative difference in delay, raising questions about the critical sum value as an indicator variable for delay. Unfortunately, as a comparative “which one is best” analysis, the paper failed to examine how the internal relationships between capacity and delay worked for each design by itself.

A paper presented in 2014 by the authors investigated the conclusions that could be reached about each design based on CAP-X, such as which traffic regimes each would be appropriate for use with [123]. Using the MUTCD signal warrant axis as a starting point, failure thresholds

were generated for multiple sizes of each design. A preliminary step was taken to validate the failure threshold used for capacity analysis, with multiple sizes of conventional signalized intersections processed for capacity and HCM average delay. In this case we see that the standardized value of 1,600 correlates approximately to 70 seconds of delay, in the middle of the LOS E regime. Seeking guidance on how to define this value, the 2010 Highway Capacity Manual [5] was found to say that: “The capacity of a system element is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions.” For lack of more defined guidance on the definition of capacity, the authors have decided to hold the 1,600 standard critical some value as a maximum threshold, and take 80 seconds of average delay, the transition from LOS E to LOS F, as the delay that corresponds to the idea of maximum capacity threshold.

8.4 Research Results

Subsequent to the generation of the critical lane analysis through algebraic means, the HCM methodology delay was generated through the use of Synchro traffic analysis software (for the conventional signalized intersection only), and microsimulation analysis was conducted using the INTEGRATION software.

8.4.1 Expected Findings Based on Capacity Analysis

The twenty-four volume combinations assessed as part of this study provide a simplified perspective on how volume demands, capacity, and delay are related for these designs. The intention of the CAP-X software is to provide a preliminary tool to determine which sub-set of designs to move forward with into simulation. Following this logic, we first examine the relationship between the volume scenarios used and the value of the critical sum generated for each design from these volumes. Although the other intersection designs in CAP-X are treated with the critical lane analysis, the capacity equations from NCHRP 572 are implemented for analyzing roundabouts, which is consistent with HCM 2010 methodologies as well [5], [20]. The supplemental roundabout capacity equations are based on field observations of gap acceptance and follow-up headway taken in the early 2000’s at roundabout locations around the United States. In data presented in **FIGURE 8-3** include the critical lane calculations for all designs examined herein, as well as the additional capacity calculations generated from NCHRP 572.

The capacity predictions for the roundabout based on the alternate method are significantly more conservative than the critical lane analysis, with only 12 volume combinations predicted to be below capacity. Using the critical lane analysis to predict how of the volume combinations tested will be below capacity, the roundabout is anticipated to have 16, the conventional intersection to have 18, the restricted crossing u-turn to also have 18, the jughandle to have 20, and the median u-turn and displaced left-turn to perform equally well and under capacity for 23 of the 24 volume combinations.

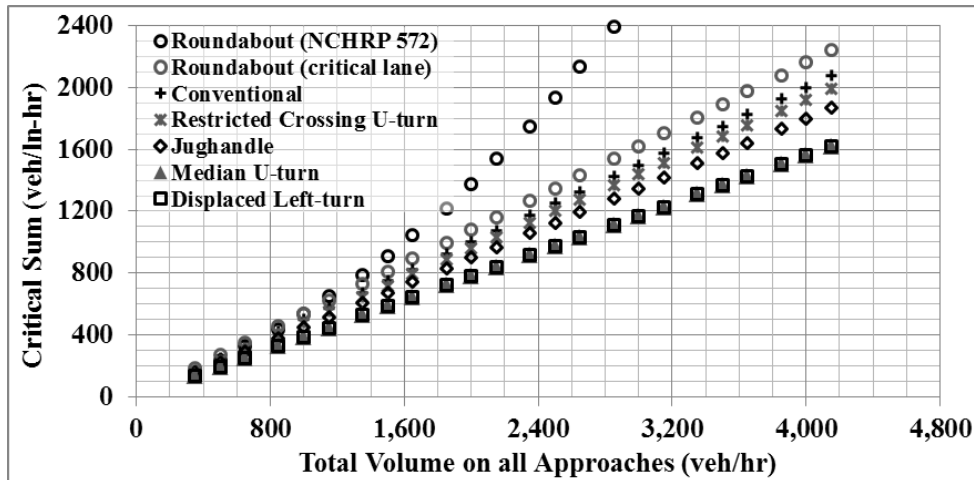
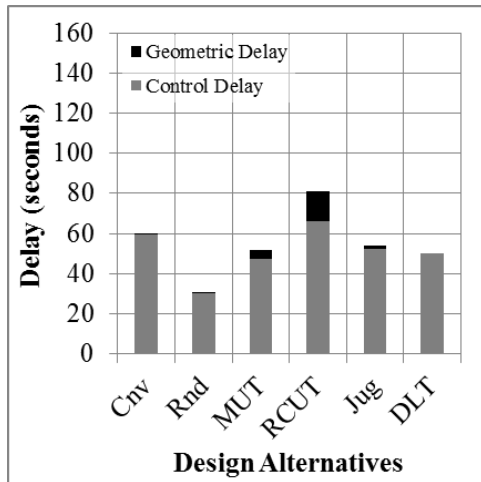


FIGURE 8-3 Anticipated failure thresholds for intersections investigated

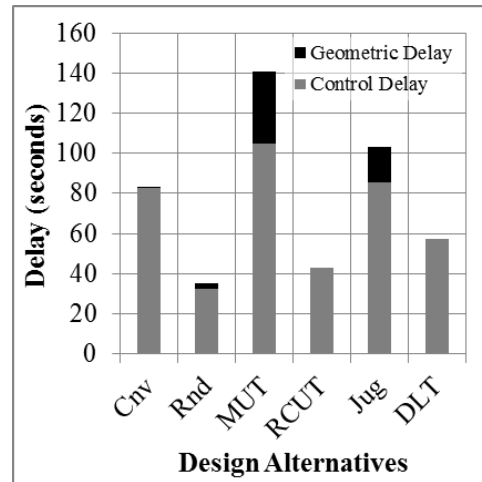
8.4.2 Measures of Delay for Comparative Analysis

The authors previously discussed their proposed for calculating total delay to vehicles traversing alternative intersection designs. The data presented in **FIGURE 8-4** displays the relative control delay and additional geometric delay for four conditions, including (a) delay for all vehicles, (b) delay for major approach left-turning vehicles, (c) delay for major approach through movement vehicles, and (d) delay for vehicles on the minor approaches. The information presented comes from volume scenario 15, with a total demand volume of 2,650 vehicles. To accommodate the space requirements for the figure, the various design names have been truncated as Cnv for conventional intersection, Rnd for roundabout, MUT for median u-turn, RCUT for restricted crossing u-turn, Jug for Jughandle, and DLT for displaced left-turn.

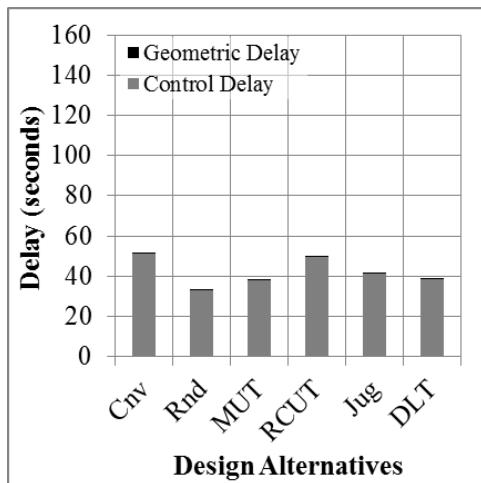
Initial observations from these charts validate our expectations, that the median u-turn and jughandle designs impart additional geometric delay in the form of travel time to major-approach left-turning vehicles, while the restricted crossing u-turn creates the greatest amount of geometric delay, with minor-approach vehicles inconvenienced. A second important observation from this chart is that the total delay imposed by the roundabout is lower than all of the other designs at this level of demand volume, and that it is not near, or significantly over capacity as predicted by the NCHRP/HCM capacity methodology. It should be noted that in simulating the roundabout we used the default critical gap of 4.5s. Furthermore, the HCM assumes that the base unopposed approach lane capacity is only 1130 veh/h/lane, which is significantly lower than the base lane capacity of 1600 veh/h/lane that was used in the simulation runs. The HCM does suggest that the parameters be calibrated to the local conditions and thus it was assumed to be 1600 veh/h/lane to be consistent with the values used for the other traffic control strategies.



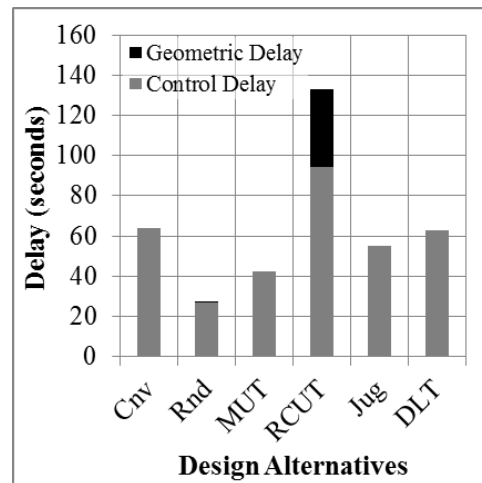
(a) Total Average Delay



(b) Average Delay – Major Left-turns



(c) Average Delay – Major Through

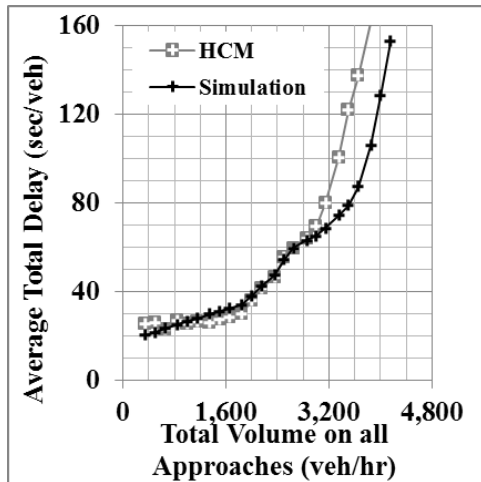


(d) Average Delay – Minor Approach

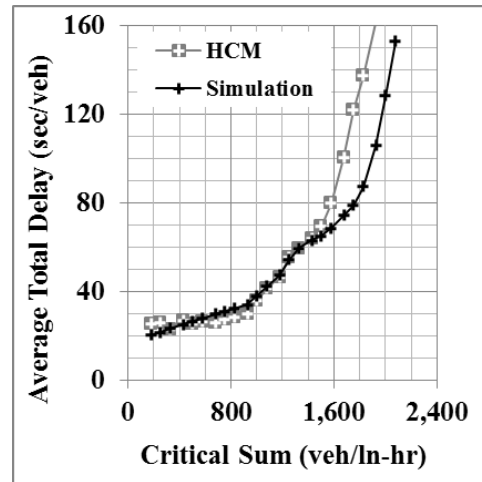
FIGURE 8-4 Control delay and total delay for a sample volume scenario, with (a) total average delay, (b) major approach left-turn, (c) major approach through, and (d) minor approach

8.4.3 Correlation of the Critical Sum Values with Average Intersection Delay

Although the author's previous research had validated the relationship between critical sum and delay for the conventional signalized intersection, we felt it was an important exercise to re-check this finding against the volume scenarios used herein, using both HCM methodology and microsimulation. The data in **FIGURE 8-5** shows the relationship between average total delay to both (a) the total volume on all approaches, and (b) the critical sum value, for a conventional signalized intersection.



(a) Average Delay vs. Total Volume



(b) Average Delay vs. Critical Sum

FIGURE 8-5 Conventional signalized intersection: relationship of average delay to (a) total volume and (b) the critical sum value

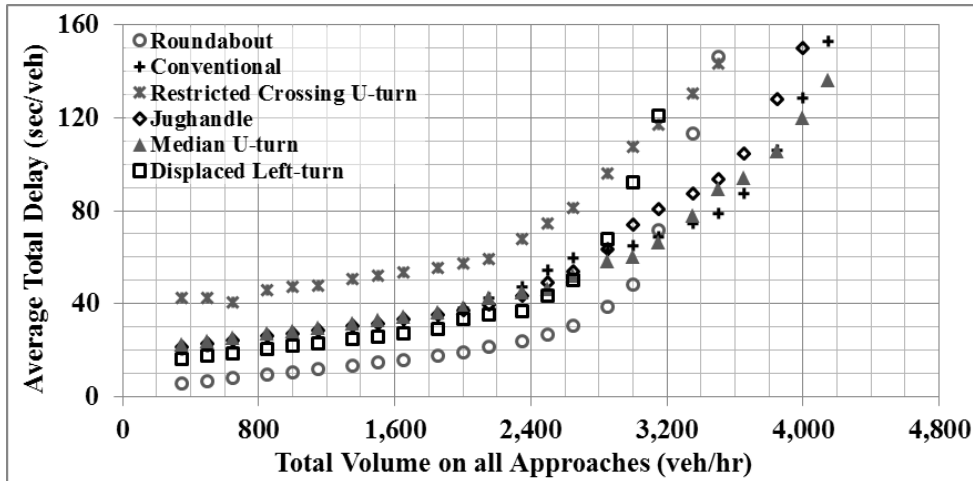
Our initial finding is that the relationship previously found between capacity and delay generally holds true with our dataset, and that our microsimulation analysis is calibrated sufficiently to provide an approximate duplication of the HCM methodology delay results. A secondary observation is of the shifting slope of the interaction line which can be tied back to the cycle length of the signal timing plans used. The low slope indicates slowly increasing delay with increasing volume-to-capacity (v/c) while the cycle length remains fixed at 60 seconds, the second slope indicates the delay imparted by both increasing v/c and increasing cycle length until the maximum is reached at 120 seconds, with the final simulation curve indicating over-saturated flow conditions with a constant cycle length of 120 seconds.

This methodology is extended to examine the relationship between volume and delay, and between critical sum and delay, for the six alternatives in **FIGURE 8-6**. In this case the roundabout is analyzed with the critical-lane approach.

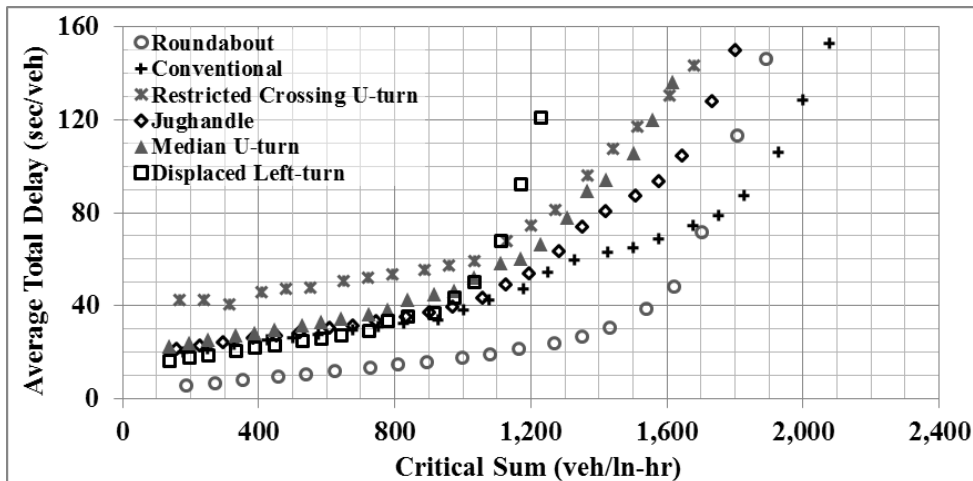
The total volume chart provides us a direct comparison between the various designs, as each design is shown relative to its peers in a vertical line for the volume scenarios. Making some observations on the relationship between the designs, the restricted crossing u-turn consistently performs more poorly than its competitor designs. The one-lane roundabout design provides significantly lower average delay per vehicle for lower volume scenarios, but is slightly less robust than some of the other designs, surpassing them in average delay once it hits capacity and breaks down. The displaced left-turn design performs the second-best of the designs, but is also sensitive to break-down once it hits capacity. The conventional signalized intersection, the jughandle, and the median u-turn are effectively equal in terms of operational performance.

The relationships between the various intersection designs are purer when examining volumes than when looking at the critical sum values. Using the delay measure of failure

previously defined at 80 seconds per vehicle, we see the conventional intersection is the most robust failing at 1,750, the roundabout fails shortly after 1,700, the displaced left-turn fails just after 1,400, the median u-turn fails around 1,300, the restricted crossing u-turn fails around 1,250, and the displaced left-turn fails near to 1,150 veh/ln-hr.



(a) Total volume versus average total delay

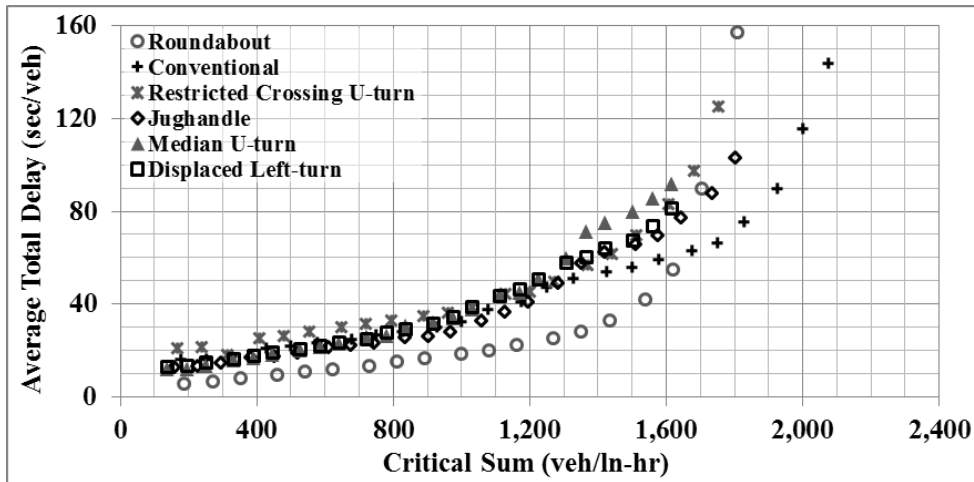


(b) Critical sum versus average total delay

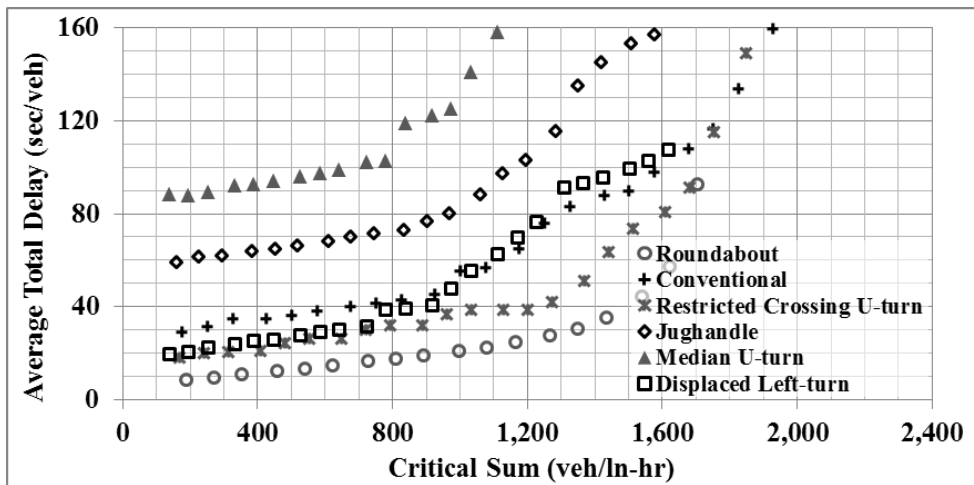
FIGURE 8-6 Average total delay relating to total volume and critical sum measures

8.4.4 Correlation of the Critical Sum Values with Movement Group Delays

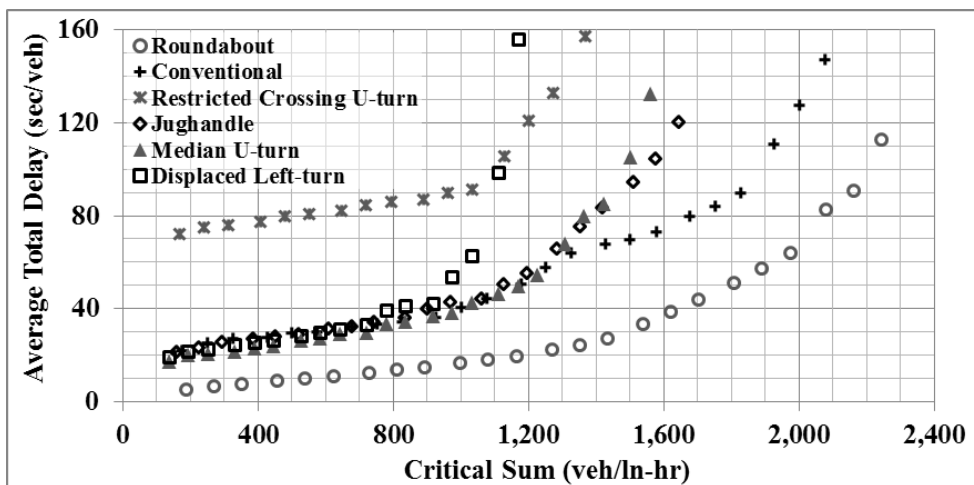
Because these designs are sometimes implemented to serve a specific sub-group of turn-movements, such as to preserve progression, the critical sum versus average delay measures are presented for (a) major-approach through movements, (b) major-approach left-turning movements, and (c) minor-approach movements in **FIGURE 8-7**.



(a) Critical sum versus average delay of major approach through vehicles



(b) Critical sum versus average delay of major approach left-turning vehicles



(c) Critical sum versus average delay of minor approach vehicles

FIGURE 8-7 Critical sum versus average delay for selected lane groups

Many of the designs included in this study are cited as potential solutions to provide significant improvements for major arterial throughput. The results here indicate that, for the volume combinations examined herein and the signal optimization methodology applied, the alternative intersection designs provide comparable performance for through-vehicles compared to the conventional intersection, with the exception of the roundabout, which performs better.

Examining the left-turn movement delays from the major approach brings up further concerns. In many cases a proposed intersection design is not permitted to have any individual turning movement operating with greater than 80 seconds of delay, which would prevent the use of the median u-turn design even for low-volume conditions.

The last movement results provided examine the minor approach. Here we see the disadvantage of the restricted crossing u-turn design, and the continued advantage of the modern roundabout. A note of caution regarding the results for the displaced left-turn intersection, the simulation created to model the displaced left design was unable to provide the expected minor-approach left-turn capacity, and for higher volume combinations the queue from this movement spilled back beyond the end of the auxiliary turn lane, blocking all traffic from the minor-approach and creating the failure pattern shown.

8.5 Analysis of Results

In general the results show the expected operational benefits and costs associated with each of the alternative geometric designs; though the benefits were lower and the costs were higher than was hoped would occur. Of greatest importance is that the predictions generated by the critical lane analysis as represented by **FIGURE 8-3** were shown to be not predictive of the results subsequently generated in **FIGURE 8-6**. Moreover, **FIGURE 8-6** additionally shows that the premise that each of these designs fails at a common value from the critical lane analysis is false. A summary of the expectations and findings is presented in **TABLE 8-1**.

TABLE 8-1 Capacity-based predictions and delay-based results

Intersection Geometry	Capacity Predictions				Simulation Observations			
	Anticipated Failure Delay	Critical Sum Failure Value	Volume at Max. Capacity	Rank	Observed Failure Delay	Critical Sum at Observed Delay	Volume at Observed Delay	Rank
Modern Roundabout	80	1,600	2,963	6	80	1,724	3,190	3
Conventional Intersection	80	1,600	3,198	5	80	1,760	3,521	1
Restricted Crossing U-turn	80	1,600	3,333	4	80	1,367	2,847	6
Jughandle	80	1,600	3,555	3	80	1,412	3,138	4
Median U-turn	80	1,600	4,103	1	80	1,319	3,378	2
Displaced Left-turn	80	1,600	4,103	1	80	1,142	2,925	5

Had this been a consulting effort instead of a research exercise, the CAP-X software would have recommended to proceed with more detailed analysis of the displaced left-turn, the median u-turn, and the jughandle intersection designs, based on their predicted robustness for dealing with

the volume combinations presented. Fortunately for the consultant a conventional alternative would also have been modeled as a base condition. Once simulation had been conducted, the consultant would have discovered that the conventional alternative outperformed the alternatives analyzed, despite the predictions of the CAP-X program, which, along with the alternative designs, may not be used a second time.

The critical sum values observed to correspond to an average of 80 seconds of delay per vehicle range from 1,142 for the displaced left-turn to 1,760 for the conventional signalized intersection. The simplest method to correct for this discrepancy would be to provide multiplier adjustment factors for each of the designs to adjust the calculated critical sum value for that design to an equivalent centralized critical sum. Unfortunately, this type of transformation would not overcome the conceptual issues identified by the results of this study. If six alternatives are considered and all found to be operating at 60% of their adjusted critical sum capacity, this does not, in turn, mean that those six designs will perform equally well for delay when simulated. Practitioners working with CAP-X will find its usefulness to be limited.

8.6 Limitations of the Current Research

The current research is limited by its scope, hampered by remaining microsimulation design questions, and limited in its ability to change the status quo.

Developing relationships between capacity and delay for each of these intersection designs will require the exploration of all of the demand volume parameters. Real-life traffic flows have a wide variety of major/minor splits, directional splits, turn percentages, and magnitude, and they are very rarely symmetrically loaded. Some or all of these parameters may have significant impacts on the relationship between capacity and delay, and a vast number of volume combinations would need to be tested to tease out the interrelationships they have.

One of the hurdles to implementing these designs with the existing analysis methods and software programs is the set of unknown driver behaviors that go along with the unusual travel movements made to traverse these designs. The calibration of a microsimulation model is an essential component to ensuring that the results of a study are trustworthy, but in this case there is very little data to use for calibration. An additional hurdle for simulation not encountered in capacity analysis is the development of signal timings for each volume combination at each intersection design. If the signal timings used are non-optimal, the legitimate differences in operational performance between the various designs may be overshadowed by signal phasing or offset inefficiencies.

The greatest limitation of this research at the moment is that the authors are not conducting this work on behalf of the Federal Highway Administration, so implementing any future recommended changes into the CAP-X program may prove difficult to accomplish. Beyond implementing future proposed changes to the program, the program currently has limited exposure to the engineering practice community, so further outreach activities may be necessary ensure use of a revised CAP-X tool.

8.7 Conclusions and Recommendations for Future Research

The volume-to-capacity measures currently provided by CAP-X give recommendations on which designs to move forward with, color coding the various geometries with green or red for good or bad, but these recommendations do not correlate with the subsequent results of simulation that they purport to predict. The results of this study indicate that critical sum analysis does not generate a common failure threshold value for all designs. Moreover, the relationship between volume-to-capacity ratio and delay varies greatly across the alternative geometries investigated. While it may be possible to develop a capacity-to-delay transformation equation with further volume scenarios and analysis, such a transformation would need to account for both adjusted failure thresholds and expected delay measures in undersaturated conditions.

As part of this research effort, the authors have developed a unified approach for dealing with travel time and delay for alternative intersections that may prove to be an efficient method to standardize for the comparative analysis of intersections in the future. By setting a uniform data collection diameter from the centroid of a junction, the desired free-flow speed can be used to calculate a constant base travel-time for all origin-destination pairs. The reported average travel time for vehicles then has a consistent modifier to generate a measure of total average delay that includes both control delay and delay due to additional traveled distances. On networks with different free-flow conditions on the major and minor approaches, such as a freeway, a weighted average speed may be used to generate the base travel times for analysis.

The authors feel that the critical lane analysis technique is a powerful tool for understanding conceptual lane allocations for geometric design, and it provides a framework to insightfully show the theoretical benefits of alternative intersection designs. However, at present time it is being presented as a preliminary analysis tool for comparative analysis of potential intersection designs in consulting. Our finding based on the small sample of volume scenarios analyzed herein is that the v/c measures currently generated by CAP-X lead to incorrect conclusions regarding which designs should be moved forward. By recommending inefficient designs for further analysis, the program may be hurting the potential for further implementation of alternative designs, the opposite of its intended purpose.

The belief of the author is that further analysis of the relationship between capacity and delay for these design alternatives may lead to the development of transformational equations that can supplement the current critical sum analysis. The incorporation of these capacity-to-delay transformation equations into a future version of the CAP-X software would preserve the positive aspects of the existing software, while increasing its utility both as a prediction tool and as an advocate for these innovative design solutions.

8.8 Acknowledgements

The authors acknowledge the Mid-Atlantic University Transportation Center, and the Virginia Department of Transportation for providing financial support in conducting this research.

Chapter 9. Conclusions and Recommendations for Future Research

This research examines three main focus areas related to the operational analysis of alternative intersections and interchanges. The first focus deals with the existing service measures for intersections and interchanges, and examines how these measures can be altered to incorporate alternative intersections into a unified approach for all roadway junctions. The second focus looks at the simulation of alternative intersections, using field data to examine the validity of microsimulation analysis for alternative intersections and interchanges, and subsequently using microsimulation to implement the proposed service measure approach to the comparative analysis of the through-about, roundabout, and conventional signalized intersection designs. The third focus area was on the planning-level tools available for alternative intersection geometries, exploring the implications of the critical lane analysis approach, questioning the robustness and validity of the method, and finally proposing a modified approach to develop a new planning level tool using individual capacity-to-delay relationships for the various designs. Future research on the operational analysis of alternative intersections and interchanges may include the further development of the proposed planning-level analysis, additional exploration of signal-timing best practices for these designs, as well as the vetting of new design geometries as they are proposed. Moving beyond the operational analysis, a great deal of work stands to be done working with these designs to develop safety impact factors for them, best-practices for pedestrian and bicycle accommodations, and an exploration of potential environmental impacts, both positive and negative, associated with these geometries.

9.1 Service Measures for Intersections and Interchanges

This research proposes a new approach for operational service measures at intersections and interchanges, replacing control delay with the proposed junction delay, a service metric that can be universally applied to all intersection and interchange geometries. Additionally, the relationship between level of service and delay is revisited with a new approach proposed that scales the grade regimes by peak-hour demand volume, and is applied uniformly for all geometry and control types.

This method causes large impacts to the origin-destination pairings (turn movements) at both alternative intersections and at conventional interchanges, with large increases to left-turn vehicle travel times and even larger decreases to right-turn vehicle travel times, once the geometric component of the vehicles travel time and path is incorporated. While the inclusion of additional travel time is essential to not give designs like the restricted crossing u-turn an unfair advantage from control delay alone, the universal adoption of this approach may greatly affect which designs are chosen based on travel time and delay impacts.

The relationship proposed by the author to convert delay to level of service based on demand volume instead of control type gives a unifying approach that can be applied consistently to all intersection types currently on the roadway, as well as those designs yet to be developed. The method speaks to the fact that intersection sizes can vary greatly within a given design type, and that drivers perceptions of delay may be influenced as much by the scope of the problem being

solved as by the control type being used to solve it. Adopting an LOS regime methodology based on demand volume would solve a number of inconsistencies in terms of comparative analysis between intersection geometries. The threshold values proposed are based on the MUTCD signal warrant approach of considering the busiest three approaches at a junction, with values set at 1,500 vehicles/hour for the low-to-medium threshold and 4,000 vehicles/hour for the medium-to-high threshold. These values are set based on engineering judgement by the authors to approximate intersections not meeting the peak-hour warrant for signalization in the low category, and junctions requiring grade separation in the high category. Further research is required including significant participant involvement to calibrate these volume thresholds.

9.2 Simulation Level Analysis

Peak-period field data from three alternative intersections are analyzed using three software applications, including INTEGRATION and VISSIM for traffic microsimulation analysis, and the HCM analysis as generated by Synchro traffic analysis software. The microsimulation applications are found to provide robust results for all types of intersection geometries, predicting delay measures for both average intersection delay and individual turn-movement delays with sufficient accuracy for design purposes. The macroscopic HCM methodology is found to accurately predict delays associated with the DDI design, but is unable to predict delays for either the DLT or RCUT designs. Primary failures for HCM methodology predictions came from the left-turn movements for the at-grade designs, with macroscopic analysis predicting oversaturated flow conditions when using observed signal timing plans, despite microsimulation accurately modeling these same movements with identical inputs. Until such a time as new methodologies for alternative intersection/interchange analysis are implemented within HCS and Synchro software, it is recommended that alternative intersection analysis be conducted solely using microsimulation software.

Having validated a test case for microsimulation analysis, the authors use this analysis technique to apply the junction delay measure developed for comparative analysis of the through-about intersection design with the roundabout and conventional signalized intersection designs. Secondary measures of effectiveness are discussed, including safety, cost, and fuel consumption. Three sensitivity analyses are conducted for this research, with a primary focus on the average delay of vehicles using the junction delay service measure. Though the through-about intersection design is found to not show significant improvements over the conventional alternatives in terms of overall intersection junction delay, it compared favorably to the other designs based on some of the secondary measures of effectiveness investigated, such as the average junction delay for through traffic on the major arterial. The ultimate conclusion of the comparative analysis is that a through-about design underperforms the conventional alternatives from many different perspectives, but it is uniquely able to safely accommodate a larger number of minor street approaches than the other designs, including approaches with large skew angles.

9.3 Planning Level Analysis

Capacity Analysis for the Planning of Junctions (CAP-X) is a planning-level tool for the comparative analysis of intersection designs developed by the Federal Highway Administration. This research documents the formulation of equations based on the critical sum method which are employed by CAP-X, and demonstrate the operational limitations of a variety of intersection designs as predicted by it. This research finds that although the tool scores highly for ease of use, the recommendations it generates from comparative analysis are not predictive of the recommendations found using microsimulation.

A number of discrepancies are identified between the equation formulations presented within the CAP-X tool, and the assumptions for signal functionality assumed in this research. Some of these errors appear to be due to typos in the software, while others are differences in interpretation of the signal phasing schemes at a given facility. The formulations presented within CAP-X are noted to produce results that are more conservative for the conventional signalized intersection and roundabout designs, in comparison to the assumptions made for this research.

The critical lane analysis method is found to be correlated fairly closely against the average delay values produced by the HCM method, at least for conventional signalized intersections. The addition of saturation flow rate adjustment factors for turning lanes is not able to reduce the amount of error observed in comparing the two measures, but it is observed to shift the correlation to the right by an appreciable amount. In defining the threshold at which an intersection ceases to function in an acceptable manner, the saturation-flow adjusted critical sum demands exhibited a transition from LOS D to LOS E at approximately 1,600 vehicles/hour/lane, with an approximately equal number of observations below 1,600 in LOS E, and above 1,600 in LOS D.

The slope of the failure thresholds for the geometric designs with one lane on each approach are observed to be generally parallel to the slope of the failure thresholds for geometric designs with two lanes on each approach. The condition with two lanes on the major roadway and one lane on the minor roadway is observed to consistently have a failure threshold with a shallower slope, indicating the intersections ability to serve higher volumes of major-street traffic flows, without improvements to service for minor-street traffic flows. In examining the comparative analysis of failure thresholds for the various designs, the roundabout is found to consistently be the least robust, with the restricted-crossing intersection being the second-least robust. The jughandle, median u-turn, and conventional intersections are seen to perform essentially equivalent to each other in terms of capacity failure, while the displaced left-turn design is predicted by this method to service the greatest demand volumes before failure is reached.

An in-depth check of the critical lane method is conducted using the Quadrant Roadway design, finding the tool to be an unreliable predictor of either vehicle delay or total travel time. Expanding on the quadrant roadway study, further work seeks to determine the degree to which the results from CAP-X are applicable. Preliminary results indicate that the CAP-X tool has significant limitations as a tool for choosing alternatives. The results show that critical sum

analysis does not generate a common failure threshold value for all designs. Moreover, the relationship between volume-to-capacity ratio and delay varies greatly across the alternative geometries investigated. While it may be possible to develop the individual capacity-to-delay relationships for each geometry, given further volume scenarios and analysis, such a transformation would need to account for both adjusted failure thresholds and expected delay measures in undersaturated conditions.

This research finds that the critical lane analysis technique is a powerful tool for understanding conceptual lane allocations for geometric design, and it provides a framework to insightfully show the theoretical benefits of alternative intersection designs. However, at present time it is being presented as a preliminary analysis tool for comparative analysis of potential intersection designs in consulting. Based on the small sample of volume scenarios analyzed for this research, the preliminary finding is that the v/c measures currently generated by CAP-X lead to incorrect conclusions regarding which designs should be moved forward for further investigation and design. By recommending inefficient designs for further analysis, the program may be hurting the potential for further implementation of alternative designs, the opposite of its intended purpose.

This research suggests that further analysis of the relationship between capacity and delay for these design alternatives may lead to the development of transformational equations that can supplement the current critical sum analysis. The incorporation of these capacity-to-delay transformation equations into a future version of the CAP-X software would preserve the positive aspects of the existing software, while increasing its utility both as a prediction tool and as an advocate for these innovative design solutions.

9.4 Recommendations for Future Research

Future research on the operational analysis of alternative intersections and interchanges may include the further development of the proposed planning-level analysis, additional exploration of signal-timing best practices for these designs, as well as the vetting of new design geometries as they are proposed. Moving beyond the operational analysis, a great deal of work stands to be done working with these designs to develop safety impact factors for them, best-practices for pedestrian and bicycle accommodations, and an exploration of potential environmental impacts, both positive and negative, associated with these geometries.

Developing a capacity-to-delay relationship for each alternative intersection and interchange design will require a significant amount of work. Details of a given simulation, such as the signal timings and location of u-turn bays, impart great impact on the experienced delays, while having no impact on the critical lane analysis values. As such, careful attention will need to be paid to each volume scenario of each design that is run to ensure that the relationship developed between the critical lane analysis capacity measure and the simulated or field-observed delay is based on optimized signal timings that generate the lowest possible vehicle delays. Each design type will need to be modeled at varying sizes, with different numbers of through- and turn-lanes on the various approaches, with a wide variety of demand volume scenarios for each geometric

scenario. The transformation equations from critical lane capacity measure to a measure of delay may only be appropriate for a given design if it is found to have a consistent relationship between capacity and delay regardless of how large it is, or what kind of demand volume scenario it is faced with.

Achieving the lowest-possible delay values for a given geometric and volume scenario will require extensive knowledge of the best practices for signal timing at each of these designs. Some designs such as the restricted crossing u-turn, reroute vehicles through multiple signalized intersections, and observed information regarding progression through these facilities will be critical to calibrating the appropriate signal timings for each situation. Adding to the complication for optimized signal timings for these designs is that the relative locations of the signals for each implementation is unique for each site, such as the length between crossover locations at the DDI or the distance from the main junction to the u-turn bays at a median or restricted crossing u-turn design.

A unique facet of the alternative intersection and interchange field of study is the potential to develop entirely new geometries, or reinterpret the current ones to develop new ways of transporting users safely and efficiently through a junction. New designs have been most effective, when they are developed to resolve a specific constraint, such as queued left-turning traffic on an overpass in the case of the diverging diamond interchange, or the need to provide access for abutters while maintaining major arterial throughput in the case of the restricted crossing u-turn design. As new designs are proposed, it is then the task of the research community to test out the potential for its use, and make recommendations on how and if to implement it in the field.

Although the bulk of research to date in the field of alternative intersections and interchanges has been focused on their operational performance for passenger vehicles, the breadth of research topics on these designs is expanding as our understanding of their vehicle performance is improving. These designs reduce and separate the number of vehicle-path conflict points, which generates a theoretical benefit to safety. As a growing number of these designs are implemented in the field, large-scale safety analysis needs to be conducted to validate the degree to which the theoretical conflict-point benefit translates into on-the-ground experienced safety benefits. The accommodation of pedestrians and bicycles at these designs is a topic of conversation that is growing among the research community, and the first preliminary studies are now coming out on this topic. If these designs are to be implemented more thoroughly on the surface transportation network, a better multi-modal understanding of them will need to be reached, with recommended practice for how to accommodate pedestrian movements within the signal timing schemes, and how to make safe and clear pathways for bicyclists wishing to traverse these junctions with vehicle traffic moving in somewhat unexpected ways. Another topic of interest not yet explored for these designs is how they impact the environment differently from conventional intersections. A potential topic to explore would be with designs such as the restricted crossing u-turn, which may increase the number of stopped vehicles while reducing the overall delay. How might this

change in traffic behavior impact fuel consumption – how would that impact differ if the market penetration of electronic vehicles increases and vehicles consume less fuel while stopped than they do now? It is hoped that the results of this research, combined with the future efforts of the research community, will expand our knowledge and embrace of alternative intersections and interchanges, and in the near future they will cease to be alternatives, and instead be the new conventional.

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